

IMPLICATIONS FROM A GEOTECHNICAL INVESTIGATION OF
LIQUEFACTION PHENOMENA ASSOCIATED WITH SEISMIC EVENTS IN
THE CHARLESTON, SC AREA

by

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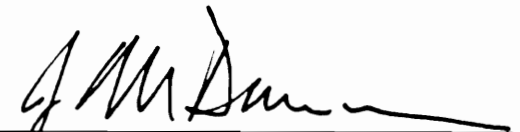
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(ABSTRACT)

First-hand accounts of sand boils and other liquefaction-related phenomena associated with the Charleston, SC earthquake of 1886 provide clear evidence that liquefaction was common in this event. Recent geologic investigations in the Charleston area have found evidence for the repeated liquefaction of sandy soils in the Charleston area due to recurring large seismic events. Although this information has led to an improved understanding of seismicity in the Charleston region, little hard data exists in terms of ground motion characteristics or levels of seismic loading. A two-year field

investigation was undertaken by Virginia Tech to study the liquefaction findings associated with the 1886 event from the perspective of geotechnical engineering. This involved defining the engineering parameters of the Charleston soils on the basis of in-situ and laboratory tests, and estimating the levels of seismic loading required to produce the observed liquefaction phenomena.

Of the sites where field tests were performed, the surficial soils were largely formed from ancient beach ridge deposits. The findings showed that soil conditions within these deposits are appropriate for liquefaction. Also, there is clear evidence that soils as old as 230,000 years have liquefied multiple times in the past 10,000 years. Many of these soils remain susceptible to liquefaction at relatively low levels of seismic shaking, although there is some evidence for progressive densification.

With respect to the seismic loadings, evidence is presented which suggests that both the magnitude and peak acceleration of the 1886 earthquake were less than what has been proposed by the seismological community ($M = 7.7$ and $0.5 - 0.6g$ peak acceleration). The findings of this study indicate that for an $M = 7.5$ event, peak accelerations in the 0.3 to $0.4g$ range would serve to explain the observed 1886 liquefaction phenomena. If it is assumed that the magnitude of the 1886 earthquake was less than 7.5 , then the estimated peak accelerations increase.

DEDICATION

This dissertation is dedicated to:

My best buddy, James R., I, (1918-1989), whose "Can't Touch This!" spirit lives within me; and,

My best friend and foundation, Dora T., who, unselfishly, continues to give her love and support; and,

The people of Cross Keys, South Carolina, who still allow me to come home when I come home.

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CHAPTER 1

INTRODUCTION

The largest historical earthquake in the eastern United States occurred on the Atlantic seaboard approximately 15 miles northwest of Charleston, South Carolina in 1886. The earthquake was estimated to have a Modified Mercalli Intensity (MMI) of X in the meizoseismal zone, and an MMI of IX in the City of Charleston. The surface wave magnitude (Ms) of the earthquake has been estimated at 7.7. Following the earthquake, extensive field reconnaissances were undertaken, with the most recognized documentation provided by Dutton (1889). First-hand observations by Dutton and others describe obvious liquefaction-related ground disruptions ranging from very large sand blows (or "craterlets") to minor ground fissures and sand vents. Of particular interest is that some of these features were unusually large compared to those observed in present-generation

earthquakes. Reported structural damage in the City of Charleston was extensive, and it is postulated that liquefaction-related ground failures were responsible for a significant part of this damage (Robinson and Talwani, 1983).

Recent paleoseismic studies have led to the discovery of relic liquefaction features in the Charleston vicinity as well as other areas along the coast of South Carolina (Obermeier et al., 1987). Liquefaction features could be found because of recent near-surface soil exposures provided by long drainage ditches, (common to the low-lying Charleston region) and large borrow pits excavated for the mining of sand. Dating of organics in the soil profile has shown that while some of the features are related to the 1886 event, many features were caused by pre-1886 episodes of ground shaking. The studies indicate that at least four episodes of liquefaction have occurred at sites in the Charleston region prior to 1886. Based on these findings, the recurrence interval for major seismic events has been estimated at about 500-1500 years for the coastal region of South Carolina.

While the geological findings have led to an improved understanding of Charleston area seismicity, it remains to study the South Carolina liquefaction discoveries from the perspective of geotechnical engineering. The present investigation was undertaken to study the engineering properties

of the soils associated with the liquefaction features, and to characterize and determine the liquefaction susceptibility of these soils. Of particular interest are:

- 1) The engineering characteristics of the sands from the liquefaction sites. Are they all equally subject to liquefaction, and are they in any way different in their seismic response relative to sands from other regions?
- 2) Variation in soil conditions from site to site that would suggest that more damage would result in one area than another if an earthquake should occur. This includes localized site response and soil amplification effects that may influence cyclic behavior.
- 3) The lateral extent and makeup of the key soil layers around the liquefaction sites, and the relation of this to the type of liquefaction features.
- 4) Evidence for progressive densification of sediments in areas where liquefaction has repeatedly occurred.
- 5) The level of seismic activity needed to cause liquefaction, as determined at sites where major, minor, and no liquefaction occurred in the 1886 earthquake.
- 6) The attenuation of seismic energy with distance during the 1886 event, as based on the accelerations estimated to be consistent with the 1886 liquefaction observa-

tions at locations close to and far from the source zone.

- 7) The relationship between the levels of seismic activity determined as per items (4) and (5) to those proposed by the seismological community.

The effort undertaken for this study involved a two-year investigation which included general field reconnaissance, researching of historical documents, drilling, Standard Penetration Testing (SPT) testing, Cone Penetration Testing (CPT) testing, compiling SPT borings and other soil data from commercial firms, and laboratory testing. The field testing program and data collection effort focussed on beach ridge deposits which are known to have the characteristics most appropriate for liquefaction. Testing was performed at sites within and outside of areas where liquefaction was prominent, and at different distances from the 1886 zone of energy release. Fifteen sites at which the extent of 1886 liquefaction was well-defined by first hand accounts or by U. S. Geological Survey personnel, were tested primarily with the CPT and to a lesser extent with the SPT. Six of the sites were located within the meizo-seismal region, while ten sites fell outside of this region. More than 35 auger holes, 57 CPT holes, and 6 SPT holes were performed by Virginia Tech personnel, and more than 2000 SPT borings were collected from outside agencies. Additionally,

a commercial firm was hired to perform six SPT's during the first year of the investigation.

Collectively, the soil data help to provide a framework of understanding for an environment where the seismic return rate is low, and where little hard data exists as to the source mechanism for the seismic shaking. The data also allow insight into the reasons for the types of liquefaction features found in paleoseismic studies, as well as inference into the level of accelerations necessary for their generation.

This report is organized to provide in Chapter 2 a background to the Charleston earthquake and the general subject of liquefaction. Chapter 3 presents the organization of the field investigations and testing program, while Chapter 4 gives details about the testing and testing procedures. Soil conditions determined on the basis of field and laboratory testing are detailed in Chapter 5. Chapter 6 presents liquefaction analyses of the soil data obtained from all sources, and discusses the implications of the results of these analyses. A summary and conclusions are outlined in Chapter 7.

CHAPTER 2

BACKGROUND

2.1 CHARLESTON AREA SEISMICITY

One of the most prominent areas of seismicity along the Atlantic Seaboard is in the vicinity of Charleston, SC. The Charleston earthquake of 1886 stands out as the largest seismic event to have occurred in this region during historic times. Following two days of several small tremors, the initial and strongest shock hit on August 13 at 9:51 p.m. with a duration of approximately 35 to 70 seconds (Dutton, 1889). The epicenter was located about 15 miles northwest of Charleston between the towns of Summerville and Middleton Place. All physical evidence and known facts about the earthquake indicate that a Modified Mercalli Intensity (MMI) of X was experienced throughout the epicentral region, while an intensity of IX was felt in the City

of Charleston. Damage throughout the Charleston region was extensive, but the population density, predominance of single story wood frame houses in the epicentral area, and the timing of the earthquake contributed to a reduction in the loss of life (60 deaths were reported). As illustrated on the intensity map in Fig. 2.1, the motions of the earthquake were felt over a large part of the U.S., and were reported to have caused "lamps to sway" in Cuba and "dishes to topple from shelves" in New York (Dutton, 1889).

Using relationships involving the attenuation of intensity with epicentral distance, Bollinger (1977) and Nuttli et al. (1979) have estimated the magnitudes of seismic shaking as $m_b = 6.6-6.9$ and $M_S = 7.7$; peak ground accelerations were placed in the range of 0.6g. In a more recent study, Chapman et al. (1989) have estimated the range of near-surface peak accelerations of the 1886 event to fall within 0.4 to 1.0 g. Although these magnitudes are indicative of strong ground shaking, no surface fault rupture was observed during the 1886 event. One explanation lies in the possibility that the hypocenter lay at a depth of 12 miles or more (Nuttli et al., 1986). Based on first-hand descriptions of the ground motions and intensity reports within the meizoseismal zone, it has been inferred that the energy release of the earthquake was initiated near Dutton's southwestern epicentrum and propagated toward the northeastern epicentrum (Nuttli et al., 1990); see Fig. 2.2.

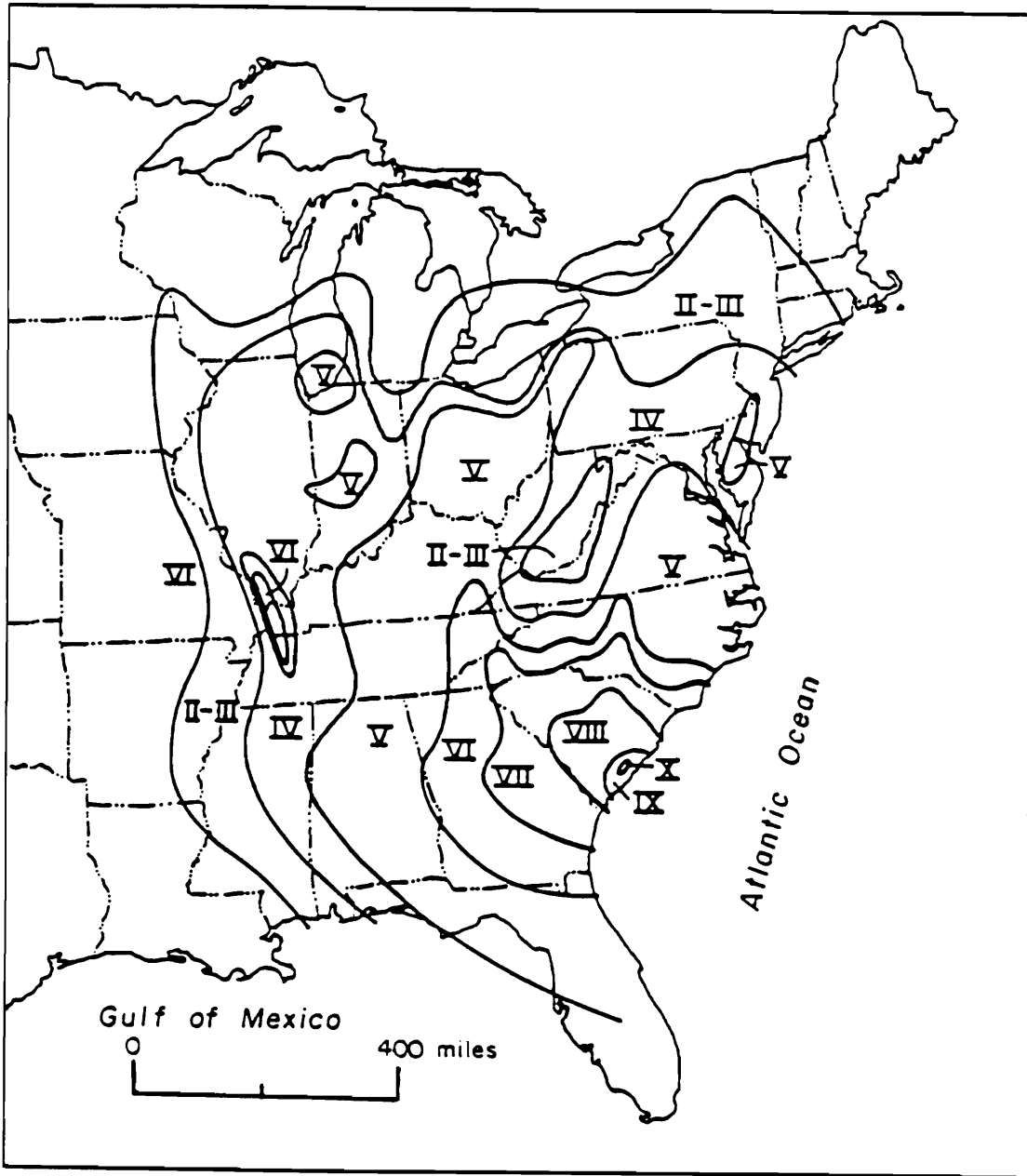
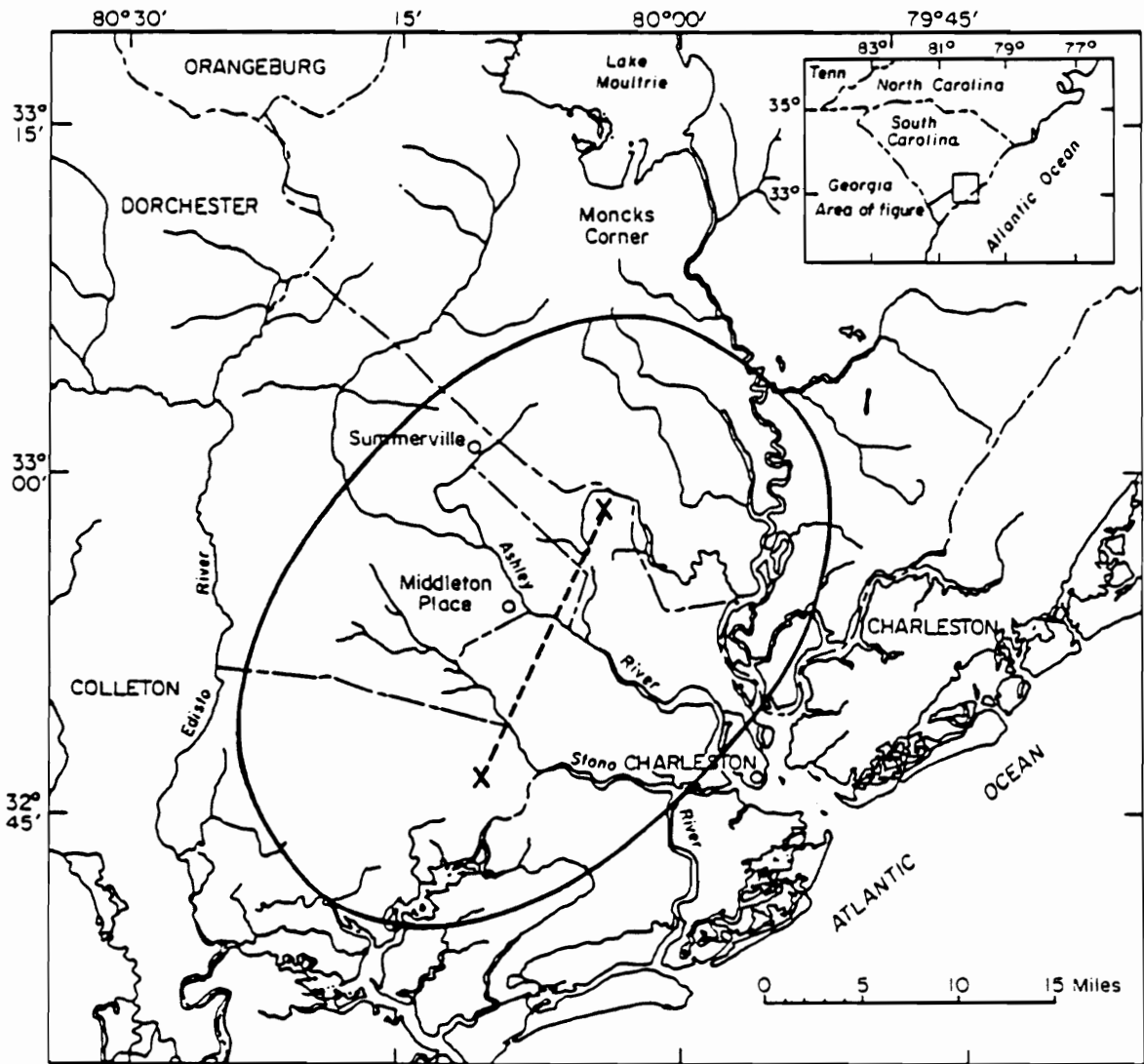


Figure 2.1 Isoseismal Map Showing Intensity Patterns for the 1886 Charleston Earthquake (Bollinger, 1977).



- X** Centers of highest intensity of 1886 earthquake (Dutton, 1889)
- Approximate boundary of meizoseismal zone (1886) (Bollinger, 1977)
- - -** Inferred linear zone of energy release

Figure 2.2 Map of Area Near Charleston, SC Showing the Meizoseismal Zone and the Inferred Zone of Linear Energy Release

A line connecting the two centers of highest intensity can be interpreted to be the surface projection of the fault rupture plane and linear zone of seismic energy release.

Small earthquakes continue to be felt throughout the Charleston region, although the overall level of seismic activity is relatively low, and consistent with its intraplate tectonic setting (Amick et al., 1989). While the cause of the 1886 event is still speculative, recent investigations have led to an improved understanding of the motion patterns and possible source mechanisms (Bollinger, 1977; Talwani, 1982). Our current knowledge indicates that the source of the seismicity near Charleston originates from one or more deeply buried and probably intersecting fault zones, although irrefutable evidence supporting a specific model has not been presented (Weems and Obermeier, 1989).

2.2 GEOLOGIC SETTING OF THE CHARLESTON AREA

The coastal plain region of South Carolina is an area of minimal local relief (3 to 10 ft.) and low ground elevation (3 to 35 ft. above MSL in the study area). Locally this area is referred to as the "Low Country" and is characterized by vast expanses of swamps and marshlands traversed by a series of high-ground ridges of Quaternary marine and fluvial sediments. The marine sediments form a

series of six, well-defined, interglacial beach ridges with associated back-barrier and shelf deposits which trend roughly parallel to the present shoreline. The geologic map given in Fig. 2.3 shows the locations of these deposits which extend along the South Carolina coast from North Carolina to Georgia. The beach deposits are extensive and occur as contiguous deposits except where cross-cut by fluvial sediments. Individual segments have reaches of up to 40 miles and widths of 5 miles. The beach ridges were formed such that they progressively increase in age and ground elevation with distance inland from the present shoreline (northwest direction); the oldest deposits are situated farthest inland and at the highest elevations. The inland limit of the marine deposits is demarked by the oldest beach ridge located approximately 35 miles from the shore.

Color and letter designations are used in Fig. 2.3 to indicate the ages of the various geologic units. The present beach system is outlined in yellow and given a Q1 designation, while the areas shown in purple and designated as Q2, represent older beach sediments of about 85,000 years in age. The 85,000 year-old ridge is located roughly five miles inland. The pink Q3 deposits range from 130,000 to 230,000 years in age and are about 10 miles from the shoreline. The next three inland ridges, designated as Q4, Q5, and Q6 respectively, are all older than 700,000 years; these

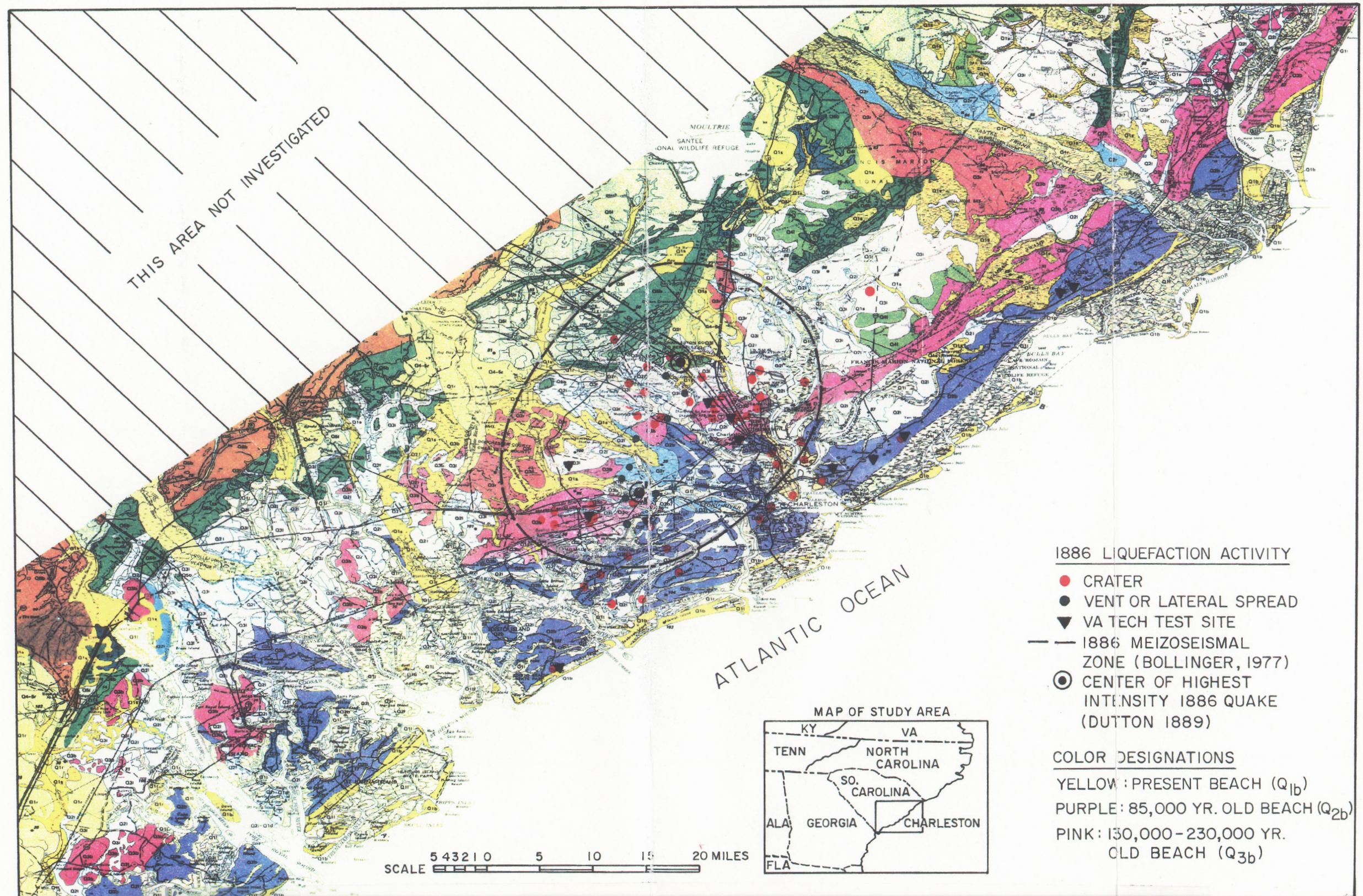


Figure 2.3 Geologic Map of Coastal South Carolina Showing Locations of Documented 1886 Liquefaction Features (map from McCarten et al., 1984).

deposits are typically clay-rich, cemented and consolidated (due to weathering), and positioned high above the water table (Obermeier, 1989). Because the likelihood of liquefaction occurring in these deposits during the Holocene and late Pleistocene is quite low, the present study is restricted to the more recent sediments designated as Q1, Q2, and Q3. These areas generally correspond to the non-hatched region in Fig. 2.3.

The geologic processes forming the beach ridges led to the deposition of fine to silty sands in the upper portions of the ridges near the crests. The back-barrier areas flanking the crests are typically 5 to 15 ft. lower in elevation and consist primarily of organic silts and clays. Fluvial deposits in the low country area typically consist of medium to coarse sand sediments interbedded with clays. The groundwater table throughout most of the region is found within three feet of the ground surface, except where lowered by man's activities, such as in the presence of drainage ditches. Soil studies conducted throughout the region on the organic-rich B_h horizon (the base of which denotes the maximum depth of the seasonal water table) indicates that the water table has been at this depth throughout the Holocene (Obermeier et al., 1985).

The setting most frequently associated with earthquake-induced liquefaction is located at the crest or flank of a Pleistocene beach ridge, where a thin cover of clayey sand

or humate-rich soil overlies uniform clean fine sands and the groundwater table is high. As stated by Sloan in the first-hand accounts of the 1886 earthquake, "craterlets are found in greatest abundance in belts parallel with (beach) ridges and along their anticlines" (Sloan in Peters and Herrmann, 1986). Fig. 2.4 shows a schematic cross-section through a low-country beach ridge typical of that referred to by Sloan. As depicted in the figure, the lee side of the ridges is structured more favorably for sand blow formation. Back-barrier and fluvial deposits have been associated with such features to a much lesser extent (Obermeier et al., 1987).

2.3 FIRST-HAND OBSERVATIONS

Following the 1886 earthquake, citizens throughout the Charleston area inquisitively observed the venting of colorful sands and clays and detected emissions of sulphurous gases throughout the epicentral region. At one location in Charleston, a citizen observed a sandblow venting up through an old 25-foot deep fire well as, "spouting up a solid column of water over two feet in diameter, to a height of fully ten feet. When I examined the well the next day, I found it nearly full of white sand." (Stockton, 1986). Newspaper reports describing violent upheavals of sand and

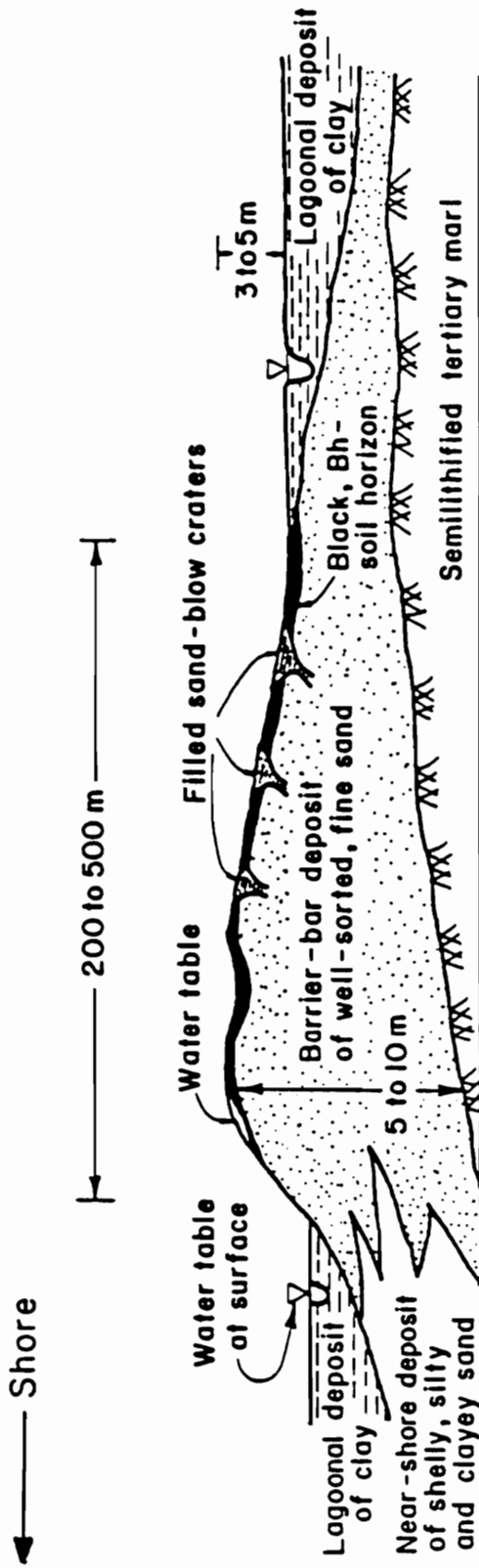


Figure 2.4 Schematic Cross-Section of Typical Pleistocene Beach Ridge (Obermeier et al., 1987)

clay from orifices having shapes of inverted cones gave locals the impression that the earthquake was volcanically induced; a newspaper account (The News and Courier, 1886) from Mt. Pleasant supported this popular belief:

Organic material was found scattered about in large quantities that a local scientist says of necessity must have come from a depth of 1,000 feet. Attempts were made to sound the cavities with sticks and poles, some of which were over twenty feet in length, but no bottom was found in any of them.

Immediately after the earthquake, personnel from the U.S. Geological Survey and the U.S. Ordnance Corps travelled to the area and, with assistance from several Charlestonians, began documenting the structural damage and features of geologic interest caused by the ground motion. Seismically induced liquefaction features superficially expressed as fissures, vents, or sandblows that expelled sand and what is described in the terminology of the time as clay, were widespread in the loose sandy soils throughout the low country area (Bollinger, 1977; Peters and Herrmann, 1986; Stockton, 1986). In his account of the 1886 earthquake, Dutton (1889) reported that:

... water was extravasted in large quantity, some point in the line of the fissure would be often enlarged by the rapid flow or outrush of water into a round hole of considerable size, with a crater-basin at the ground surface. These craterlets were of all dimensions, from the most diminutive up to 20 feet or more in diameter... As a general rule, the water brought up great quantities of sand and silt... The quantity of water discharged during the earthquake was so great, that every stream-bed, even though ordinarily dry in summer,

was awash. It was asserted by many of the residents in some parts of the epicentral tract that the waters were spouted upwards to great heights... That here and there it was thrown up in jets to a height of fifteen or twenty feet is rendered probable by finding the sand and mud smirching the limbs and foliage of trees overhanging the orifices.

The primary areas of documented liquefaction were along the South Carolina Railroad (SCRR) tracks at Ten Mile Hill and along the historic Charleston and Savannah Railroad near Ravenel and Hollywood, SC. In these regions, sandblow craters of up to 20 feet in diameter and rifted vents up to 2000 feet long were found with surrounding land covered with expelled water and sands to depths of up to two feet (Peters and Herrmann, 1986). The photograph in Fig. 2.5 illustrates a large crater located along the South Carolina Railroad line near Ten Mile Hill. The photograph in the figure was taken a few days after the 1886 event. Craters of this size were exceptional, and were generally confined to the vicinity of Ten Mile Hill. The crater shown above was located at a site adjacent to the new Charleston International Airport/Air Force Base. After an extensive reconnaissance, Sloan (in Dutton, 1889) placed approximate boundaries on the areal extent of craterlets caused by the earthquake; see map in Fig. 2.6. It is instructive to note that while spectacular expressions of liquefaction were observed throughout the epicentral tract following the 1886 quake, a significant amount of the structural damage in the City of Charleston has been attributed to liquefaction-related ground failures

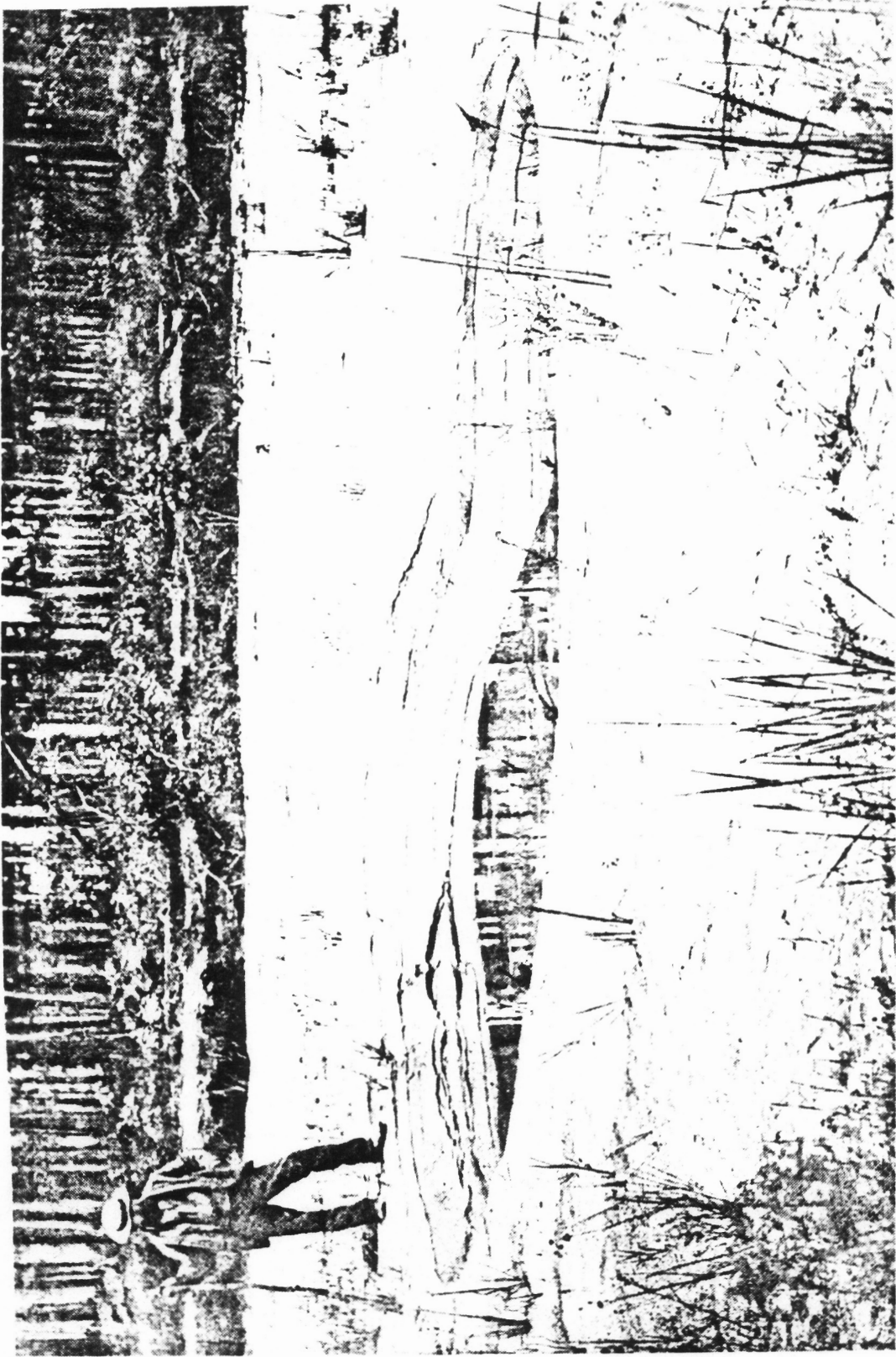
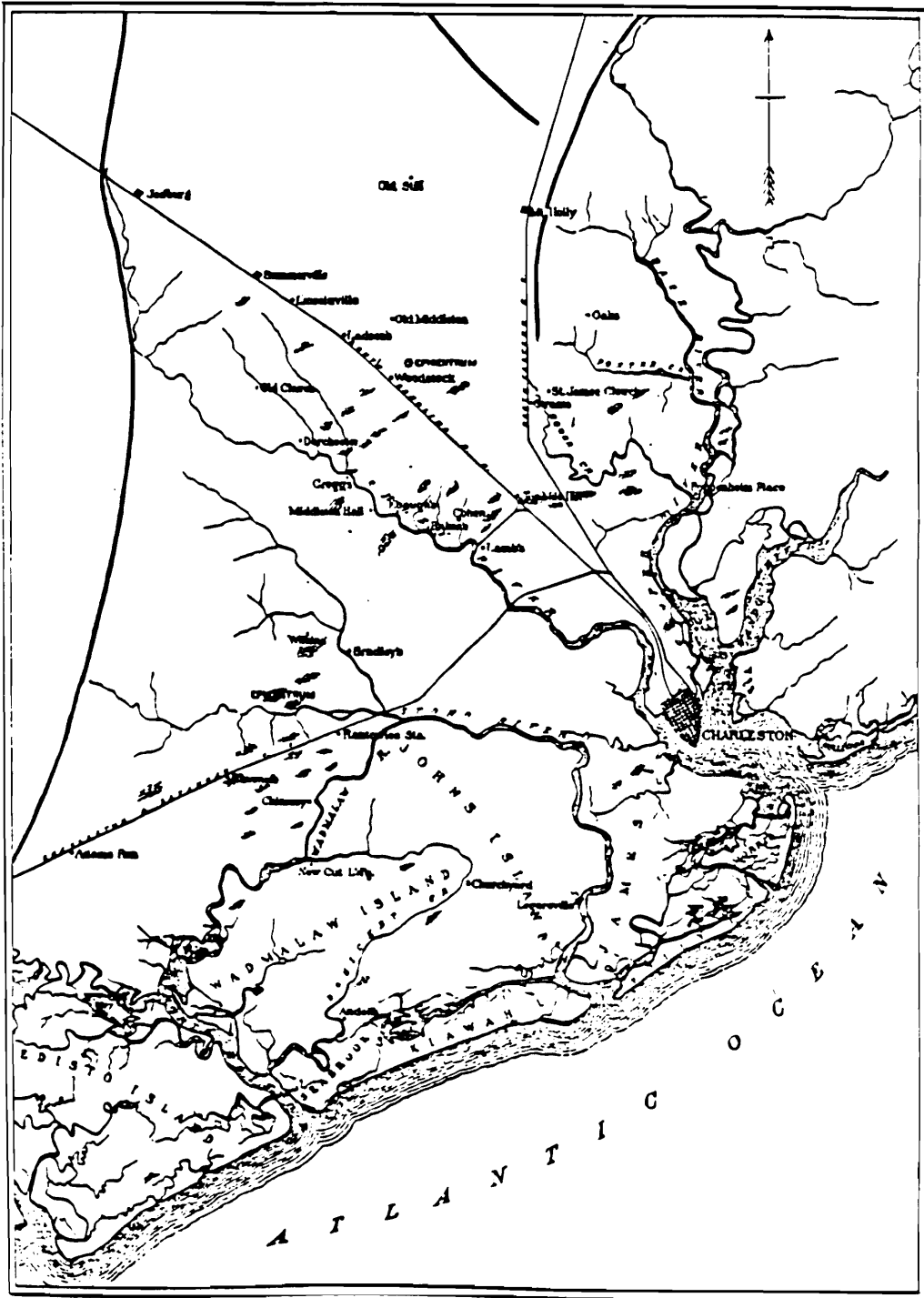


Figure 2.5 Large Sand Crater Located Near Ten Mile Hill Following 1886 Quake.



(From Dutton, 1889)



-  Line limiting zone of craterlets
-  Area conspicuous for craterlets



Figure 2.6 Distribution of Craterlets Formed During the 1866 Charleston Earthquake (Dutton, 1889).

in both natural ground as well as man-made fills (Robinson and Talwani, 1983).

Because the 1886 observers were diligent in describing features such as "sand mounds, fissures and cracks which issued water" -- obvious manifestations of liquefaction -- the reliability of the historical record is considered high. Several key circumstances add to this reliability: First, although liquefaction at some sites undoubtedly occurred at depth and manifested no surficial expressions, the historical observations (of surficial expressions) still represent an important relative measure of how much liquefaction occurred throughout the low country region. This idea is important in that the soils associated with liquefaction throughout the Charleston area apparently have similar susceptibilities to liquefaction (Dickenson, et al. 1988; Obermeier, 1989). Second, a series of thorough reconnaissances were performed by a team of official observers after the 1886 quake. The locations of reported liquefaction phenomena were well-referenced and included sketches or photographs of some of the more spectacular events. Recent findings of paleoliquefaction studies by workers at the U.S. Geological Survey support these observations (Obermeier, 1987). Many of the official observations were performed along and referenced to points along railroads and roadways which trended along the high-ground crests of old beach deposits. Also, many of the present-day transportation

routes are identical to the older routes which meant that the locations of numerous historical observations could be re-established with reasonable accuracy. Inasmuch as the ancient beach deposits are vulnerable to liquefaction, it seems likely that most expressions of significant liquefaction would have been observed, and that the historical record is particularly meaningful in documenting the extent of prominent liquefaction.

The map in Fig. 2.3 shows the location of all reported liquefaction features from the 1886 event, as well as 1886 features recently unearthed by U.S. Geological Survey workers. All site locations are shown in relation to Dutton's centers of highest intensity and Bollinger's meizoseismal zone. The map shows that the occurrence of liquefaction in the 1886 quake was primarily associated with the ancient marine deposits, although significant features were also located in alluvial sediments.

2.4 PALEOSEISMIC INVESTIGATIONS

The surficial manifestation of seismically induced excess pore pressures at depth can range from minor sand boiling and venting to large "explosive" sand blows. Both mechanisms involve fracturing and displacement of the overlying soil, followed by the extravasation of liquefied sand

and water. The surficial expressions of craters or vents are eventually obscured by infilling, weathering, and material collapse from their walls; but a distinctive morphology, which cuts through the surrounding undisturbed soil, is preserved within the soil profile. Figs. 2.7 and 2.8 illustrate this morphology for typical explosive and non-explosive type features. The schematic in Fig. 2.7 depicts a prehistoric "explosion" crater discovered at Hollywood, SC, by S. Obermeier (see Obermeier et al., 1987), while that shown in Fig. 2.8 illustrates a typical vent feature.

Recent discoveries of relic liquefaction features in the low country region of South Carolina have expanded knowledge about seismicity in that area. Numerous liquefaction features, attributed to the 1886 earthquake as well as other episodes of Quaternary ground motion, have been discovered in the walls of drainage ditches and sand pits in the low-lying region around Charleston (Obermeier et al., 1985; Sims, 1975; Sieh, 1978; Thorson et al., 1986; Talwani and Cox, 1985). The collection of liquefaction expressions identified by researchers at the U.S. Geological Survey include (Weems and Obermeier, 1989): 1) sand craters; 2) sand volcanos; 3) ground shattered by sand intrusions; 4) V-shaped sand intrusions; and, 5) blocks on level ground with opposite facing shears caused by back and forth oscillations.

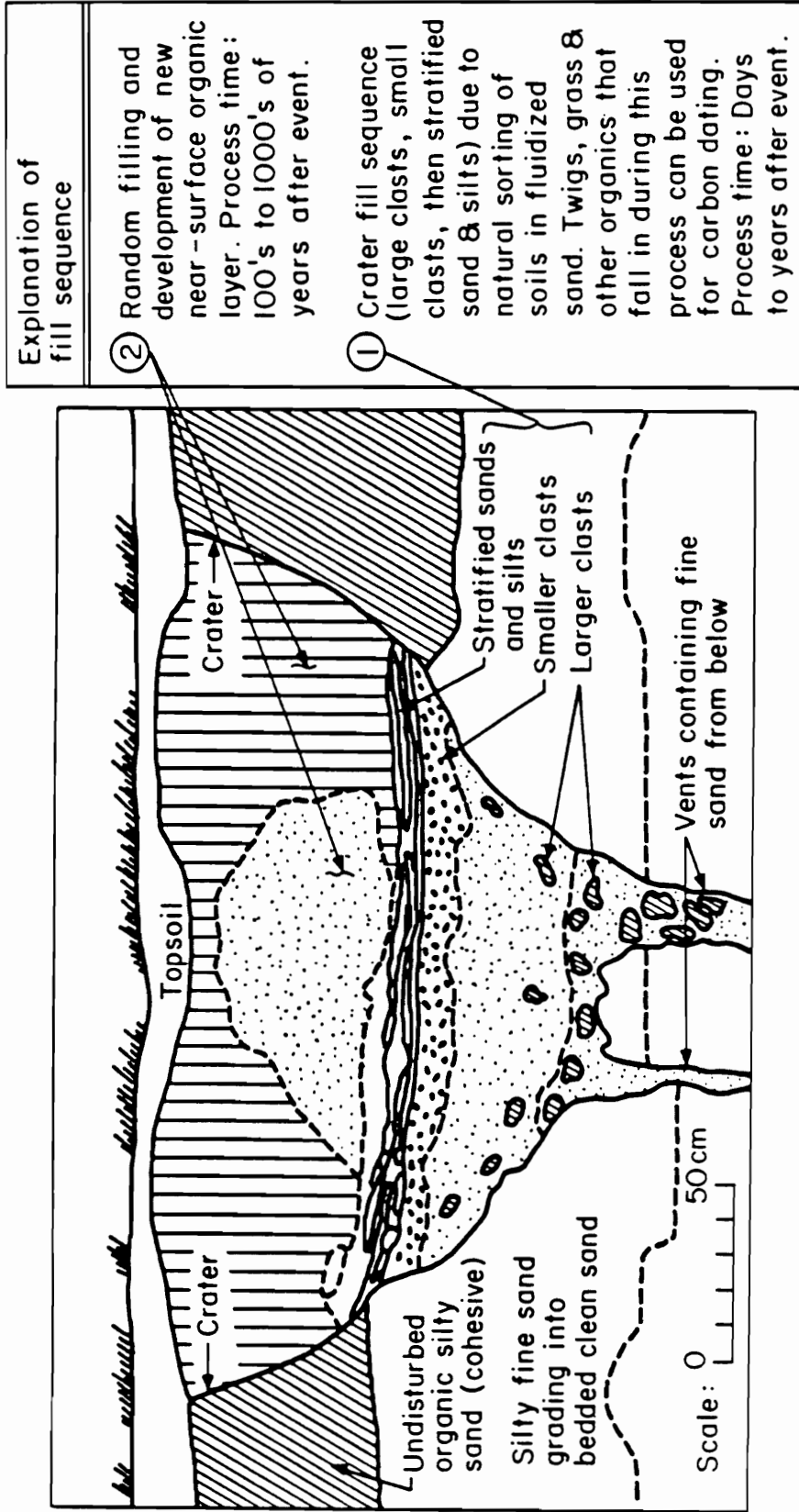


Figure 2.7 Schematic Cross-Section of Filled Prehistoric Sand Crater from Hollywood, SC (Obermeier et al., 1987).

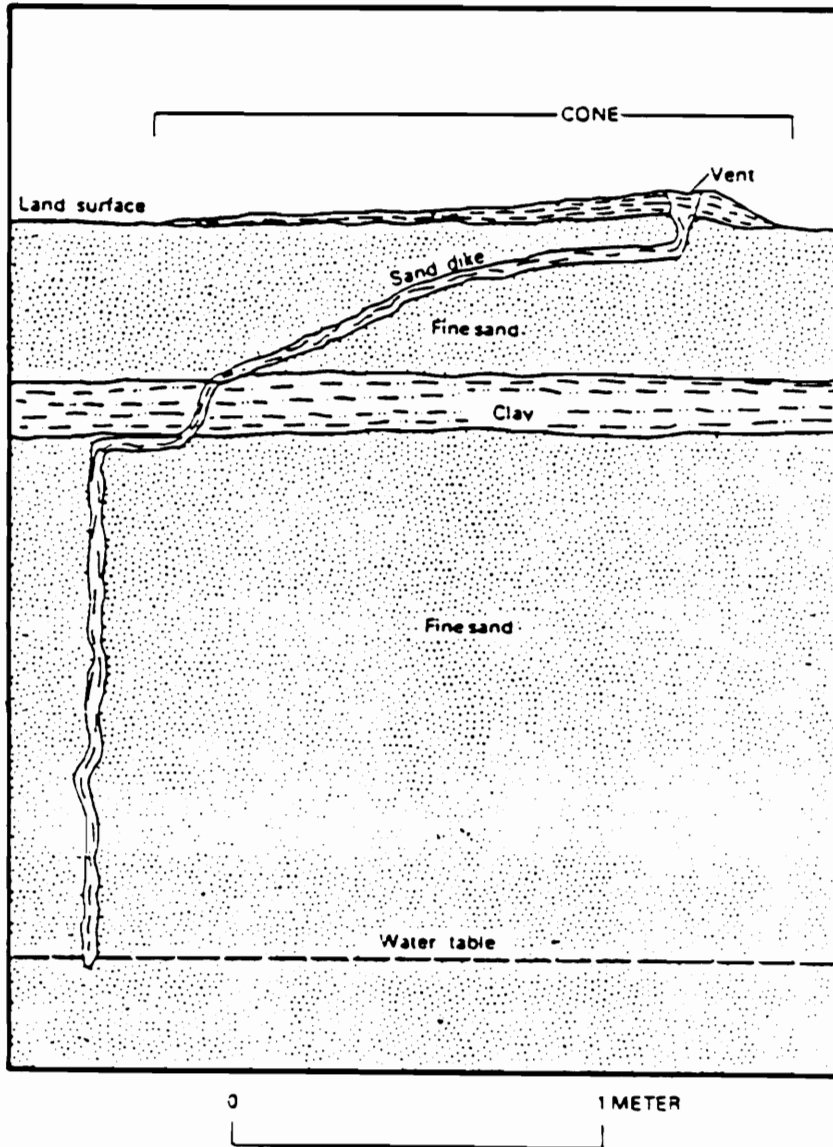


Figure 2.8 Schematic Cross-Section of Typical Vent Feature.

Radioisotope (C^{14}) dating of infilled organic matter found within these features has indicated a recurrence of large seismic events ($m_b > 5.5$) in the Charleston area. Our knowledge of the rate of this recurrence is being steadily refined as more field work is performed. Based on recent information, Weems and Obermeier (1989) report that:

- 1) The most compelling evidence for the recurrence of liquefaction-inducing earthquakes is for 1500-1600 years, but recent discoveries can be interpreted to support a recurrence interval as low as 500 to 600 years.
- 2) Earthquake-induced liquefaction in the Charleston area has been occurring intermittently for at least the last 30,000 years.
- 3) Ground shaking throughout the mid to late Holocene (last 5000 to 6000 years) has been stronger near Charleston than elsewhere in the South Carolina Coastal Plain region; and the source of the shaking for large prehistoric quakes near Charleston has been located within the 1886 meizoseismal zone.

Recent paleoliquefaction studies by other researchers (Amick et al., 1989) support these convictions, but suggest that at least one episode of significant ground shaking was centered away from Charleston during the last several thousand years. This possibility has also been suggested by Obermeier (1989).

2.5 MECHANISM OF LIQUEFACTION

The liquefaction of a loose, saturated granular soil occurs when cyclic shear stresses passing through the soil deposit induce a progressive increase in excess hydrostatic pore water pressure (Seed and Idriss, 1971; Ishihara, 1985; National Research Council, 1985). During an earthquake the cyclic waves that propagate upward from the underlying rock formation induce the tendency for a loose sand layer to decrease in volume. Cyclic shear straining and the volume contraction associated with collapse of the metastable structure of sand cannot occur simultaneously because the duration of shaking is much faster than the time necessary for drainage of pore water. This process is diagrammatically represented in Fig. 2.9. If undrained conditions during the seismic disturbance are assumed, an increase in pore water pressure and resulting decrease of equal magnitude in the effective confining stress is required to keep the loose sand at constant volume. The degree of excess pore water pressure generation is a function of the initial density of the sand layer and the level and duration of seismic shaking. If the layer is initially loose enough, pore pressures can be generated which exceed the confining stress. At this state no effective stress, or interangular

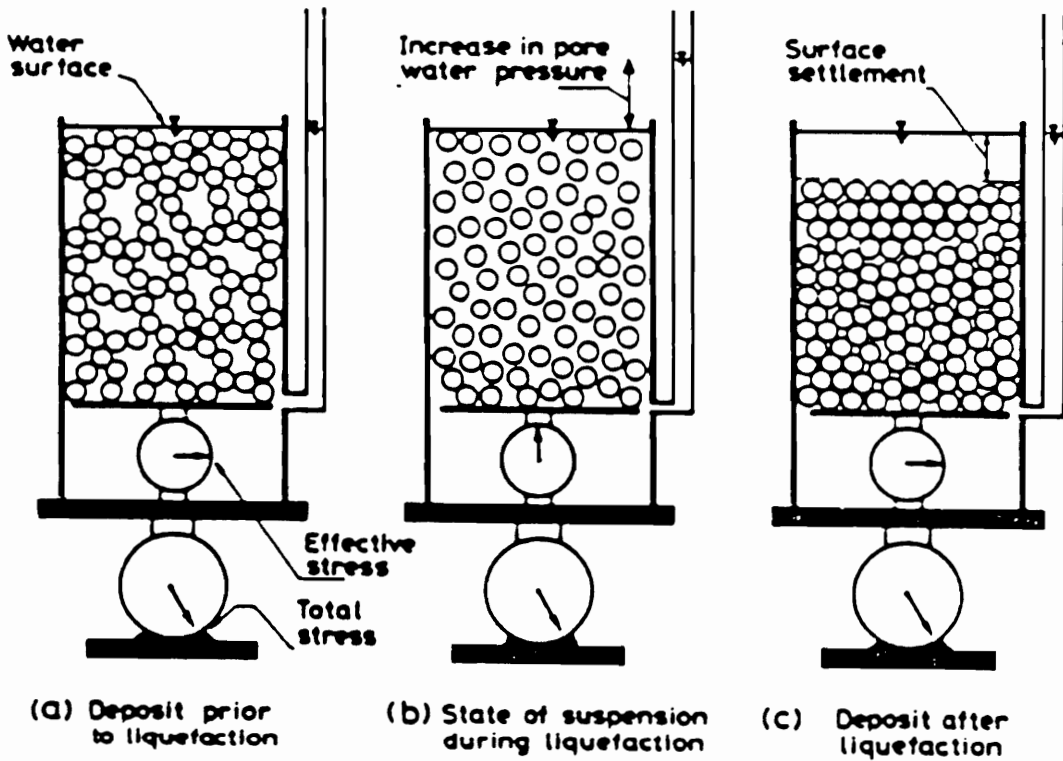


Figure 2.9 Stress States During Liquefaction of a Loose Sand (Ishihara, 1985).

stress, exists between the sand grains and a temporary complete loss of shear strength is experienced (Fig. 2.9 (b)). This loss of strength is termed liquefaction, after which settling of the sand grains causes an expulsion of pore water towards the surface.

Even though a condition of zero effective stress develops in a sand during cyclic loading, the sand may still exhibit considerable resistance to shear during a subsequent undrained loading. Castro and Poulos (1977) have used the critical void ratio and steady-state strength concept to demonstrate for medium sands that although initial liquefaction ($\sigma' \simeq 0$) can occur as a result of cyclic loading, no large displacements are experienced. This has been termed cyclic mobility and corresponds to the limited shear strain potential of Seed (National Research Council, 1985). Cyclic mobility can lead to excessive ($\simeq 10\%$) permanent deformations but not instability due to the tendency for sands to the left of the critical state line (Fig. 2.10) to dilate under loading following initial liquefaction, thereby reducing the excess pore pressure. In this context, undrained cyclic straining of very loose sands permits the particles to rearrange so that their cyclic resistance is less than their undrained steady-state strength (similar to residual strength). This behavior is "true liquefaction" and results in a collapse of the sand structure and large deformations. A thorough review of liquefaction and cyclic

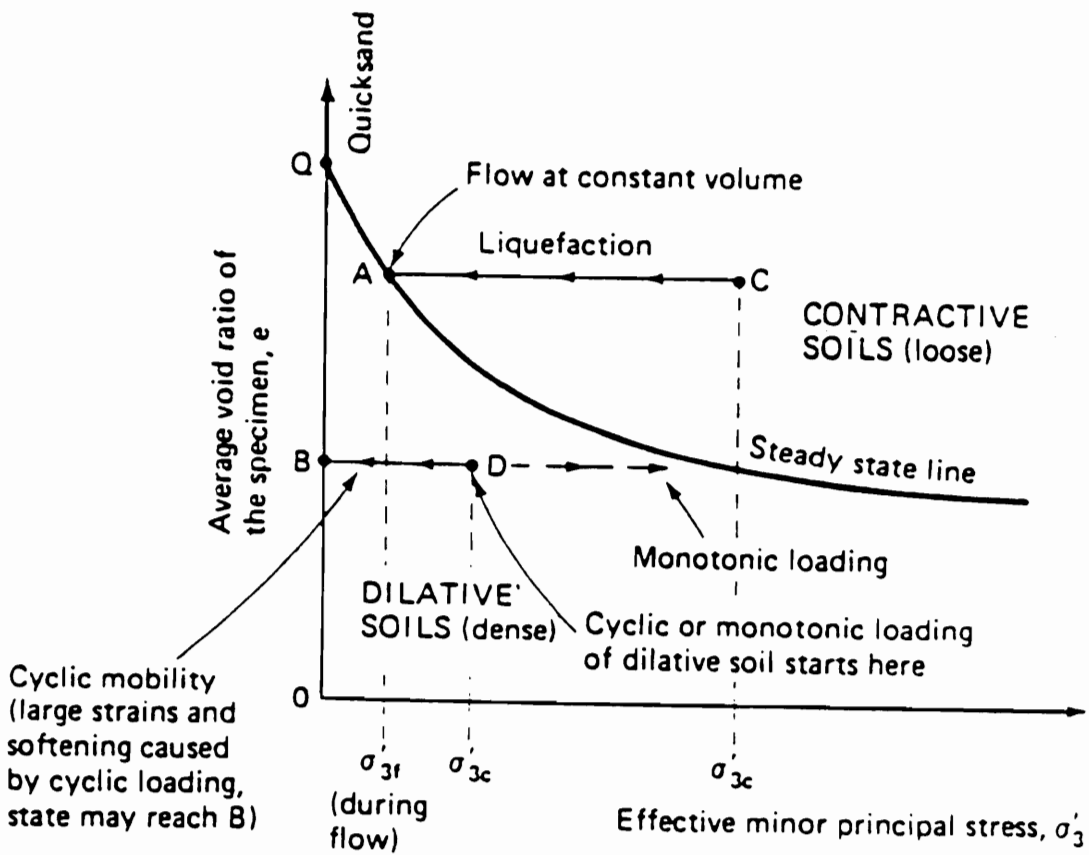


Figure 2.10 State Diagram Showing Liquefaction Potential Based on Undrained Tests of Saturated Sands (Castro and Poulos, 1977).

mobility is contained in the National Research Council report on liquefaction (1985). The formation of a sandblow or vent requires the generation of excess pore pressures and large strains which can only be realized during liquefaction of loose sands. This consequently greatly overshadows the contribution to increased pore pressures developed during cyclic mobility of dense sands which may be interlayered with the loose material.

2.5.1 Formation of Liquefaction Features

Sand boils, vents and blows are indicative of liquefaction at depth. Sand boils are formed by water venting to the ground surface from zones of high pore water pressure generated at shallow depths by the tendency for compaction of granular soils during seismic shaking, as previously described. The surficial expression of liquefied material is primarily dependent on the intensity of seismic ground motions, as minor venting of mud and sand may be visible at a MM intensity of VIII, become notable at IX, and large and spectacular at X (Richter in Bollinger, 1977). Based on field observations, a magnitude (m_b) of 5.0 has been presumed as the threshold value for liquefaction-induced ground failures (Youd and Perkins, 1978). Magnitudes of 5.5 and 6.2 have also been postulated as values required for the initiation of liquefaction in specific regions (Talwani and Cox, 1985; Weems et al.,

1986). A value closer to the former would appear likely for the very loose sands in the Charleston area (Obermeier, 1987).

The surface manifestation of these features is also governed by both the extent of the liquefiable layer and any overlying liquefaction resistant layers. If a liquefiable layer is overlain by a thin mantle of liquefaction resistant material that can be easily fractured or heaved, the extrusion of fluidized sand will occur as mild sand boiling and fissuring. The presence of a thick overlying, cohesive layer will require large pore pressure generation to hydraulically fracture this overburden. Venting through non-cohesive soils may involve different mechanisms. Following the initiation of liquefaction, a cavity of water can form at the base of the resistant non-cohesive material, which migrates toward the surface by continuous sloughing from the roof of the cavity (National Research Council, 1985). If the cavity approaches the surface, the pressurized, soil-laden water in the cavity can break through almost explosively to form a sandblow. This behavior can result in the formation of sandblows which reach the surface several minutes after the cessation of strong ground motions. In cases where thick overlying layers have been fractured, the sandblows are found to be very large and fewer in number because the breakthrough of the first vents release pore pressures thereby inhibiting concurrently forming cavities.

Based on field observations at sites in Japan and China where seismically induced liquefaction features were and were not apparent, Ishihara (1985) has postulated relations between the thicknesses of the liquefiable and overlying non-liquefiable layers and the surficial exposure of vented material. Fig. 2.11 demonstrates the effects that the layer system and ground motion intensity have on the recurrence of observed ground failure. In the Charleston environment, it appears that evidence of liquefaction at depth can be completely masked regardless of the thickness of the liquefiable layer if the surficial resistant layer is greater than about 10 ft. in thickness (Obermeier et al., 1986). This observation is consistent with Ishihara's findings that liquefaction is generally not observed at the ground surface if the capping layer is 10 ft. or more in thickness.

In the first hand accounts of the 1886 earthquake are descriptions of liquefaction features ranging from small vents to spectacular 20-foot diameter sandblow craters formed during pore pressure releases. Strong venting of fluidized sand is favored when the liquefiable layer is capped by a thin layer of non-liquefiable, cohesive material. In this situation, excess pore pressures in the liquefiable layer exceed the strength of the capping layer and a sand blow crater surrounded by an ejection sheet of liquefied sand and overlying material is formed. The development of a sand blow crater is shown in Fig. 2.12 (Gelinis,

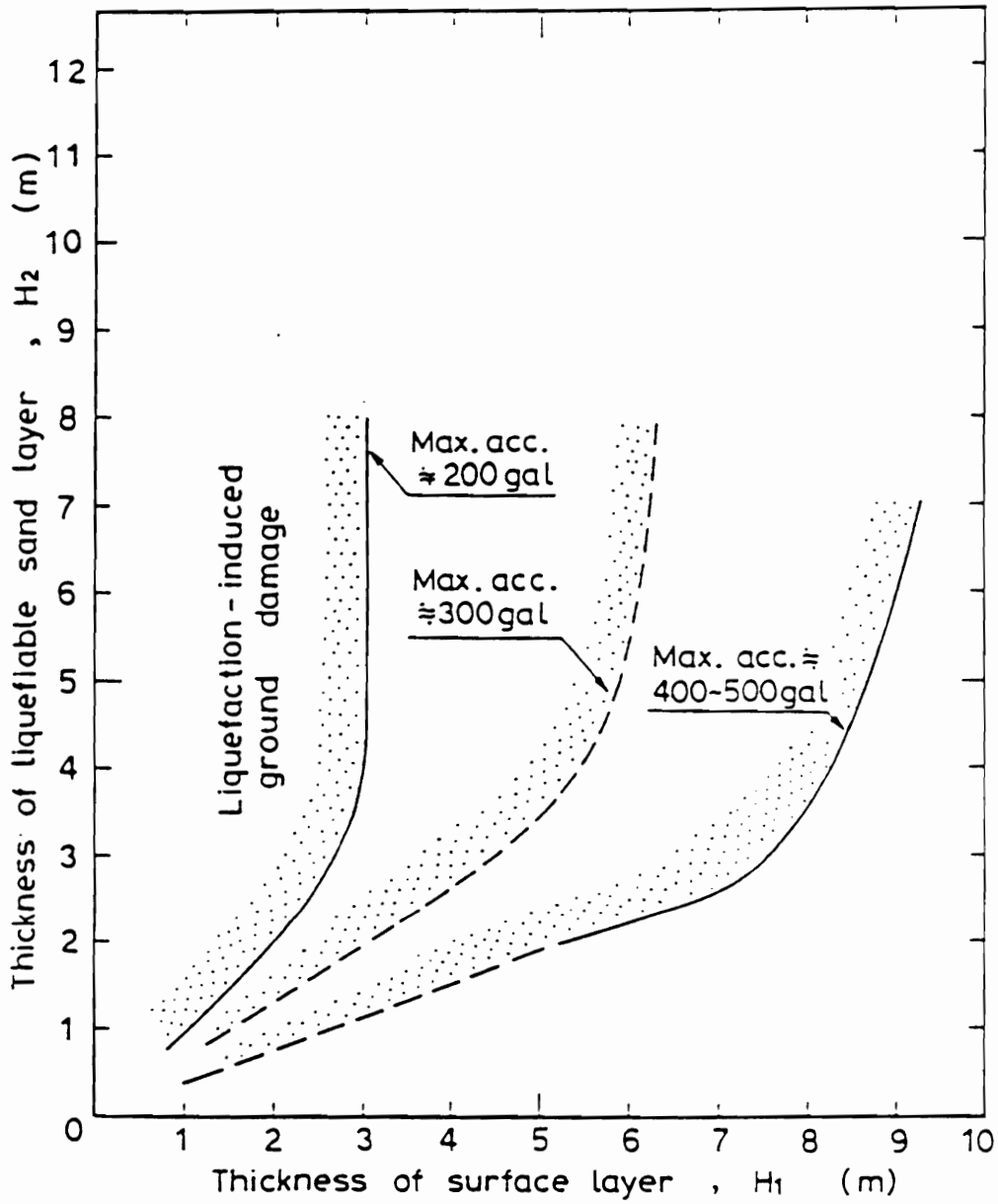


Figure 2.11 Boundary Curves for Site Identification of Liquefaction-Induced Damage (Ishihara, 1985).

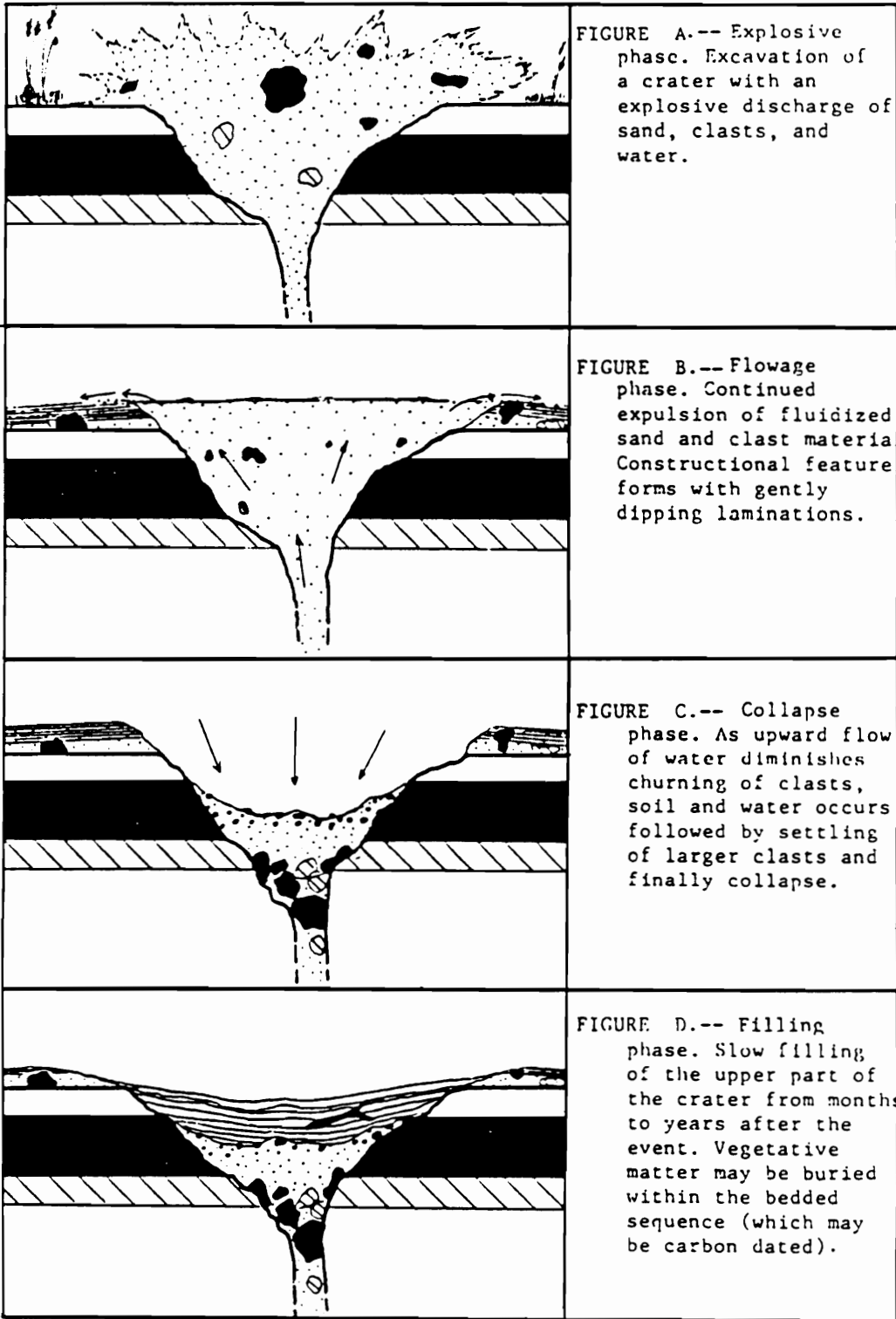


Figure 2.12 Sequential Development of a Sandblow Crater (Ishihara, 1985).

1986). Numerous moderate to large relic sandblow features having similar morphology to those described in Dutton (1889) have been discovered in the extensive network of drainage ditches in the Charleston region.

2.5.2 Recurrence of Liquefaction at the Same Site

Although the contraction of a saturated, loose sand during cyclic loading and resulting dissipation of pore water leads to a global increase in density of the layer, it is possible for a portion of the layer to be redeposited in a configuration that is as loose or looser than the initial arrangement. Laboratory triaxial and simple shear tests on Ottawa sands at relative densities between 37 and 70% have shown that 15 loading cycles of shear strains above a threshold value of approximately 0.5% create a structure in the sand sample which is significantly more sensitive to liquefaction than the structure created by consolidation (Finn et al., 1970). These tests also indicated that loadings of more than 15 cycles of shear strains at $\pm 2.0\%$ have a reducing effect on the resistance of sand to liquefaction in future cyclic loading tests. The range of single amplitude, cyclic shear strains where pore pressures become equal to the initial confining stress under laboratory cyclic loading has been found to fall between 2.5 and 3.5% (Ishihara, 1985). An average value for cyclic shear strain of 3.0% is generally used as a criterion to specify the

onset of liquefaction for loose sand, and is large enough to greatly reduce the liquefaction potential of a natural deposit in future earthquakes.

Laboratory re-liquefaction studies are supported by field observations at numerous sites which have experienced repeated liquefaction (Youd, 1984). Paleoseismic investigations in the Charleston region have also located several sites where repeated liquefaction of loose sands has occurred during several episodes of seismic ground motion. Several mechanisms could account for the retained low resistance to cyclic shearing of a liquefied sand layer. Youd (1984) has postulated that the liquefaction of a uniform layer begins at the upper portion where confining stresses are the lowest and propagates downward. The following compaction process begins at the base of the layer and proceeds to the top. If settling of the sand grains occurs after strong ground motions have ceased, it is possible to create a loose zone at the top of the layer which is repeatedly liquefiable until the entire layer has been progressively densified. Another condition may exist when liquefied layers are capped by impervious layers that inhibit the vertical release of pressure. In this situation excess pore pressures are either dissipated laterally throughout the layer, or are reduced by a readjustment of void ratio in which the layer is globally undrained, but a portion becomes looser and more susceptible to re-liquefac-

tion. Field tests were performed during this investigation at sites where liquefaction has repeatedly occurred to find evidence for the progressive densification of sands and to determine the liquefaction susceptibility of these sediments to past and future seismic loadings.

CHAPTER 3

FIELD INVESTIGATIONS AND TESTING PROGRAM

3.1 SCOPE OF FIELD INVESTIGATIONS

A three-part field study was undertaken to provide a geotechnical perspective on the liquefaction phenomena associated with Charleston area seismic events. This included collection of historical observation data, implementation of an in-situ testing program, and gathering of available soil data from outside sources. First-hand observation reports were used to establish the nature, severity, and extent of liquefaction that occurred during the 1886 seismic event. Special emphasis was placed on using the historical record to establish the outer limits of 1886 liquefaction behavior. The second stage of the field investigation involved implementing an in situ testing program aimed at evaluating soil conditions at sites which did and did not exhibit

liquefaction behavior. Because it was impossible for the research group to cover the broad study area, additional soil data were obtained from local consultants and government agencies.

3.2 HISTORICAL DATA COLLECTION

The main objective of the historical data collection effort was to create a database of observed liquefaction behavior following the 1886 quake. The primary sources of information for this database were Dutton's report of 1889 and a later summary by Peters and Herrmann (1986). Both reports contain detailed accounts of sand craters, vents, and boils, along with descriptions of ground disruptions and foundation failures indicative of liquefaction at depth. Re-establishing the locations of some features required a major effort, especially where field reconnaissances were performed in remote areas. Some of the more spectacular features were referenced to distance markers along railway lines or roadways and could be relocated with a high level of confidence.

Research was performed at the South Carolina Historical Society in Charleston in search of additional accounts of liquefaction throughout the low country area. Emphasis was placed upon establishing the outer limits of observed

liquefaction behavior. Special attention was paid to reports of minor ground disruptions reported at sites such as Georgetown, South Carolina and Savannah, Georgia. Newspaper clippings, journals, reports, and photographs which suggested liquefaction-type behavior following the earthquake were documented. A series of 1886 maps of the Charleston area were found at the Historical Society and used to locate several features described in the Dutton report. It should be noted that individuals at the Southern Railroad Company in Charleston were also helpful in locating points along the old railroad lines.

Two days were spent at the Georgetown Rice Museum examining evidence of liquefaction in that area. The best documentation of ground disruption was found in newspaper articles from the Georgetown Enquirer which reported three sites of minor ground subsidence following the event. As compared to the Dutton report, the newspaper site descriptions were poor, and additional work was required to determine the locations of the observations. The Savannah Historical Society and Public Library were also contacted to establish how much information was available on the Savannah, Georgia area. It was learned that virtually all of the newspaper clippings, photographs, and reports from the Savannah area were compiled and sent to the Historical Society in Charleston following the earthquake.

Other locations of 1886 features were extracted from historical research work done by Obermeier (1987), Amick et al. (1989), and U.S. Nuclear Regulatory Commission (1987). Collectively, the historical information from all sources was compiled and their locations mapped on a geologic map of coastal South Carolina (see Fig. 2.3). The map illustrates the abundance and types of liquefaction features associated with various geologic units as well as the outer bounds of observed liquefaction behavior.

3.3 FIELD TESTING PROGRAM

The liquefaction induced failures associated with the great earthquakes of 1964 in Niigata, Japan and Anchorage, Alaska prompted the need for an increased knowledge of the cyclic behavior of liquefiable soils. Over the past two decades the wealth of laboratory and field investigations has determined many of the factors which influence the liquefaction susceptibility of soils (Castro and Poulos, 1977; Finn et al., 1970; Seed et al., 1983; National Research Council, 1985; Ishihara, 1985; Ladd, 1977). Today it is accepted that liquefaction occurs in sandy and silty soils, if the silt content is relatively non-plastic. Many of these reports acknowledge the difficulty encountered with obtaining undisturbed samples of loose, non-cohesive soils

and the deleterious effects that stress-release, loss of in-situ sand structure and void ratio reduction can have on the results of laboratory cyclic shear tests. The difficulty in obtaining undisturbed, representative samples of loose sands has led to the general acceptance and increasing use of in-situ testing methods for the direct measurement of soil properties influencing the liquefaction potential (Peck, 1979).

The principal focus for in-situ evaluations of liquefaction susceptibility has been the Standard Penetration Test (SPT) due to its large data base compiled over years of world-wide use. More recently, the Cone Penetration Test (CPT) has gained favor due to its standardized method of operation and its continuous record which provides a picture of the variability of soil profiles. The continuous record obtained by the CPT is especially important in that thin liquefiable layers can be detected which may otherwise be missed by the SPT. The general success in using both the SPT and CPT for the evaluation of liquefaction potential seems to indicate that most of the factors which influence the liquefaction susceptibility of a soil (relative density, stress history, cementation, etc.) likewise influence the penetration resistance of the soil (Douglas, 1981; National Research Council, 1985).

The objective of the field testing program undertaken for the present study was to utilize the CPT and to a lesser extent the SPT to define subsurface conditions at sites

within and outside of areas where liquefaction was prominent. The study was primarily involved with the development of information in the ancient beach ridge deposits from the North and South Carolina border in the north to the South Carolina-Georgia border in the south. The vast majority of the historical accounts of liquefaction were confined to these areas. At sites where 1886 liquefaction was prominent, penetration tests were carried out as close as possible to liquefaction features which were either historically documented or recently discovered by workers at the U.S. Geological Survey.

The first phase of field work was performed during the summer and fall of 1987. Several days during the summer of 1987 were spent in the Charleston area doing initial field reconnaissance and test site selection. Research personnel from Virginia Tech were assisted by Mr. Stephen Obermeier of the U.S. Geological Survey during much of this time. His experience and knowledge of the geology and engineering characteristics of soils in the South Carolina lowlands was invaluable in test site selection.

Initial in-situ testing began in the fall of 1987 and primarily involved CPT testing and auger borings. Most of the work was focused in or near the epicentral area of the 1886 event. This phase of the project involved approximately 200 hours of drill rig time over a 29-day testing schedule. 35 exploratory 40 to 45 ft.-deep

boreholes were augured at ten sites and 24 CPT's were conducted at seven sites. The auger borings were conducted adjacent to the CPT holes to allow for both visual classification of soil type and obtaining grab bag samples for sieve analyses. In addition to CPT testing, six SPT's were conducted by a commercial firm. The SPT's were conducted close to the CPT holes for calibration purposes. Shelby tube samples of soft clay and Cooper Marl were taken for testing in the laboratory at Virginia tech. Fig. 3.1 shows a photograph of the mini-cone testing operations set-up for testing at the Oakland Plantation.

The second phase of field testing began in the summer of 1988 and represented an extension of the first year's work. The primary objective of this second phase of the study was to investigate sites away from the epicentral area, and to do additional testing at key sites from the first year's study. The CPT was primarily utilized in this phase of the study, although SPT's were conducted at several sites. The testing schedule for this stage of the work extended for a 30-day period. During this time, a total of 34 CPT's, six SPT's, and two auger borings were conducted at eight sites. The SPT's were used for purposes of calibration with the CPT as well as for obtaining sand samples for grain-size analyses. Shelby tube samples of soft clays and Cooper Marl were taken at all sites where they were encountered.

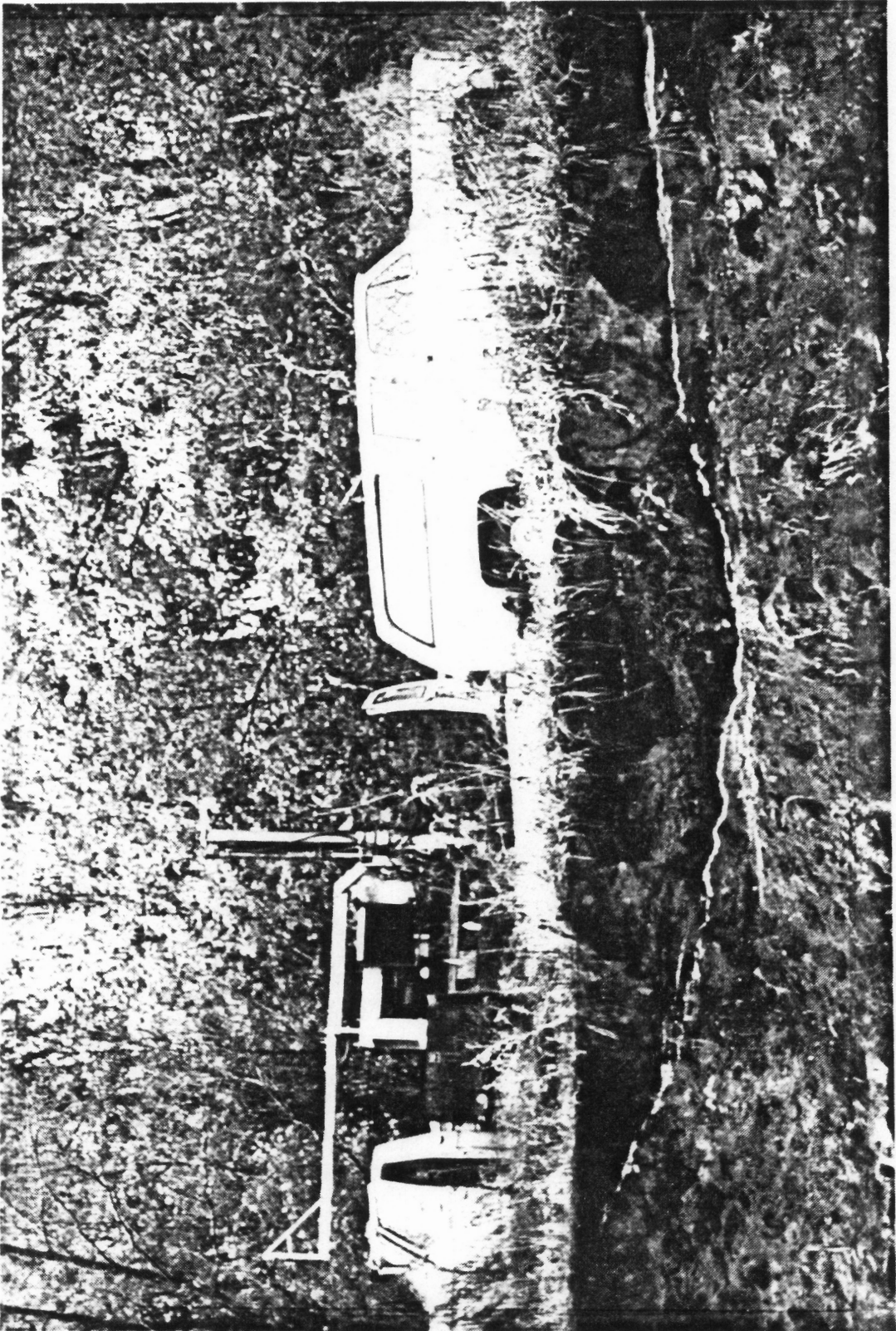


Figure 3.1 Cone Testing Operation at the Oakland Plantation Site. A Thin White Seam of Vented Sand can be Seen in the Wall of the Sand Pit.

3.4 VIRGINIA TECH TEST SITES

To characterize the liquefaction potential of soils throughout the low country region, and to examine the effects of seismic energy attenuation from the source zone, field sites were chosen on the basis of: 1) how much liquefaction occurred there in 1886; and, 2) their proximity to the zone of energy release. The on-site presence of excavations or ditches to afford exposure of the near-surface soils was also a consideration in the site selection process. The determination of the amount of liquefaction that occurred at each field site was based on assessments by U.S. Geological Survey personnel and written accounts of first-hand observers.

Table 3.1 lists the test sites investigated by Virginia Tech for this study. The distance of each site from the epicenter is indicated along with the amount of liquefaction which occurred there during the 1886 event. The sites are graded as having experienced extensive, moderate, minor, or no liquefaction. The table also lists the geologic environment of each site and summarizes the field work performed there.

The map in Fig. 2.3 shows the Virginia Tech test site locations and includes the sites of all known occurrences of liquefaction stemming from the 1886 event. The figure indicates the occurrence of liquefaction features to be

TABLE 3.1 --Virginia Tech Test Sites

Site	Distance From Energy Release (mi.)	Extent of Liquefaction 1886	Geologic Environment/ Age Classification**	Paleoseismic Investigation	1886 Observation	# of Augers	# of SPTs	# of CPTs
Sod Farm	4	Minor	Beach/Interm.	Yes	Yes	2	0	2
Eleven Mile Post	4	Moderate	Beach/Older	Yes	Yes	4	0	1
Ten Mile Hill	5	Extensive	Beach/Older	Yes	Yes	4	0	1
Hollywood	5	Moderate	Beach/Older	Yes	Yes	12	8	11
Montague	6	Minor	Beach/Older	Yes	No	1	0	1
Warren	6	Minor	Backbarrier/Older	Yes	Yes	5	0	3
St. Michael's Church	13	Minor	Fluvial/Modern	No	Yes	0	4*	0
Mt. Pleasant Pits	15	None	Beach/Interm.	Yes	No	0	1	5
Mark Clark Bridge	18	?	Beach/Interm.	No	No	0	2*	3
Oakland Plantation	19	Minor	Beach/Interm.	Yes	No	1	1	8
Edisto Island	19	Minor	Beach/Interm.	No	Yes	0	2*	8
Santee School	34	None	Beach/Interm.	No	No	1	6*	2
South Tibwin	34	None	Beach/Interm.	Yes	No	0	1	3
Georgetown	56	Minor	Beach & Fluv./Older	No	Yes	0	0	4
Pavley's Island	65	Minor	Beach/Interm.	No	Yes	0	0	4

*Performed by Commercial Firm

**Age Classification: 1) Older - ~130,000 to 230,000 yrs; 2) Intermediate - ~85,000 yrs.; 3) Modern - Present

predominant within the beach deposits and it is these areas that were focussed upon by the field testing program. The sites are more clearly illustrated in Fig. 3.2 where each site is identified by letter designations. Testing was performed within the beach deposits at locations within and outside of the meizoseismal zone. Six sites within the meizoseismal zone and eight sites that fell outside of this region were subjected to penetration testing by Virginia Tech personnel. Two other field sites were investigated for this study, although the penetration testing was performed by commercial firms. The following sections briefly describe the sites and discuss their significance. Detailed site information is reserved for Chapter 5.

Field testing within the epicentral region primarily involved the Ten Mile Hill and Hollywood areas. Moderate to extensive liquefaction behavior was reported in the vicinity of Ten Mile Hill during the 1886 event, while only minor liquefaction activity was observed near Hollywood. Ten Mile Hill lies in the heart of the meizoseismal zone, about 10 miles northwest of Charleston. Hollywood, SC is southwest of Charleston, and situated approximately 4 miles from the 1886 zone of energy release.

Three sites in the Ten Mile Hill area were investigated. The sites included the Ten Mile (TM) and Eleven Mile (EM) sign posts sites, and the Montague (MO) site. The Ten Mile and Eleven Mile sites were chosen on the basis of

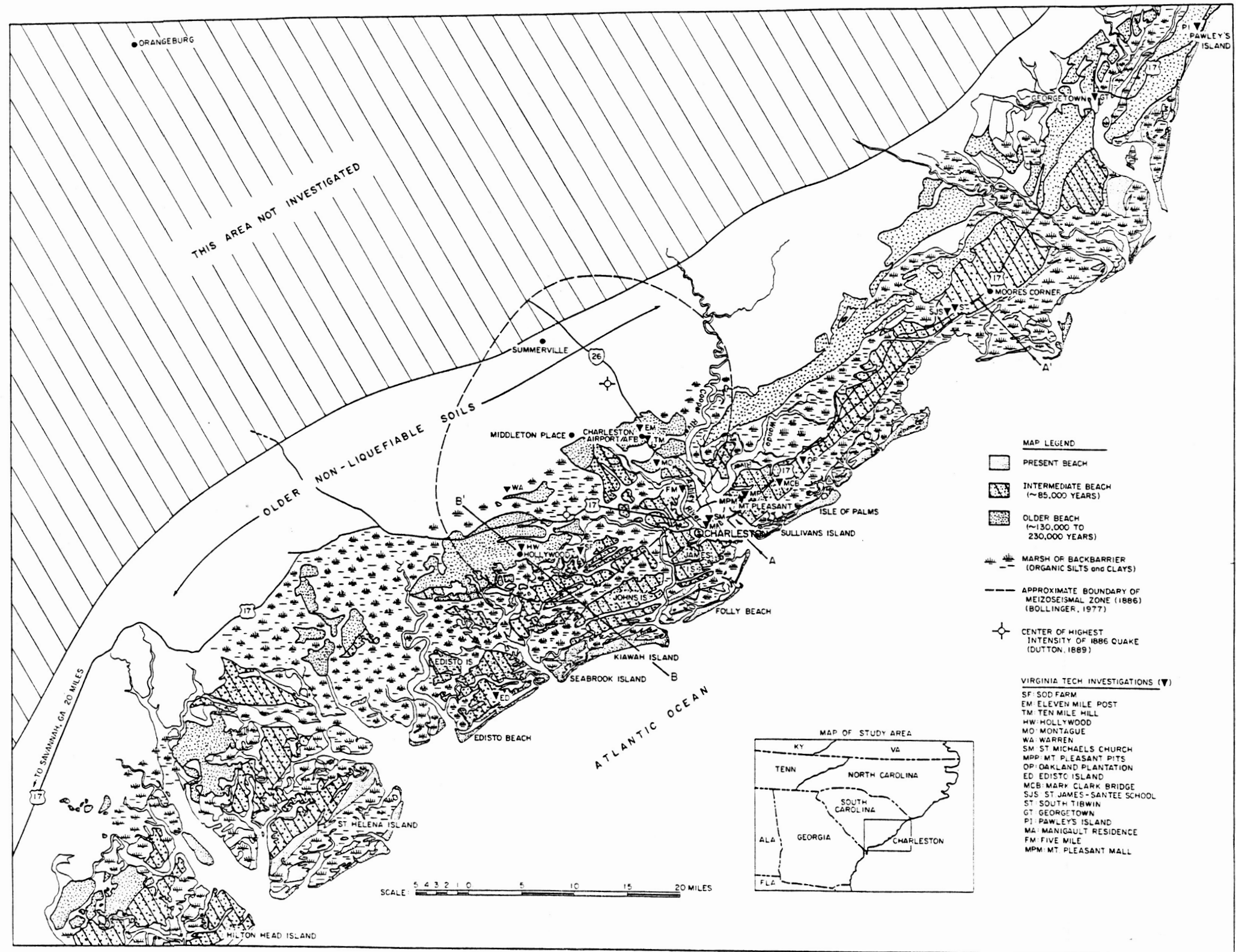


Figure 3.2 Map of Coastal South Carolina Showing Ancient Beach Deposits and Virginia Tech Test Sites.

reports by Sloan (in Dutton, 1889) and Dutton (1889) which showed photographs of craters and other liquefaction-induced ground failures following the 1886 event. Supplemental boring data for the area was obtained from the nearby Charleston International Airport/ Air Force Base. The Montague site was chosen on the basis of a small liquefaction vent that was discovered there by U.S. Geological Survey personnel, and was subjected to auger borings and CPT testing as well.

The sites near Hollywood, SC comprise the other area of field testing within the meizoseismal zone. The sites include Hollywood Ditch (HW), Warren Farm (WA), and Sod Farm (SF). All three sites are situated roughly four miles from the epicenter, and each apparently experienced minor to moderate liquefaction during the 1886 earthquake. The Hollywood Ditch site is significant because of the exposure afforded by drainage ditches that traverse the site, and because of paleoliquefaction studies performed there by U.S. Geological Survey workers (Obermeier et al., 1985; Obermeier et al., 1987; Weems et al., 1986; Gelinas, 1986; Cox, 1984; Talwani and Cox, 1985). Most of the penetration testing was conducted along the Hollywood Ditch.

Field work outside of the meizoseismal zone was focussed along the 85,000 year-old inland beach ridge. This was because liquefaction outside of the epicentral area in 1886 was primarily associated with this deposit. Seven

sites within these areas were subjected to penetration testing. The Cities of Charleston and Georgetown and the Town of Parkersville, which are all located outside of this deposit, comprise the remaining areas investigated outside of the meizoseismal zone.

The first echelon of sites outside of the epicentral region include the City of Charleston (SM) and five sites in the Mt. Pleasant, SC area -- Mt. Pleasant Pits (MPP), Mt. Pleasant Shopping Mall (MPM), Isle of Palms Connector/Mark Clark Expressway Bridge (MCB), and Oakland Plantation (OP). Both Charleston and Mt. Pleasant are about 15 miles from the epicentral area. Liquefaction-related ground failures in natural ground and man-made fills were reported at several locations in Charleston following the 1886 quake. For this study, numerous SPT borings were obtained from local consulting firms for downtown locations at or close to those areas. The Mt. Pleasant sites are significant because very minor to no liquefaction was observed there in 1886 or evidenced in recent paleoseismic studies; the area represents the outer limit (north of the epicenter) of ground motions large enough to cause liquefaction. Because SPT's for the Mt. Pleasant Shopping Center sites were available from commercial outfits, only four of the five Mt. Pleasant area sites were subjected to CPT and SPT testing by Virginia Tech personnel.

Sites further away from Charleston include the Saint James-Santee School (SJS) and South Tibwin (ST) sites which are about 35 miles northeast of the epicentral area. These two sites are situated along U.S. Highway 17 which runs along the crest of the 85,000 year-old beach ridge. The Georgetown (GT) and Parkersville/Pawley's Island (PI) sites are in an older beach deposit and lie about 60 miles from the zone of energy release. These sites represent the northern extent of Virginia Tech field work.

Edisto Island (EI) lies 20 miles southwest of the epicentral region and was an area of minor liquefaction during the 1886 event. SPT and CPT testing was performed at several locations on the island which consists of 85,000 year-old beach sediments, and recent sediments near the present shoreline. No field sites south of Edisto Island were investigated by Virginia Tech.

3.5 DATA OBTAINED FROM OTHER SOURCES

During the second year of field work in the Charleston area, supplementary soil data for this study were solicited from local consulting firms. Four firms in Charleston and one in Savannah, Georgia, were contacted to establish how much soil information was available for the study area. It was learned that soil data (mostly SPT's) were available for

much of the downtown area of Charleston and surrounding locales, including many sites that were situated within the older, inland beach ridges and along the shoreline of nearby resort islands. Because of continued recent development along U.S. Highway 17 within the 85,000 year-old beach deposit, additional data were available for sites in these areas. Numerous SPT logs from the Charleston International Airport/Air Force Base (Ten Mile Hill area) were donated by personnel at the Navy Facilities Engineering Command in Charleston. In addition to our efforts, additional soil data were collected by Civil Engineering students and faculty at The Citadel in a subcontract to this study.

More than 2000 SPT borings were collected, along with an assortment of other soil information. These data included:

- 1) SPT borings;
- 2) Grain-size analyses and soil classification tests;
- 3) Atterberg limits on various soils;
- 4) Consolidation tests on soft clays;
- 5) UU test on soft clays;
- 6) Pressuremeter test results;
- 7) In-situ shear wave velocity data on sands, clays, and marls; and,
- 8) Damping and shear modulus parameters from laboratory tests on sands, clays, and marls.

For efficient management, these data were compiled and catalogued in a computer database.

CHAPTER 4

LABORATORY AND FIELD TESTING EQUIPMENT AND PROCEDURES

4.1 LABORATORY TESTING PROGRAM

In the course of the present study, a laboratory testing program was implemented to study the Charleston area soils. Soil samples were obtained from each of the test sites and transported to the geotechnical laboratory at Virginia Tech for testing. The tests performed include:

- 1) Grain-size analyses
- 2) Specific gravity tests
- 3) Unconsolidated-Undrained tests
- 4) Consolidated-Drained tests
- 5) Grain angularity
- 6) Atterberg limits
- 7) Cyclic resistance tests

4.1.1 Grain-size Analyses

More than 60 sieve analyses and 15 hydrometer tests were performed on sands taken from Charleston area sites. Sand samples were gathered in three ways: 1) grab bag samples taken directly from liquefaction features exposed in trenches or sand mine excavations; 2) cuttings from auger borings; and, 3) split-spoon samples from Standard Penetration Tests. In all cases, logs were maintained to keep track of samples taken from zones thought to have and have not liquefied. Characteristics of liquefied sands could then be compared to those which did not liquefy.

All tests were performed according to ASTM D-422. In cases where more than 10% fines were present in the soil, the sample grain size was analyzed using a hydrometer apparatus. Gradation curves for each sample were established on semi-logarithmic plots and used to determine uniformity, grain-size, and percent fines content. More than 20 additional grain-size analyses were performed on sands taken from the Hollywood site by engineering students at The Citadel, in a supplementary study to the present one.

4.1.2 Specific Gravity Tests

Specific gravity tests were performed on both sand and clay samples taken from the Hollywood, Oakland Plantation, South Tibwin, and Georgetown sites. Details of the test procedures are outlined by Cullen (1985). The procedure

basically involved the calibration of a pycnometer device by a theoretical and experimental method. Subsequently, a relationship of pycnometer weight versus temperature was established for each method and compared. In cases where the theoretical pycnometer curve differed significantly from the experimental curve the experimental curve was used in the analyses. The subsequent specific gravity determination was performed according to Lambe (1951). It may be remembered that specific gravity is a necessary parameter for hydrometer analyses, as well as a benchmark for comparison of the Charleston sands and clays to other soils.

4.1.3 Unconsolidated Undrained Tests on Clays and Marls

Unconsolidated-undrained (UU) testing was performed on clays sampled at several of the test sites. The purpose for UU testing was to establish undrained strength values as well as to gain insight into the behavior of the Charleston clays. Correlations which relate static strength to dynamic moduli are present in the literature (Seed, 1970). The UU data also allow comparison of the Charleston clays to well-studied clays such as San Francisco Bay Mud or Mexico City Clay for which the dynamic behavior is well defined.

Five Shelby tubes of the clay and six tubes of Cooper Marl were successfully sampled and taken back to the laboratory at Virginia Tech for testing. Two of the five tubes of soft clay contained lean clays with interbedded

sand lenses and were too friable to extract and trim -- UU tests on these materials were abandoned. Additionally, two of the Shelby tubes of marl contained material which was too stiff to be extracted from the tubes. Hence, the Cooper Marl material which was successfully sampled and tested probably represents a lower bound of stiffness and strength for this material.

The remaining six tubes were extracted, trimmed, and tested in a triaxial apparatus setup with electronic data acquisition. Three samples were taken from each tube of clay and marl and tested 10, 30, and 50 psi cell pressures, respectively. A total of nine UU tests were performed on the soft clay and the Cooper Marl for a total of 18 tests.

4.1.4 Consolidated-Drained Tests on Sands

Consolidated-drained (CD) tests were performed at Virginia Tech by Cullen (1985) in a prior study of Charleston area sands. The tests were performed on reconstituted specimens consisting of sands taken from liquefaction features at the Hollywood Ditch site. Samples were reconstituted by air pluviation (see Cullen, 1985) to a relative density of 50%. Saturation was achieved by flushing the samples initially with carbon dioxide and then with distilled, de-aired water. Subsequently, a 60 psi backpressure was applied until the B-parameter reached 0.97 or higher. Tests were performed at four different effective

consolidation pressures -- 5, 10, 15, and 20 psi. Two tests were run at each consolidation pressure for a total of eight CD tests. A strain rate of 15% per hour was used. Stress-strain and volume-change data were recorded during each test. Subsequently, a strength envelope was established and used for comparison to other sands. A more complete description of the test procedures is given by Cullen (1985).

4.1.5 Grain Angularity

Sands taken from the Hollywood site were examined under a microscope at 40x and 100x for the determination of angularity of the sand grains. Angularity is important in that it has been suggested by Vaid et al. (1985) that angular sands are more resistant to liquefaction than sands with rounded or sub-rounded shapes. The shape of the Charleston sands were compared to other sands throughout the literature.

4.1.6 Atterberg Limits

Atterberg limit tests were run on the soft clay and marl material used in UU testing. Additional tests were performed on sand samples which contained a significant percentage of fines. Liquid and plastic limit tests were conducted to the ASTM D-4318 standard. The results were subsequently plotted and the plasticity index of the material established. Plasticity index is an important

parameter for comparing the Charleston clays to "unusual clays" like those in Mexico City. Also, it was important to determine whether or not the fines contained in the sands exhibited plastic behavior. .

4.1.7 Cyclic Resistance Tests

The primary objective of the cyclic testing program was to characterize the liquefaction resistance of Charleston sands relative to that of other sands. Eight undrained cyclic triaxial tests were performed on sands taken from liquefaction features found at the Hollywood Ditch site. The tests were performed on the MTS machine in the geotechnical laboratory at Virginia Tech and were done by Cullen (1985). All aspects of the testing were done in accordance with Silver et al. (1976). The tests were controlled by an MTS Controller and an MTS 436 Control Unit. The applied wave function was an invert sine wave supplied by an MTS Digital Function Generator at a frequency of 1 cycle/sec. The samples were loaded by a MTS load frame driven by a hydraulic pump. Further information concerning the testing equipment and techniques is given by Cullen (1985).

The cyclic testing program involved three initial series of tests on the "standard" sand Monterey 0/30 for which cyclic resistance curves were available in the literature (Mulilus [1977], Milstone [1985]). This phase of the cyclic testing program was used to assess the effects of

sample preparation techniques and saturation procedures, as well as operator and equipment effects. Accordingly, the Monterey sand was tested under conditions as close as possible to those established as standard by Silver et al. (1976). Subsequently, resistance curves were compared to judge the reasonableness of the test results. Good agreement was found between this work and that of other researchers, giving confidence to the testing program.

Unsuccessful attempts to retrieve undisturbed sand samples from the Hollywood Ditch site necessitated sample preparation by an air pluviation technique described by Cullen (1985). In this study, all samples were reconstituted to a relative density of 50% and tested at an effective confining pressure of 15 psi. The 50% density was chosen based on the fact that a majority of the resistance curves available in the literature were established for sands tested at or near this density. Measurements of load, axial displacement, and pore pressures were recorded during the tests using a Honeywell visicorder. Subsequently, cyclic resistance curves were established for the Hollywood sands and compared to other sands in the literature.

4.2 IN-SITU TESTING PROGRAM

In-situ tests in the Charleston area were performed in two phases over a two-year period. The initial phase of testing was undertaken during the summer of 1987 and involved a two-week field reconnaissance followed by four weeks of mini-cone penetration testing and auger borings during the fall of that year. Testing during this phase of the work was limited to the 1886 meizoseismal zone area. The second phase of field work began in the summer of 1988 and included standard-size cone penetrometer testing, standard penetration testing, Shelby tube sampling, and auger borings. During this second phase, work was performed as far south (from Charleston) as Edisto Island, SC and as far north as Pawley's Island, SC.

4.2.1 Mini-Cone Penetrometer Testing

Cone penetration testing during the summer and fall of 1987 was performed with a mini-cone penetrometer system developed at Virginia Tech by Sweeney and Clough (1986). The mini-cone is essentially a scaled-down version of the standard-size cone penetrometer. The primary impetus for its development stems from the mini-cone's reduced end bearing and friction sleeve areas which correspond to significantly less reaction force required for insertion. Investigations in areas where large rigs are not accessible

are facilitated by the reduced thrust requirement of the mini-cone system.

The mini-cone has a projected tip area of 0.65 in^2 (4.2 cm^2) with an apex angle of 60 degrees, and a friction sleeve with an area of 10.0 in^2 (64.55 cm^2). These areas compare to those of the standard-size cone at a tip and friction sleeve area of 1.6 in^2 (10 cm^2) and 23.2 in^2 (150 cm^2), respectively. The end bearing capacity of the mini-cone is 2000 lbs. This bearing force corresponds to a q_c reading of 220 bars and is sufficient for liquefaction studies involving the penetration of shallow, loose to medium sand deposits. A schematic of the mini-cone is given in Fig. 4.1.

Scaling effects involved in using the mini-cone, as opposed to the standard cone, are related to the ratio of the cone diameter to: 1) the thickness of interbedded layers; and, 2) the grain size of the tested material. The thinnest layer the cone bearing can fully identify is about 10 to 20 tip diameters (Robertson and Campanella, 1986). Thus, in interbedded deposits, the tip bearing does not reach full value for the material unless the layer thickness is 14 to 28 inches for the standard cone, but only 9 to 18 inches for the mini-cone. In thinly interbedded deposits, the mini-cone provides increased resolution and contributes to a more jagged end bearing record as compared to the standard cone which tends to average the resistance of thin

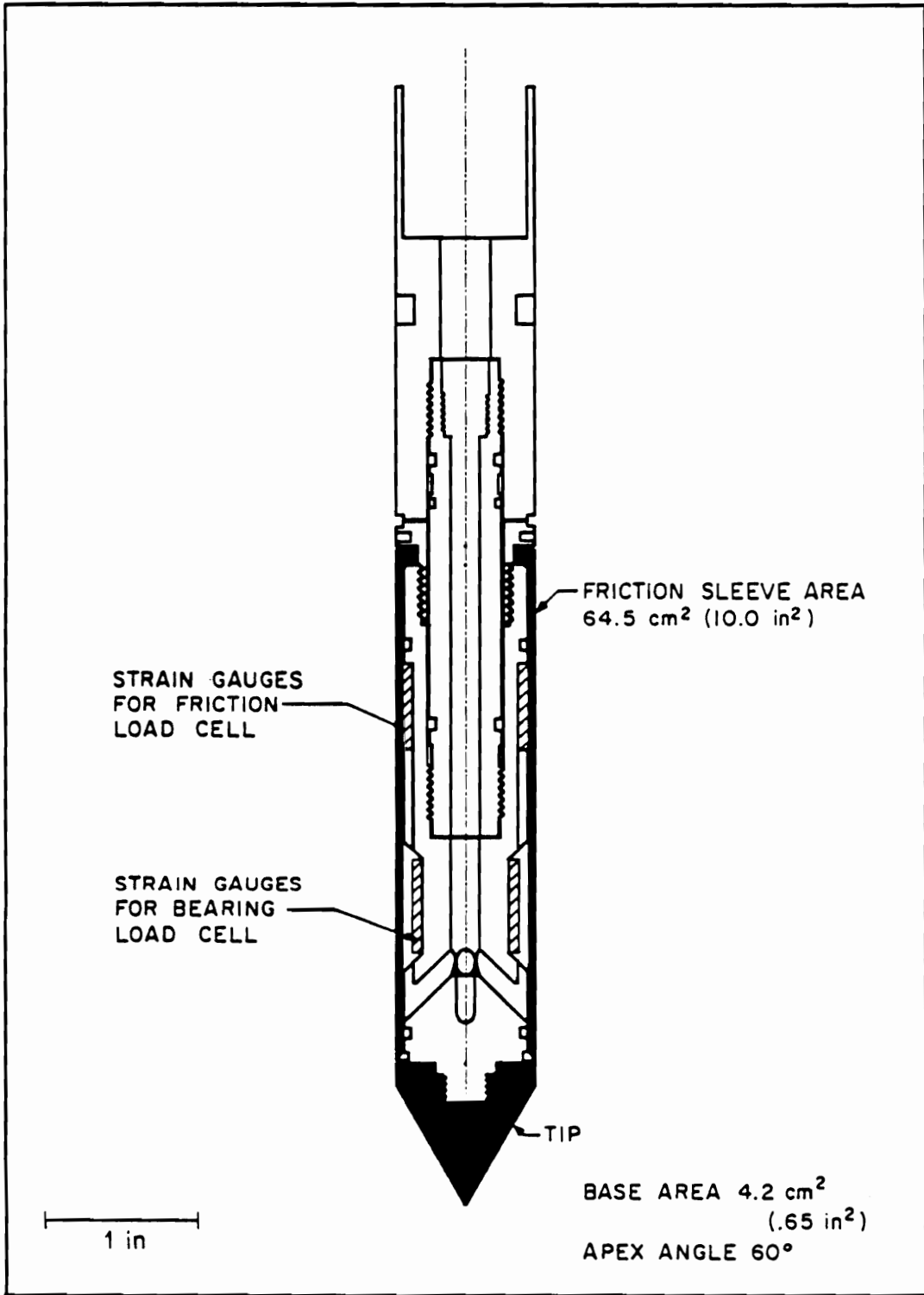


Figure 4.1 Schematic Cross-Section of the Electric Mini-Cone Penetrometer

layers. This is an advantage of the mini-cone system for liquefaction studies.

A modified Ford F350 truck equipped with an engine on the flatbed to power an Acker drilling system was used for mini-cone testing in the Charleston area. The drilling system was used to advance the cone and to perform auger borings. Fig. 4.2 illustrates the cone testing system. The rig was equipped with solid-stem augers, a donut hammer, and drill rods for performing SPT's. Utilization of the solid stem auger precluded the use of drilling mud or casing in boreholes, thereby rendering the system unusable for conducting SPT's in the loose, saturated sands encountered in the Charleston area. The rig weighs approximately 7000 lbs. and after preliminary testing, was found to be too light for pushing the mini-cone through dense material or to depths greater than about 40 ft. This was due in part to the fact that the cone was pushed by an insertion system located at the rear of the truck. As a result, the majority of the reaction force was supplied by the rear of the truck and only a percentage of the truck's full weight was utilized. A system was subsequently developed whereby the rear of the truck was anchored by cables to two nine-foot sections of screw augers inserted into the ground on either side of the drill rig prior to each test. The augers along with the gross weight of the rig provided an estimated pushing capacity of 3 tons. The hydraulic auger head used

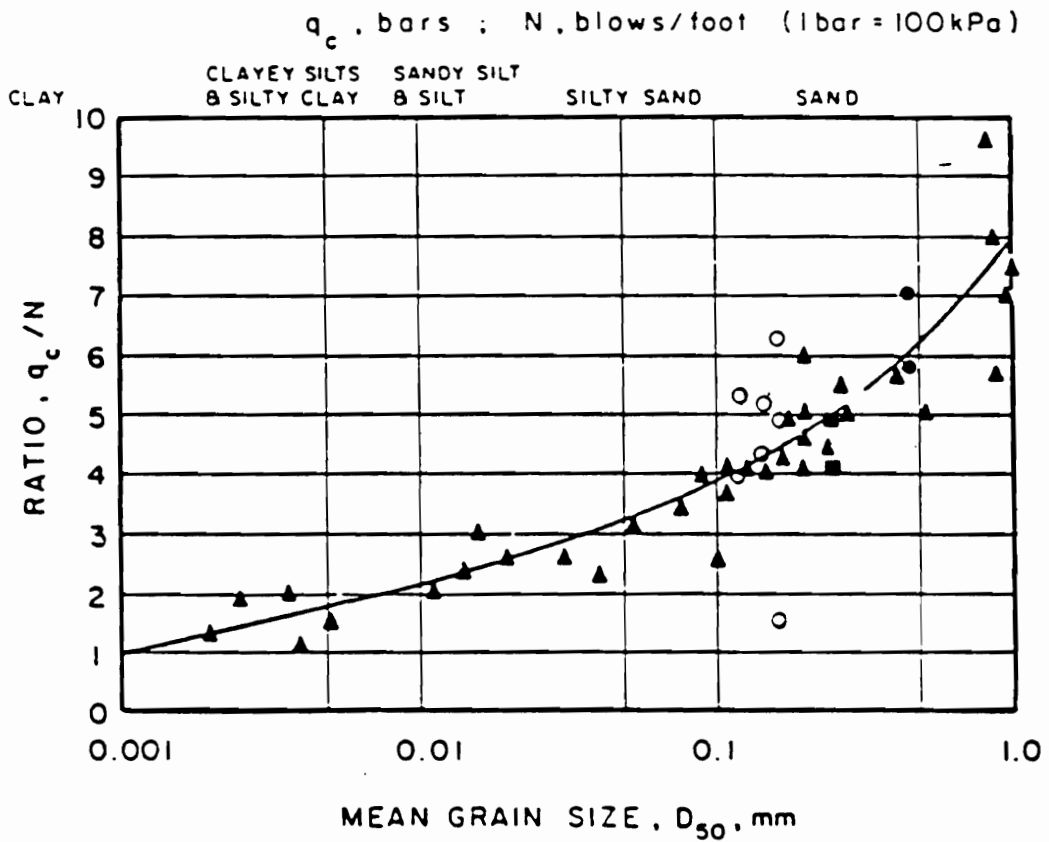


Figure 4.2 Variation of the Q_c/N Ratio With Mean Grain Size (Robertson and Campanella, 1983).

for insertion of the cone was not designed for rate-controlled raising and lowering. As a result, the hydraulic system on the rig had to be fitted with a rate control valve that restricted flow to the thrust-generating hydraulic cylinders. This modification allowed the cone to be pushed at a constant rate of 2 cm/sec as specified by the ASTM (D-3441) standard.

The cone rods used to push the cone were inexpensive schedule 100 water pipe with an outside diameter of 0.84 in. The pipes were cut to the desired length and threaded with standard-size tapered pipe threads which fit the mini-cone. The large loads transferred to the small diameter rods required that the unsupported length of the rod be held at a minimum to prevent buckling during cone advance. An optimum rod length of 2.5 ft. was found to be adequate against buckling while maintaining a reasonable production rate. Full-capacity loads were transmitted to the cone rods during several tests and resulted in no buckling or other distress of the cone rods.

Each 2.5-foot section of cone rod was joined together with thin-walled steel gas line couplings. The outside diameter of the test rod matched the outside diameter of the cone making it necessary to use thin-walled gas line couplings instead of thick water pipe couplings because the smaller protruding area required less reaction force to be pushed through the soil. An added benefit of the couplings

is that they act as friction reducers which expand the diameter of the hole upon insertion and reduce the amount of side friction between the cone rods and the surrounding soil. This allows for deeper penetration of the cone. This reduction in rod friction, however, is counterbalanced by an increase in bearing and friction force locally around each coupling. It has been suggested in the literature (Robertson and Campanella, 1986) that in uniform clean angular fine sand, an increase in cone rod area of 25% is the most efficient in reducing rod friction and results in an increased penetration force at the tip. Interestingly, the use of the gas line couplings of 1.05 in. O.D. results in a 30% increase in the cross sectional area of the cone rod.

A special thrust head device was fabricated for cone advance and retrieval. The head was designed to push and pull directly on the pipe couplings. A special sliding clamp apparatus was incorporated into the thrust head which was designed to pull directly on the pipe couplings for cone retrieval. The thrust head was slotted to prevent damage to the penetrometer wires during advance.

4.2.1.1 Data Acquisition

The electric cone penetrometer produces a continuous signal of tip bearing and friction sleeve readings that must be recorded and processed. In the mini-cone, 10 VDC excitation voltage for the strain gauges is transmitted through a

seven-wire cable threaded through the cone rods and passes across gauges wired in a wheatstone bridge configuration. The tip and sleeve gauges of the mini-cone have a full-scale out-put of 5 mV and 1 mV respectively. The tip and sleeve signals comprise the first two channels of data that were recorded, the third channel was supplied from an analog-to-digital triggering device used to signal depth increments during cone advance. The triggering device was a specially designed spring-loaded spool which was wound with several feet of fishing line. Eight evenly spaced magnets were inset along the perimeter of one side of the spool. A Hall-effect transducer was built into the spool housing and generated an electric pulse each time a magnet passed. During cone testing, the spool was attached to a stationary part of the rig's boom while the free end of the fishing wire was attached to the thrust head. As the thrust head advanced during each push, the spool unwound and caused the magnets to pass by the Hall-effect device. The magnets were strategically located and the diameter of the spool specifically designed such that each 1 cm advancement of the thrust head caused a triggering of the device. The electric pulses were sent to the computer by means of a wire which extended from the spool to the computer and subsequently processed into depth information. As the thrust head was raised at the end of each push, the fishing line rewound itself around the spool. The spool was equipped

with a switch which disabled the Hall-effect device as the thrust head was raised.

The primary data collection tool was an IBM portable PC which recorded the return signals from all three channels. The PC was powered by a portable field generator and was used to run data acquisition software which recorded and processed the incoming data. The acquisition software along with an electronic interface system were developed by Dr. T. Brandon of Virginia Tech. The PC data acquisition system was driven by pulses from the depth trigger which was set-up to trigger the computer to record tip and sleeve readings at every 1 cm of penetration. Real time plots of tip and sleeve resistance were displayed on the screen while the test was in progress. To prevent overloading the cone, the acquisition software was programmed to alert rig operators whenever the mini-cone's load limit of 2 tons was reached. The three-channel digital output was stored on floppy diskettes allowing test records to be easily prepared, copied for archival storage, and plotted using available spreadsheet editing software. An analog strip chart recorder was utilized as a backup recording system. The strip chart recorder plotted only the tip and sleeve channels. The recorder was stopped during each rod assembly and the test depth was written on the strip chart. The strip chart plot proved beneficial in assessing the validity of any peculiarities which may have occurred during testing

or in the digital data. The entire data acquisition system was mounted in the back of a support vehicle which could be enclosed for testing in unfavorable conditions.

4.2.1.2 Preliminary Field Testing

The several months prior to the first phase of field testing in the Charleston area were spent preparing the mini-cone penetrometer system and modifying a drill rig obtained from the Virginia Tech Agronomy department for this work. At the beginning of the first year of the project, three mini-cones were machined and electrically outfitted at Virginia Tech as backup to the original device. Each of the cones were tested for accuracy and repeatability, calibration and linearity, output stability, temperature effects, and water tightness. A detailed discussion of the factors affecting electric cone measurements is presented by Robertson and Campanella (1986). After establishing a method that would allow for proper and repeatable cone performance, a site was needed with soil conditions similar to those in Charleston, SC. The Pepper's Ferry site located near Virginia Tech on a bank of the New River is a fluvial terrace deposit, a portion of which is comprised of uniform silty sand overlying limestone at depth. This site was used to test and refine field procedures.

Only minor modifications were necessary to make the drill rig functional for penetration testing. As discussed earlier, the two primary modifications involved installing a

rate control for the hydraulic thrust head used to advance the cone and the development of an anchoring system to increase the reaction of the drill rig. After completion of concurrent developments on the drill rig and data acquisitions system, a series of 6 CPT's and 2 SPT's were performed at the Pepper's Ferry site to determine the repeatability and reliability of the cone under field conditions. Sieve analyses were run on the split-spoon samples so that established q_c -to-N (cone tip reading to SPT blow counts) conversion relationships could be used to compare the CPT data to equivalent tip bearing values from the SPT blow counts (see Fig. 4.2). As illustrated in Fig. 4.3, the cone provided repeatable field data which was judged to be reliable based on visual soil classifications, relations to SPT's, and the soil classification chart developed for the standard-size electric cone. The data also plotted favorably on the classification chart used to relate cone bearing, q_c , to friction ratio, $f_s/q_c \cdot 100$, (see Fig. 4.4). Before travelling to Charleston, a final check of the equipment and calibration of the cones were performed.

4.2.1.3 Testing Procedures

Prior to each mini-cone test, soil borings were carried out for visual soil descriptions and grab-bag sampling. While auguring, particular notice was given to the density of the sands at shallow depths. After the soil log had been made, two nine-foot sections of screw auger were inserted

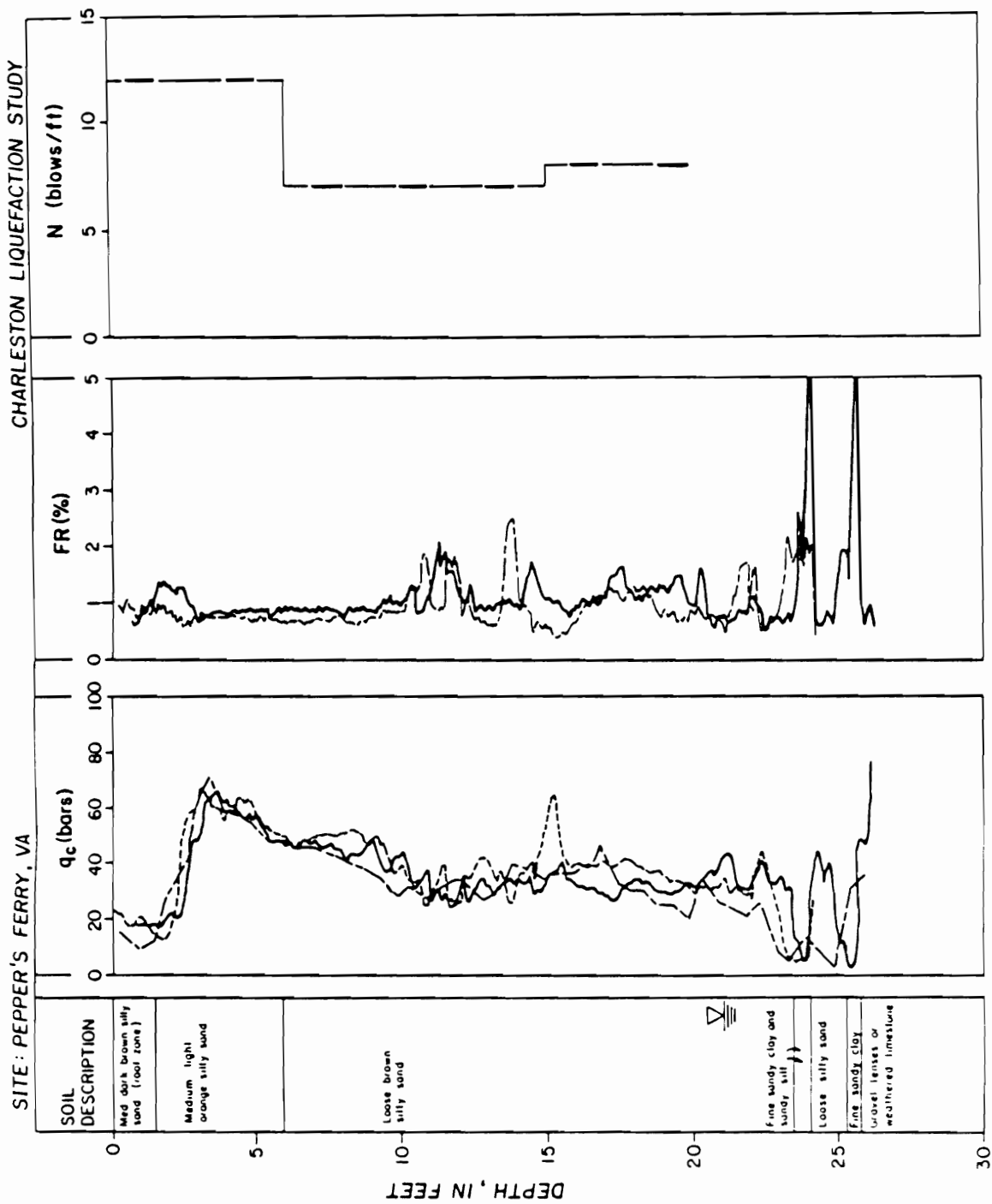
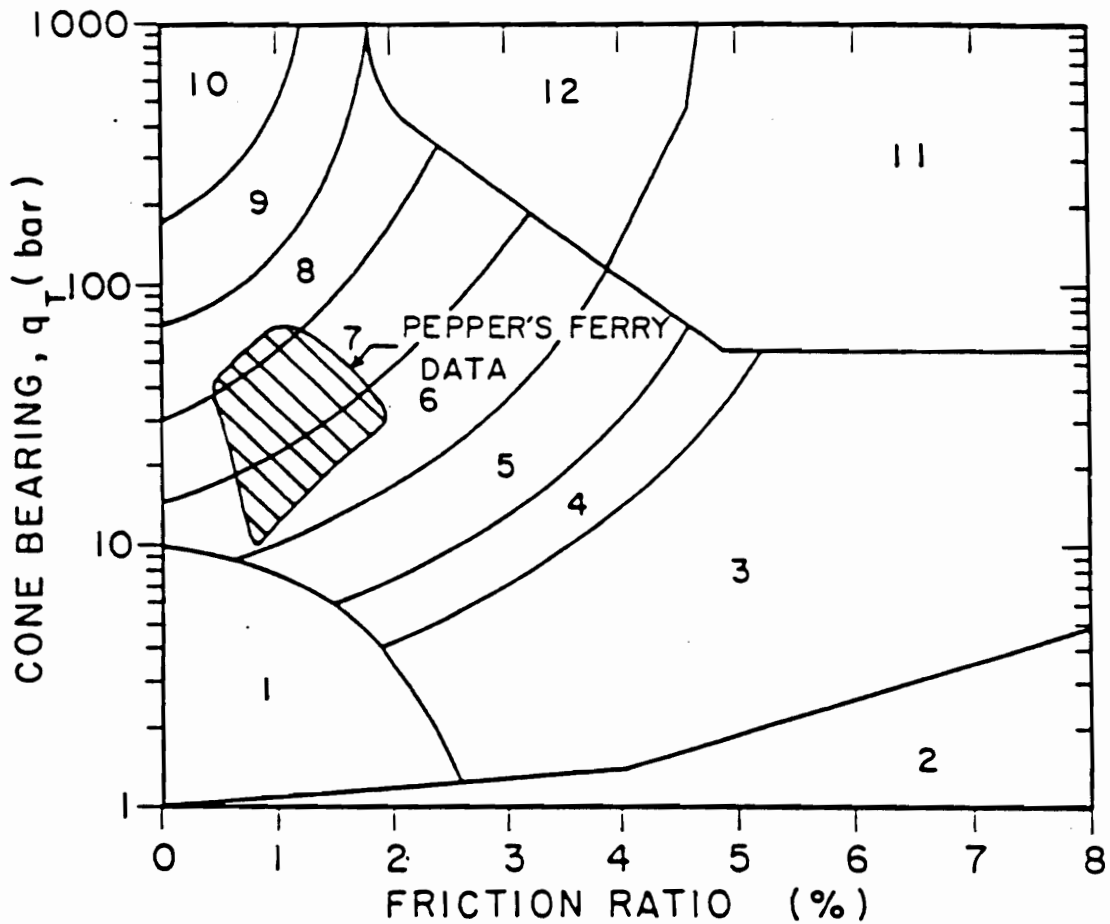


Figure 4.3 Penetration Records for the Pepper's Ferry, VA Test Site.



Zone	Qc/N	Soil Behaviour Type
1)	2	sensitive fine grained
2)	1	organic material
3)	1	clay
4)	1.5	silty clay to clay
5)	2	clayey silt to silty clay
6)	2.5	sandy silt to clayey silt
7)	3	silty sand to sandy silt
8)	4	sand to silty sand
9)	5	sand
10)	8	gravelly sand to sand
11)	1	very stiff fine grained (*)
12)	2	sand to clayey sand (*)

(*) overconsolidated or cemented

Figure 4.4 Simplified Soil Classification for Standard Electric Friction Cone (Robertson and Campanella, 1986).

into the ground on either side of the drill rig. The rear of the rig was anchored to the augers by means of a cable and a load binder. During initial testing at the Hollywood site it was discovered that when cone tests were begun from the ground surface the thrust capacity of the drill rig was typically reached at depths of 20 to 25 ft. This was presumably due to large bearing forces acting on the cone rod couplings as they penetrated dense sands layers encountered near the ground surface. After several penetration tests at the Hollywood site demonstrated that the sands in the top ten feet of the profile were dense to very dense and exhibited extremely low liquefaction potential, it was decided to start the cone tests at the base of the dense sand layer. This was accomplished by pre-augering each cone test hole to the base of the dense sands and subsequently backfilling the hole with sand to maintain lateral rod support. CPT's began at the base of the dense sand and ended when thrust capacity had been exceeded at depths ranging from 30 to 45 feet. At other sites CPT's were always begun at the ground surface and extended the full length of the cone rods (45 ft.) or refusal, unless similar difficulties were encountered.

After pre-augering, the rig operators were free to prepare the rig and data acquisition system simultaneously. The universal auguring head was removed and the cone thrust head and extension rod emplaced. The depth trigger was then

bolted to the rig boom and the retractable trigger wire attached to the thrust head. The final step involved dialing the rate control valve to the proper setting for the 2 cm/sec rate of penetration.

Temperature equilibration of the penetrometer to subsurface conditions was achieved by placing the cone under the groundwater level in a pre-augered borehole for not less than 15 minutes. With the data acquisition and drill rig ready for testing, the penetrometer was then brought back to the surface and a zero-load reading of the tip and friction sleeve was recorded. The cone was then pushed to the starting depth and the test begun. The cone was pushed in 2.5 ft. increments between which data acquisition recording was stopped, the thrust head was backed off, and the next section of cone rod was assembled and tightened with wrenches. The data recording was resumed upon cone advancement. This sequence was repeated until thrust capacity was reached or all of the cone rods were used.

At the end of the test, the rod was retrieved using the sliding collar on the thrust head which clamped around the cone rod couplings. The cone was brought to the surface and a final zero load reading was recorded and checked against the initial value. Test data was then saved and copied onto floppy diskettes and the rig was converted back to the auguring mode and moved to the next test location.

4.2.2 Standard-size Cone testing

The second phase of the project which was undertaken in the summer of 1988 involved extensive standard-size cone penetrometer testing. Development of the testing system primarily involved securing a cone penetrometer, specially modifying a 20-ton drill rig, development of a data acquisition system, and preliminary field testing to check equipment performance and optimize production. Initial coordination of the system was begun by research personnel at Virginia Tech several months prior to the commencement of testing in July of 1988.

A standard-size electric cone penetrometer manufactured by Fugro, Inc. was used for field testing. The Fugro cone had a tip area of 1.6 in.^2 (10 cm^2), a friction sleeve area of 23.2 in.^2 (150 cm^2), and was rated at 5 tons capacity. It may be remembered that cone capacity, the limiting range of tip and sleeve measurement, determines the resolution of cone measurements in the field. Thus, cone capacity must be chosen such that the highest resolution is obtained over the range of resistances encountered in the field. The 5 ton capacity of the Fugro cone used in this study corresponds to a tip resistance of 450 bars and was considered to provide good resolution in the loose to medium-dense Charleston soils.

Bonded strain gauges are located inside the cone and are used as built-in load cells that record separately end

bearing stress, q_c , and side friction stress, f_s . Both the tip and friction sleeve load cells consist of a wheatstone bridge with two active strain gages per arm of the circuit. This arrangement compensates for bending strains and records only axial strains applied to the cone. Signals were relayed from the cone back up to an electronic data acquisition system through a specially designed cone cable obtained from Hogentogler, Inc. A waterproof connector was used to prevent electrical shorting due to water entry below the water table.

A 10-wheeled, 20-ton, 1959 Failing 1250 drill rig was used for penetration testing in the Charleston, SC area. The rig had a rear-mounted drilling system which operated on a power take-off from the main engine, a 563 cu. in. International gasoline powerplant. Torque was applied to the auger head by a 25-foot 3- 3/8" diameter kelly which was powered by a rotary table located on the deck of the rig. The kelly was housed inside a 35-foot boom with three attached winch lines which were used for raising and hanging sections of pipe during SPT testing. The drill rig had an inboard air compressor which was used to power a set of air wrenches kept on board for faster equipment set-up and repairs.

The rig was originally set up for rock coring and required several modifications to be converted into a cone penetration system. Modifications primarily involved

outfitting the rig with a specially designed thrust bracket used to advance and extract the cone penetrometer. The bracket was fabricated from boiler-plate steel and was securely bolted to the kelly of the drill rig which could be hydraulically raised and lowered at a constant rate. A thrust head specifically designed by Hogentogler, Inc. to push directly on the cone rods was welded to the thrust bracket. The thrust head had exact dimensions to fit the cone rods and was slotted to prevent damage to the cone cable during cone insertion. Extraction of the cone was achieved by a rod-pulling adapter designed to fit the Hogentogler thrust head. The adapter had threads which matched the cone rods and was screwed into the back of each rod section as the rods were systematically pulled out of the ground. The insertion system including the thrust bracket with the attached thrust head could be bolted onto the kelly or removed in less than 5 minutes. This was an important consideration in that the boom could not be lowered with the thrust bracket in place making it necessary to remove the bracket each time the drill rig moved to a new location or whenever the rig was to be used for SPT's.

Standard-size cone rods of 1 meter length were used for cone advancement. During preliminary field tests, it was discovered that an unsupported length of 5 to 6 ft. could be used without buckling of the cone rods during insertion. As an added measure of prudence, a steel ring was welded to the

rear deck of the rig to brace the rods which were fed through the ring during cone advancement. The brace cut the unsupported length to about 2-1/2 ft. and no buckling was observed up to full thrust capacity. To achieve greater penetration depths, a friction reducer was used to minimize side friction along the cone rods. The friction reducer was simply a 1-inch section of over-sized pipe which was slipped over the first cone rod and welded in place. As recommended in the literature (Robertson and Campanella, 1986), the cross-sectional area of the reducer was designed to be 25% greater than the cross-sectional area of the cone rod. Enough cone rods were carried on the rig to extend the test to a depth of 50 ft., more than adequate for liquefaction analyses. A wooden cone rod rack was made to facilitate handling and stacking of the rods and speed production during testing.

4.2.2.1 Data Acquisition

Except for minor improvements, the data acquisition system used in this phase of the study was essentially the same as that used in conjunction with the mini-cone testing program. The standard-size electric cone penetrometer is identical to the mini-cone penetrometer with respect to the electrical workings of each. The standard-size electric cone produces a steady signal of tip and friction sleeve readings that must be recorded and processed into information. This was accomplished by the use of an electronic

data acquisition system which consisted of four primary components:

- 1) a portable IBM PC
- 2) an analog-to-digital (A/D) conversion box
- 3) an electronic depth trigger, and
- 4) the software CONETEST which recorded and managed the incoming data.

The system recorded three channels of information including the tip resistance, the sleeve friction resistance, and test depth.

A portable IBM PC was used as the primary data collection tool. The PC was powered by a portable field generator and ran the software CONETEST which was developed by Dr. T. Brandon of Virginia Tech to record and process the information being sent by the cone. The CONETEST software was an improved version of the data acquisition software used for mini-cone testing. CONETEST recorded and displayed the tip and sleeve resistances along with the depth on the screen in real time. The depth measurement was recorded by a special electronic trigger which sent signals back to the computer for each 1 cm of depth. The complete acquisition system was housed in the back of a companion support vehicle which provided protection from the elements. A special cabinet was designed and constructed to house all of the components of the acquisition system. The cabinet was padded which gave protection to the equipment during

transit. It also provided more organization, minimized set-up time and space, protected cable connections and minimized pull-outs, and made the system more portable. Other aspects of the data acquisition system remained essentially unchanged from those discussed earlier in the section on the mini-cone penetrometer.

4.2.2.2 Preliminary Field Testing

Several weeks prior to mobilizing to the Charleston area were spent modifying the drill rig and optimizing the cone testing system. Much less time and effort was necessary in the development of the standard-size cone system as compared to the mini-cone penetrometer system due to the fact that:

- 1) the standard cone had a history of acceptable performance and was designed for field testing
- 2) the 20-ton drill rig was heavy enough to push the cone to adequate depths without the design of an anchoring system to increase reaction force
- 3) the use of a standard size cone rod allowed for greater unsupported lengths of the rods during insertion which significantly increased the production rate
- 4) the data acquisition system was essentially the same as that developed for use with the mini-cone system
- 5) preliminary test sites were already located and well-tested with the mini-cone and SPT's -- thus, a good

data base was available for confirmation of the results obtained with the new system.

The Fugro cone was tested in the laboratory for repeatability, watertightness, and calibration. The acquisition system was checked by simulating field tests in the laboratory and using the CONETEST software to record the necessary measurements. After the rig modification was completed and the data acquisition system and cone operation verified, the equipment was taken to the Pepper's Ferry site for preliminary field testing. Five CPT's were performed at the site which is located adjacent to the New River and is composed of 20 to 25 ft. of loose to medium-dense silty sand. A second site, Kip's Farm, located closer to the Virginia Tech campus, consisted of a residual deposit of medium to stiff clays and silts and was selected for three CPT's. During this preliminary testing phase, the system was tuned for production and repeatability and the results were checked with prior CPT and SPT data obtained from the test sites. The results were found to be in excellent agreement with existing data obtained from the site. Based on the favorable performance of the overall system along with the respectable production rate of 6 to 8 50-ft. CPT's per day, the system was judged adequate for mobilization to Charleston, SC.

4.2.2.3 Testing Procedures

Prior to cone testing at new locations in the Charleston area, SPT's were performed to obtain samples for classification and grain size analyses, to identify the soil layers of primary interest, to establish the depth of the water table, and to provide a calibration of the CPT results with SPT blow counts. After the SPT was completed, the crew of two worked quickly to convert the rig into the CPT mode. All of the SPT test rod and sampling equipment was laid off to the side and the special thrust bracket was bolted to the kelly with an air-powered wrench. Simultaneously, the companion support vehicle which carried the PC computer, the portable generator, and pre-strung cone rod was backed into position a few feet behind the drill rig. The support vehicle operator prepared the data acquisition equipment by setting up and starting the generator, bolting the special trigger device to the rig's boom, starting up the PC, and connecting the cone cable to the computer data acquisition box. The cone was checked for proper zero and the data acquisition software and trigger device were checked for proper operation. For temperature equalization of the cone with the subsurface temperatures, the cone was placed in a pre-bored hole which extended below the groundwater table. This was an important consideration in that the air temperature in the Charleston area was almost always close to or greater than 100° F. The cone was kept in the borehole for

approximately 15 minutes before the test was begun. During this time the advancement rate of the thrust head was checked and adjusted if necessary for the 2 cm/sec rate. The rig was checked for level and shored up as necessary with timber lagging. The exact location of the cone test was recorded in a field book and referenced to some permanent landmark. Lastly, the results of the prior SPT were discussed between the operators and the maximum desirable depth for the CPT was determined.

After all of the pre-test checks were complete, the cone was secured in the proper position and advanced into the ground. The cone was pushed in 1-meter increments, the length of each of the standard cone rod sections. One crew member operated the drill rig while the other ran the data acquisition system. The operator running the acquisition system monitored the penetration data displayed on the computer screen and recorded any peculiarities noted during the test. Also it was his job to alert the rig operator whenever the maximum thrust capacity of the cone was approached. At the end of each push the thrust head was stopped and raised, data acquisition was put in standby mode, another section of pre-strung cone rod was added, hand tightened, and inserted into the thrust head for advancement. The data acquisition system was then put into record mode and the thrust head was advanced. This sequence was maintained until the test was complete or until the

thrust capacity of the system was reached. Enough cone rods were carried to reach a maximum test depth of 50 ft.

At the end of the test the data acquisition system was shut off and the test data were saved and copied onto floppy diskettes. The cone rods were retrieved using a rod-pulling device which adapted to the thrust head on the drill rig. The rod puller was machined to match the threads of each cone rod section. The device was screwed into the back of each rod and the thrust head was subsequently raised to extract the rods from the ground. The rods were extracted, disassembled one by one, cleaned with a damp cloth, and stored in the cone rack. Care was taken to clamp the rods such that they did not fall back into the cone hole during extraction. This process was continued until the cone was out of the ground. The cone was then cleaned, checked for o-ring damage and either put away for the day or lowered into a pre-augured hole below the water table to await the next test. Each cone test including advancement and retrieval of the cone to a depth of up to 50 ft. could be performed in approximately one hour. This corresponds to a production rate of 6 to 8 CPT's per day but does not include the time necessary for initial set-up at the beginning of each day.

4.2.3 Standard Penetration Testing

Standard Penetration Tests (SPT's) were performed at various sites throughout the Charleston area. A standard split-spoon sampler 1.4 inches in diameter was driven into the ground with a 140-lb. hammer falling 30 inches. The number of blows necessary to drive the sampler 18 inches into the ground were recorded as blowcounts. The counts were recorded in 6-inch increments and the blowcounts taken over the last two increments (last 1 ft.) were used for engineering purposes. The hammer used in the testing was a safety type hammer which was calibrated to determine how much of the theoretical energy reached the sampler. It was determined that between 50% and 70% of the energy reached the sampler. This agrees with the generally accepted value of 60% for most hammers of this type.

Due to the loose saturated sands encountered throughout the Charleston area, all of the SPT holes had to be filled with bentonite mud to prevent collapse. Bags of the mud mixture were carried along in the companion vehicle and mixed with water on-site as needed. The drill rig was equipped with a 200 gallon mud pan which was used to store the bentonite mixture. A portable gasoline-powered water pump with a 100-ft. hose was carried on board to pump water from the nearest source for mixing of the mud. Two 55-gallon drums were also mounted on the deck of the drill rig and filled with water before mobilizing to each new SPT site.

Fortunately in all cases, a nearby creek, pond, or water spigot could be located at all of the SPT sites.

Set-up of the drill rig to perform SPT's basically involved raising and securing the boom into place, leveling the drill rig, hanging several sections of drill rod along with the safety hammer on winch lines attached to the top of the boom, putting the mud pan in place and securing a source of water for mixing, and readying the sample bags and field notebook. If CPT's had been performed immediately before, the thrust bracket and depth trigger had to be removed from the boom before testing.

Continuous sampling was performed to a depth of 20 ft., after which sampling was performed at intervals of 5 ft. Although continuous sampling in the top 20 ft. of the profile was timely, the tests were facilitated by the fact that 25 ft. lengths of drill rod could be hung on the winch lines on the boom during drilling operations. This meant that in the upper 20 ft. of the profile, separate rods could be used for sampling and drilling. This eliminated the need for removing the drill bit and replacing it with the sampler each time the next depth was to be sampled. The testing procedure involved drilling to the desired sample depth, lowering the sampler into the hole, hammering the sampler into the ground, and raising it to the ground surface. Once the split-spoon sampler reached the surface it was unscrewed from the drill rod which, with the hammer still attached to

the top, remained hanging on a winch line. The sampler was broken apart, the soil inside classified, logged, and bagged, and the sampler cleaned and screwed back onto the hanging drill rod for the next sample. During this time the rig operator drilled the hole to the new test depth, raised the rod section with the drill bit attachment, lowered the hanging rod section with the sampler, and the sampler was again driven into the ground. This sequence was continued until the test was complete. The majority of the tests were carried down to the stiff underlying marl layer which was typically found at depths ranging from 30 to 70 ft. The water table elevation was recorded at each site no sooner than 24 hrs. after the SPT had been completed. A typical 50-foot SPT with continuous sampling to 20 ft. took about 5 hours to be completed by the two-man crew.

4.2.4 Shelby Tube Sampling

Shelby tube samples of soft clay and stiff Cooper Marl were taken from all sites at which they were encountered. At most sites, good sample recovery of both materials were achieved. In a few locations however, the Cooper Marl was too hard to be sampled and resulted in collapsed Shelby tubes. All told, five sample tubes of the soft clay and six tubes of Cooper Marl were successfully sampled.

The sampling procedure involved determining the appropriate test depth, drilling to the desired depth, pushing

the tube into the ground, waiting for a few minutes, slowly twisting the tube for 1/2 turn, and retrieving the tube. Once the tube reached the ground surface, it was immediately sealed with hot paraffin wax, capped with a plastic top, wiped clean, and labelled. The tubes were then transported back to the laboratory and stored in a 100%-humidity room until they were readied for testing.

CHAPTER 5

SOIL CONDITIONS

Coverage of soil conditions affecting the liquefaction potential of sandy sediments throughout the Charleston area focuses upon the near-surface soils of the beach deposits. Soil information is presented for sites that fell within and outside of the 1886 meizoseismal zone. Those sites which fell within the meizoseismal zone are presented in the first section of the chapter, and those located outside of this region are included in following section. A discussion and summary of both sections are given at the end of the chapter.

Soil conditions are characterized on the basis of soil data from Virginia Tech work as well as data acquired from outside agencies. The Virginia Tech soil data are used to determine soil conditions at specific sites where the extent

of liquefaction that occurred during the 1886 event is clearly defined. The soil data obtained from other sources are used in a more general manner to characterize soil conditions over broad areas of the study region, and many of these areas are not clearly defined in terms of 1886 liquefaction activity. These data do however provide insight into the variation of soil conditions within and among the different geologic environments, and in a general sense, indicate how much liquefaction and ground disruption should occur throughout the study region for different levels of seismic shaking.

Where available, the following parameters that relate to conventional liquefaction potential evaluation are presented for each site: relative density, grain-size distribution, grain angularity, soil fabric, strain history, age of the deposit, and cementation. The influence of these parameters upon the cyclic behavior of sandy soils is well-demonstrated in the literature (Castro and Poulos, 1977; Finn et al., 1970; Seed et al. 1983; National Research Council, 1985; Ishihara, 1985; Ladd, 1977). Chapter 6 of this report will consider dynamic site response and soil amplification effects which can also influence the cyclic behavior and liquefaction resistance of soil deposits.

The relative density of sands can be estimated through correlations that relate penetration resistance (blowcounts,

N; or cone tip resistance, Q_c) to overburden pressure and relative density. For this study, relationships by Gibbs et al. (1957), and Lunne and Christoffersen (1985) are used for this purpose. All SPT blow counts and cone tip resistances have been converted to values normalized to an effective over-burden pressure of 1 TSF. Where appropriate, the blow counts have also been converted to values corresponding to 60% of the theoretical free-fall hammer energy.

5.1 SITES WITHIN THE 1886 MEIZOSEISMAL ZONE

Six sites that fell within the 1886 meizoseismal were investigated by Virginia Tech personnel. All of the sites had either clear relict liquefaction features, or were known from historical references to have exhibited surficial liquefaction. In exploring these sites it was recognized that there was the possibility that liquefaction induced during the 1886 event disturbed the soils, and changed them from the conditions that existed prior to the quake. This possibility is considered in the analysis of each site. In addition to the Virginia Tech data, soil data from sites at 23 different locations within the meizoseismal zone were collected from four consulting firms and two government

agencies. These data are used in areas not covered by the research group.

VIRGINIA TECH SITE INVESTIGATIONS

5.1.1 Hollywood Ditch Site

The Hollywood Ditch site is located close to the town of Hollywood, SC and is situated within the 1886 meizo-seismal zone, approximately four miles from Dutton's southwestern "center of highest intensity". This location is shown on the map in Fig. 3.2. The studied area consists of two 12-foot-wide by 8-foot-deep drainage ditches situated perpendicular to each other; one ditch is oriented east-west and extends some 9000 ft. along the flank of a Pleistocene beach deposit, while the other runs transversely across the beach crest in a north-south fashion, and is 1000 ft. long. The site lies in an older beach environment with soils dating to about 200,000 years old (McCartan et al. 1984), although a refined study of the Hollywood sediments by Weems et al. (1986) estimates the soils to be 120,000 to 130,000 years old.

The ditches at Hollywood offer one of the most extensive exposures of relic liquefaction features that have ever been observed, and it is this exposure which has prompted

paleo-liquefaction studies by the U.S. Geological Survey. Their investigations focussed along the east-west trench which offers more exposed area and runs parallel to and flanks the old beach ridge where sand blow formation is more likely. Findings at the site by Obermeier (1985, 1986, 1987) revealed some 162 relict liquefaction features representing five separate episodes of soil liquefaction. Interestingly, only 24 of these features have been attributed to the 1886 quake, and these were mostly minor expressions forming thin cracks in the overburden, while the formation of sand craters as large as the 8-foot diameter crater depicted in Fig. 2.7 were common in prehistoric quakes. Further, significant lengths of the ditch were free of any 1886 expressions which would appear to indicate that the degree of liquefaction at Hollywood during the 1886 quake was moderate to minor. This is surprising considering the proximity of the Hollywood site to the zone of energy release (4 mi.). The lack of significant ground disruption also suggests that soil conditions were essentially unchanged by the 1886 seismic event.

Hollywood Ditch was the first and most extensively tested field site of the study. A total of 15 auger borings, and 13 CPT's were performed by Virginia Tech, and eight SPT's were done by a commercial firm. The penetration tests were performed along the east-west ditch at the

locations shown in Fig. 5.1. Fig. 5.2(a) shows a generalized soil profile developed from the test holes. The site is covered by a two-foot organic layer underlain by a thick stratum of clean fine sand which extends to a depth of about 15 ft. Soil profiling work performed perpendicular to the east-west trench (stations N0560' and S0725', Fig. 5.1) indicates that this sand stratum extends roughly uniformly in both directions (see Fig. 5.2(b)). Below 15 feet, the sands quickly increase in fines content and plasticity and occur as interbedded layers of sandy clays and clayey sands. The entire site is underlain by stiff Cooper Marl which was found at depths ranging from 45 to 60 ft. The water table was encountered at a depth of about 6 feet during penetration testing, although it would be expected that prior to ditch excavation, it was within a few feet of the ground surface.

The upper 15-foot sand stratum represents the only soil at the Hollywood site that is potentially subject to liquefaction. A recent study of sediment mineralogy at the ditch concluded that the source sands for the liquefaction features observed there should lie between 5 and 15 ft (Gelinas, 1986). Fig. 5.3 shows the gradation curves for more than 40 samples taken from this layer during SPT testing and auguring. The sand is shown to be uniform with a uniformity coefficient of 1.24 to 2.10, and fine, with a

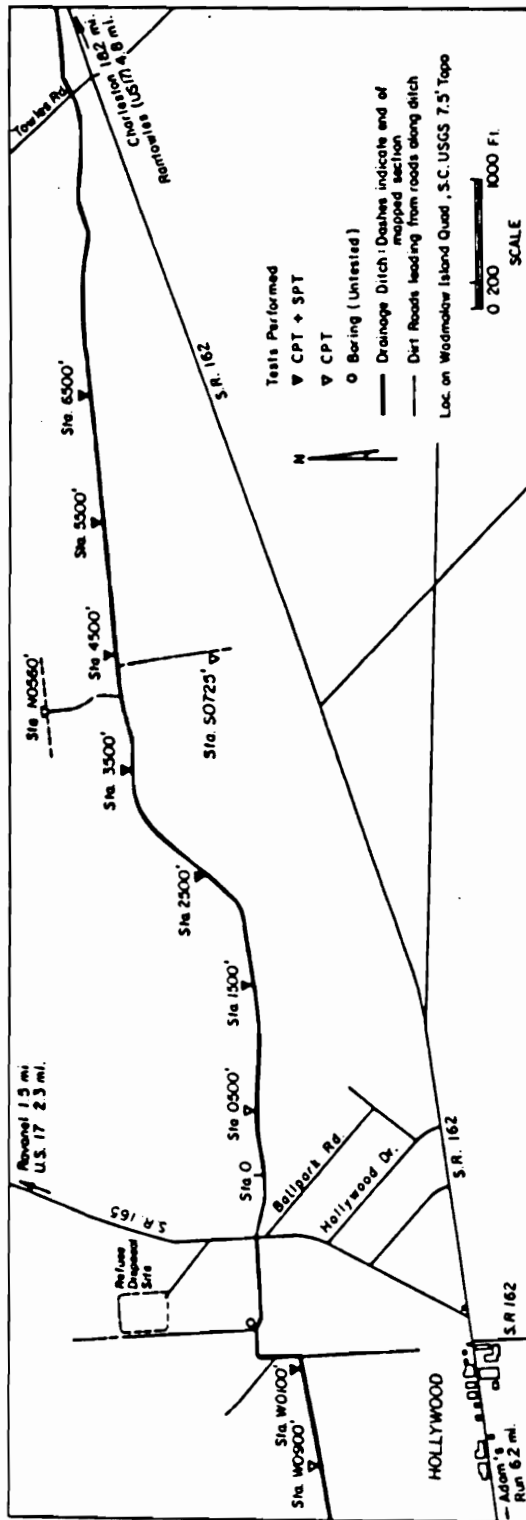


Figure 5.1 Location of Penetration Tests Performed at the Hollywood Site.

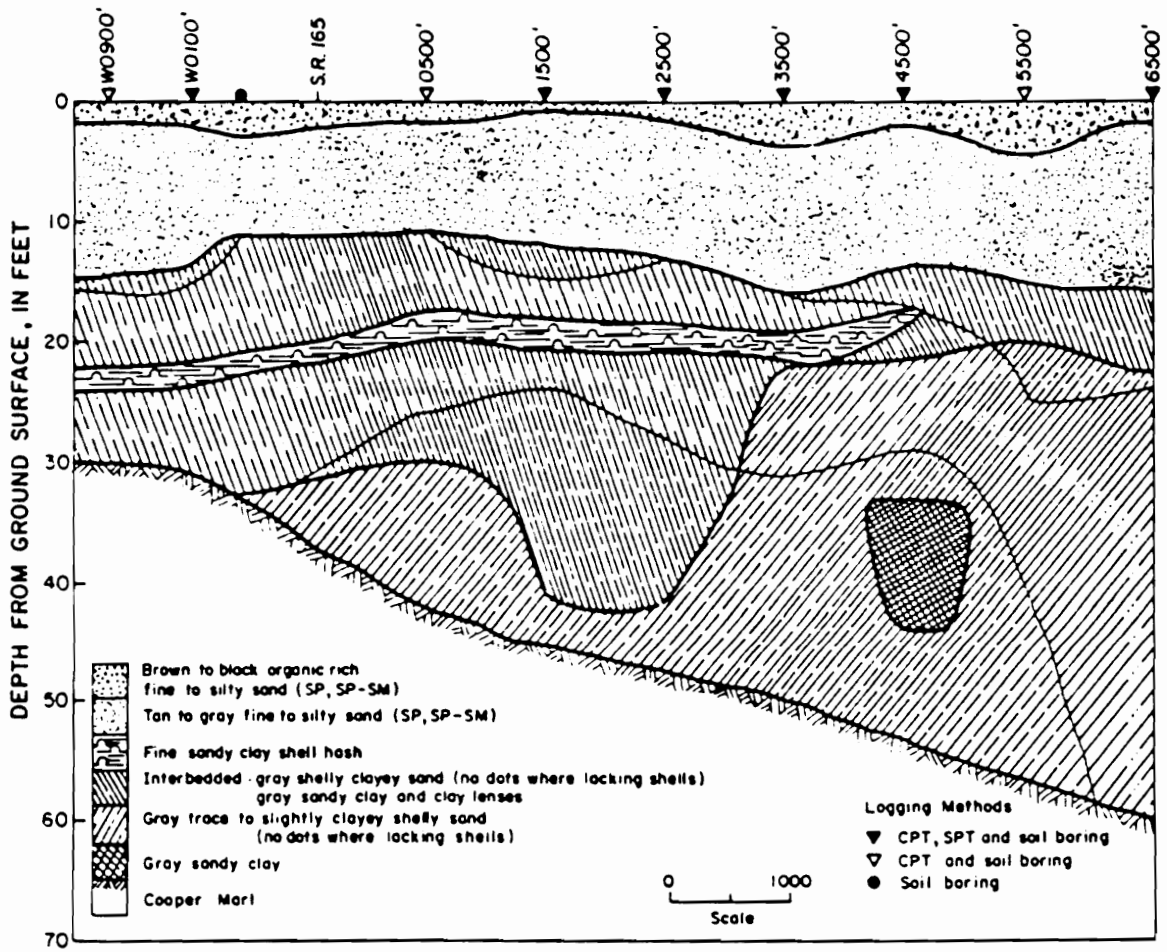
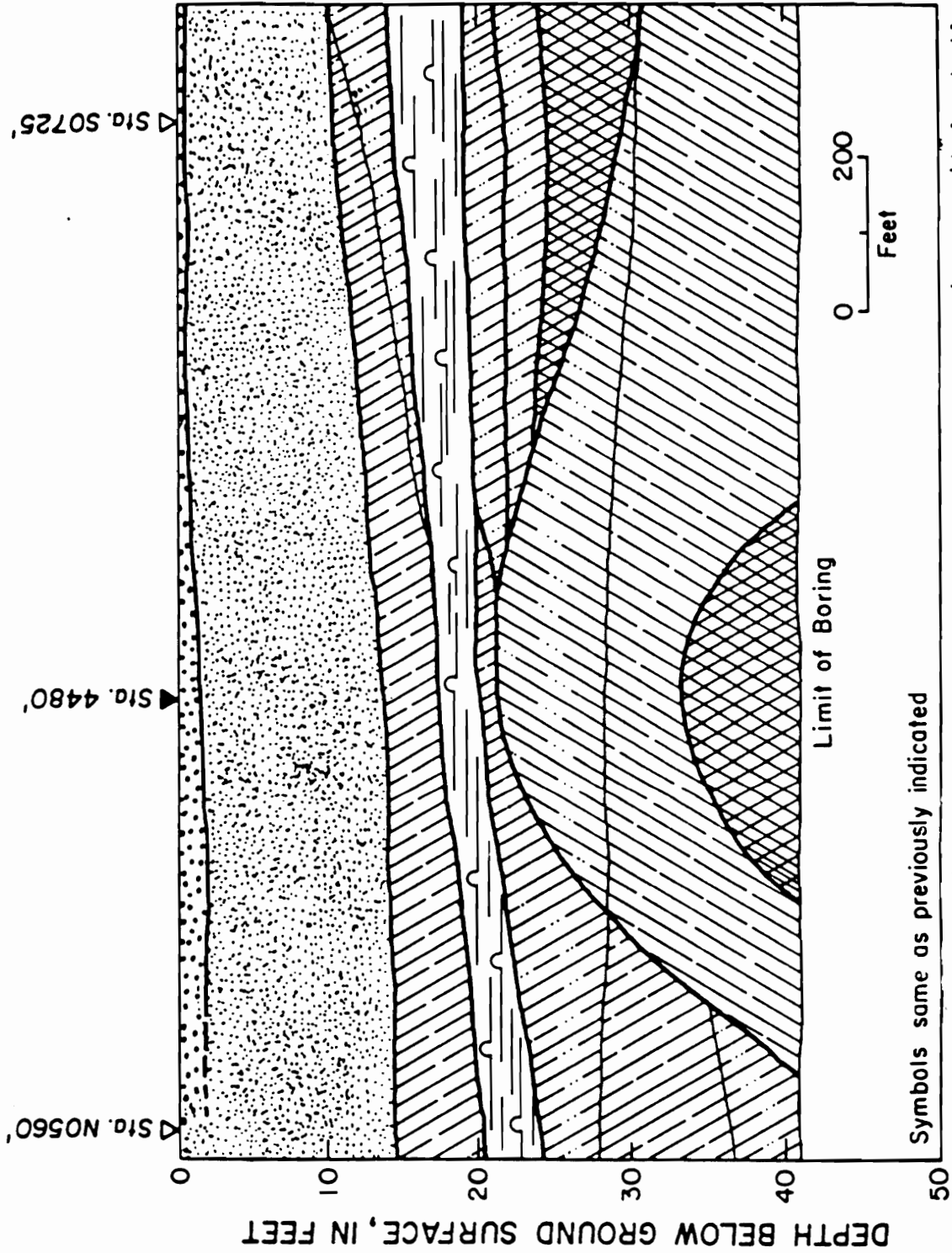


Figure 5.2(a) Generalized Soil Profile Along the Drainage Ditch at the Hollywood Site.



Symbols same as previously indicated

Figure 5.2(b) Generalized Soil Profile Transverse to the Drainage Ditch at the Hollywood Site.

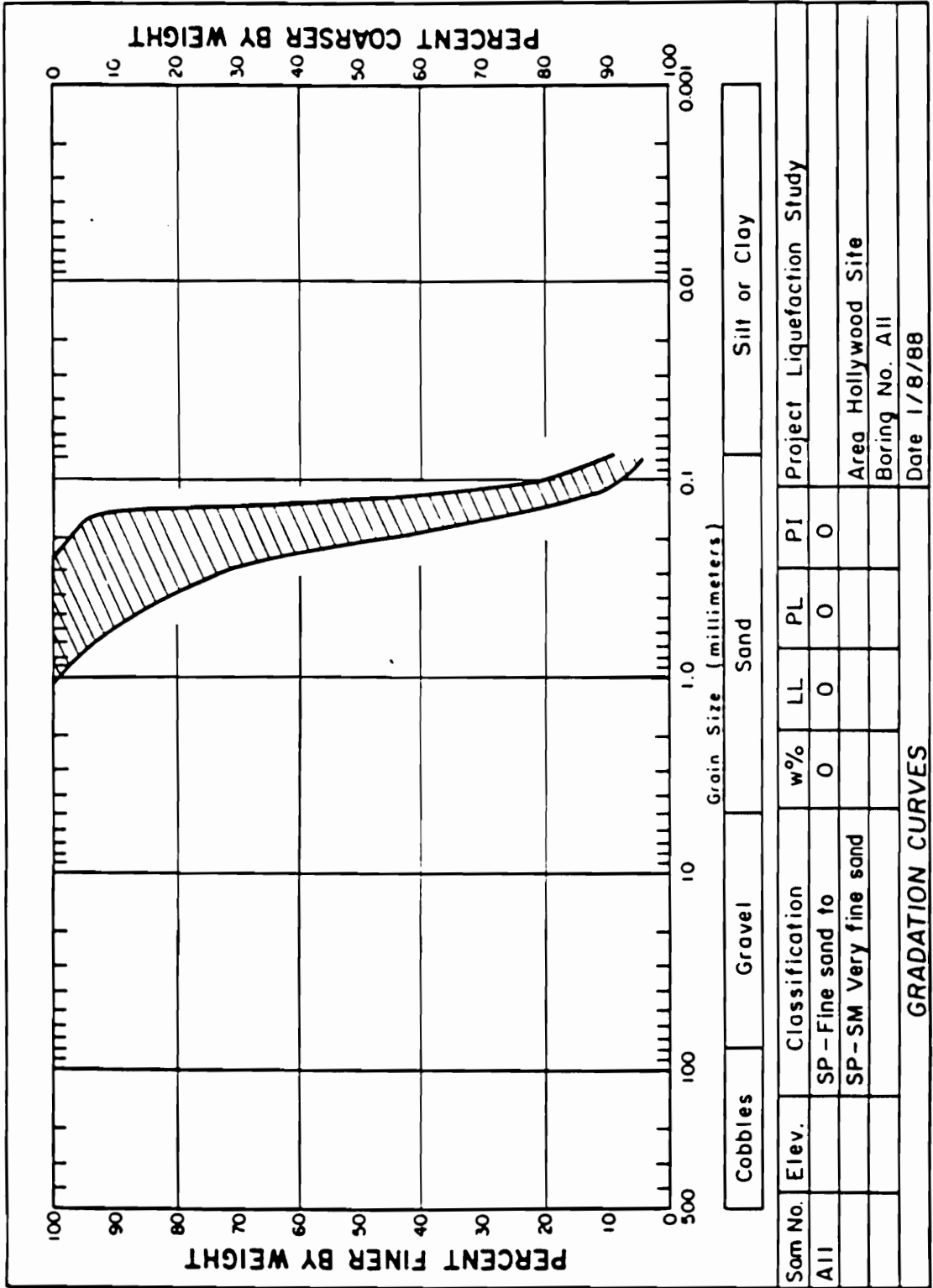


Figure 5.3 Range of Gradation for Hollywood Samples Taken From Depths Between 5 and 16 Feet.

median grain diameter of 0.12 to 0.21 mm. The fines content of the sand ranged from 2 to 9% with an average of less than 5%. All fines were predominantly non-plastic silts. A study of grain angularity (Cullen, 1985) found the sands to be angular to sub-angular. Specific gravity tests indicated a value of 2.67 for the sand. With the exception of the top organic layer, no cementation of soil particles was observed in any sand samples.

Fig.5.4 illustrates SPT blowcounts from the upper 15-foot sand layer. The blowcounts are shown to be consistent along the ditch indicating the sand density to be medium to a depth of about 5 ft., medium to very dense from 5 to 8 ft., medium from 8 to 12 ft, and very loose to medium from about 12 ft. to the bottom of the stratum. Blowcounts in the loose material near the bottom of the stratum range from 2 to 20 blows/ft., and it is this layer that is most suspect for 1886 liquefaction activity (Dickenson et al, 1988; Obermeier, 1989).

In Fig. 5.5, results are shown for six CPT's performed adjacent to the SPT holes. The figure shows resistance patterns consistent with those obtained with the SPT's. The tip resistances in the bottom loose layer range from 10 to 100 bars and average about 50 bars. A histogram of Q_c values from this zone is presented in Fig. 5.6 where a grouping of values is observed between 20 and 60 bars.

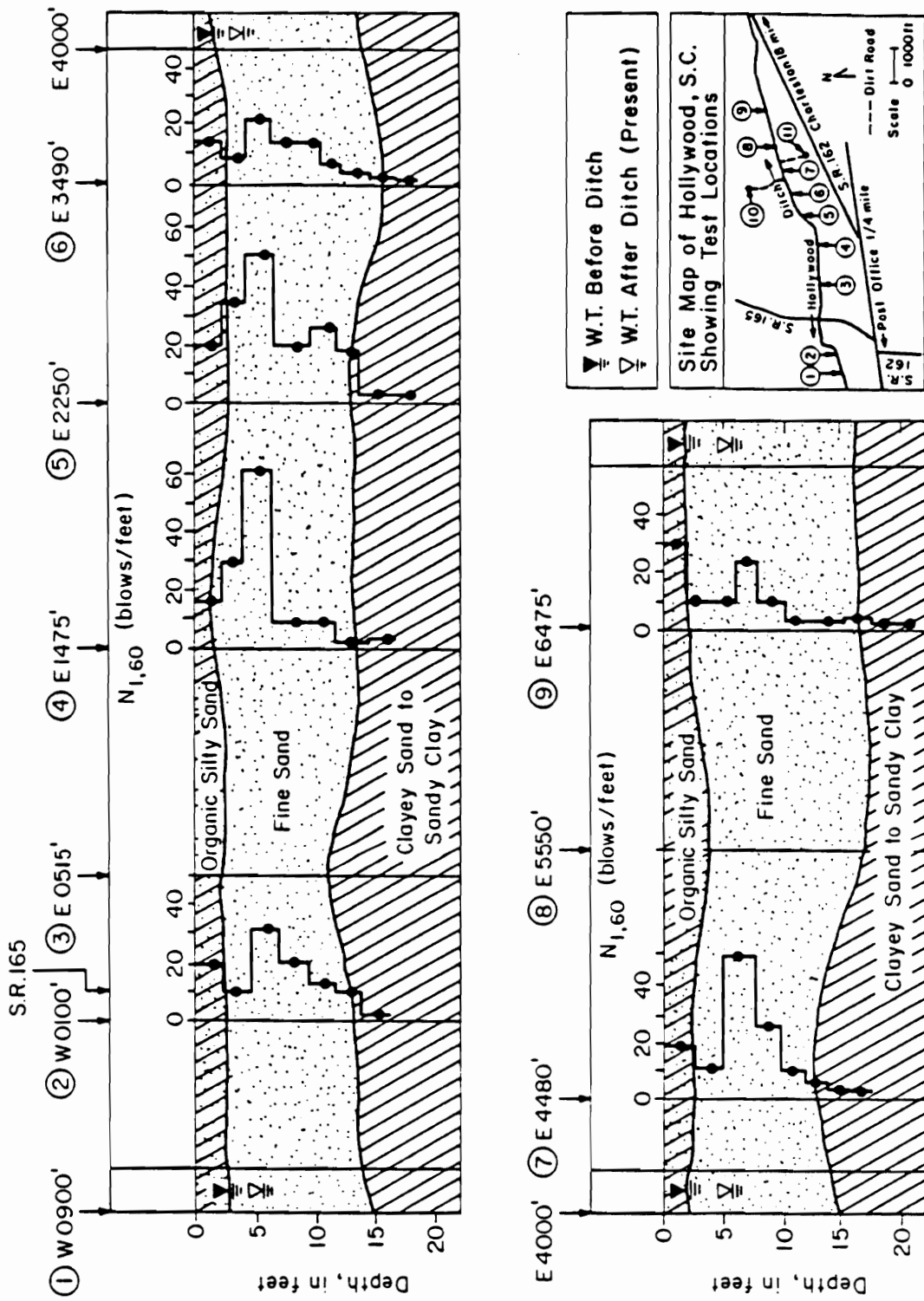


Figure 5.4 Soil Profile Showing SPT Data From Hollywood Site.

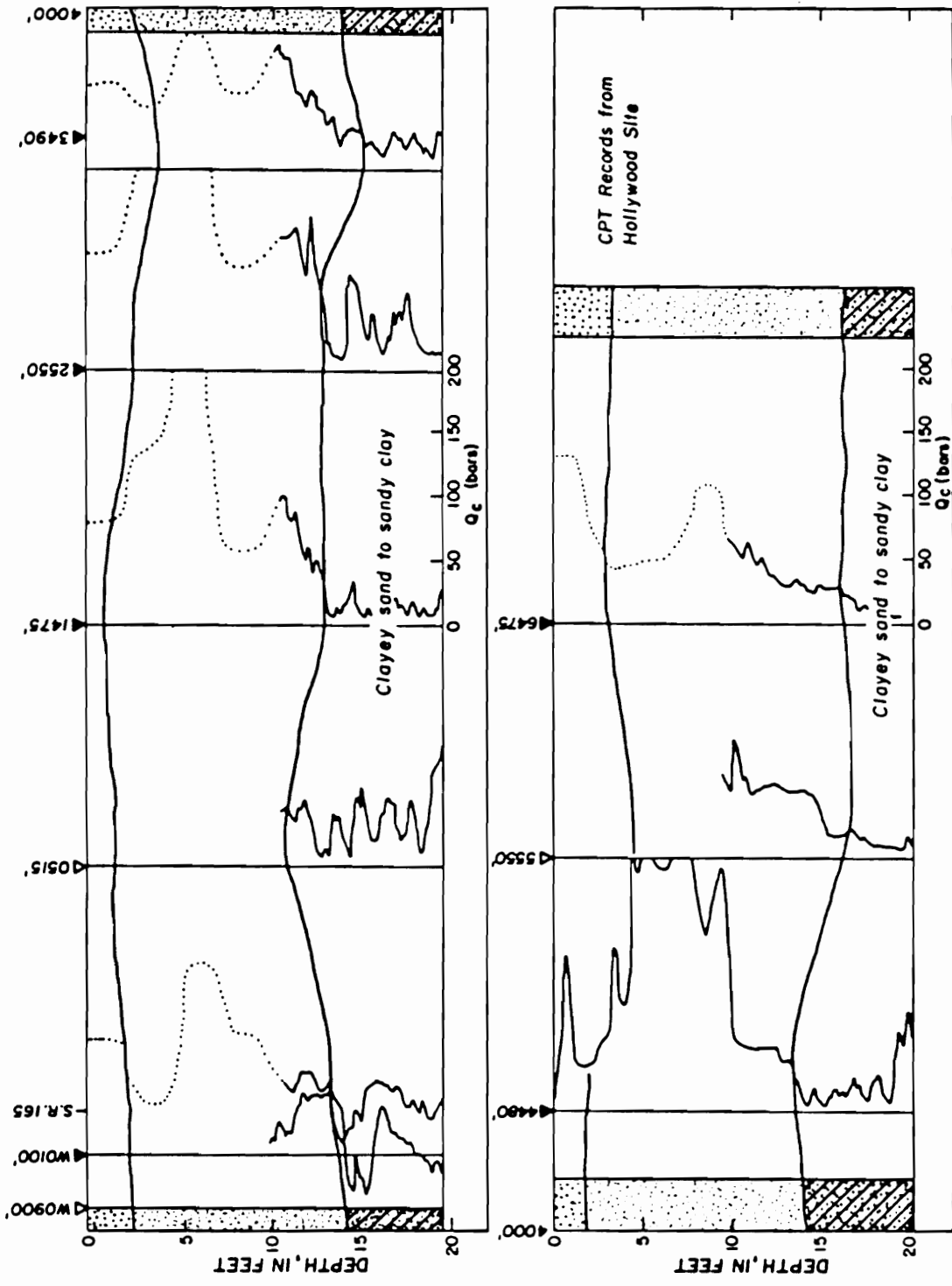


Figure 5.5 CPT Records for Stations Along the Hollywood Drainage Ditch.

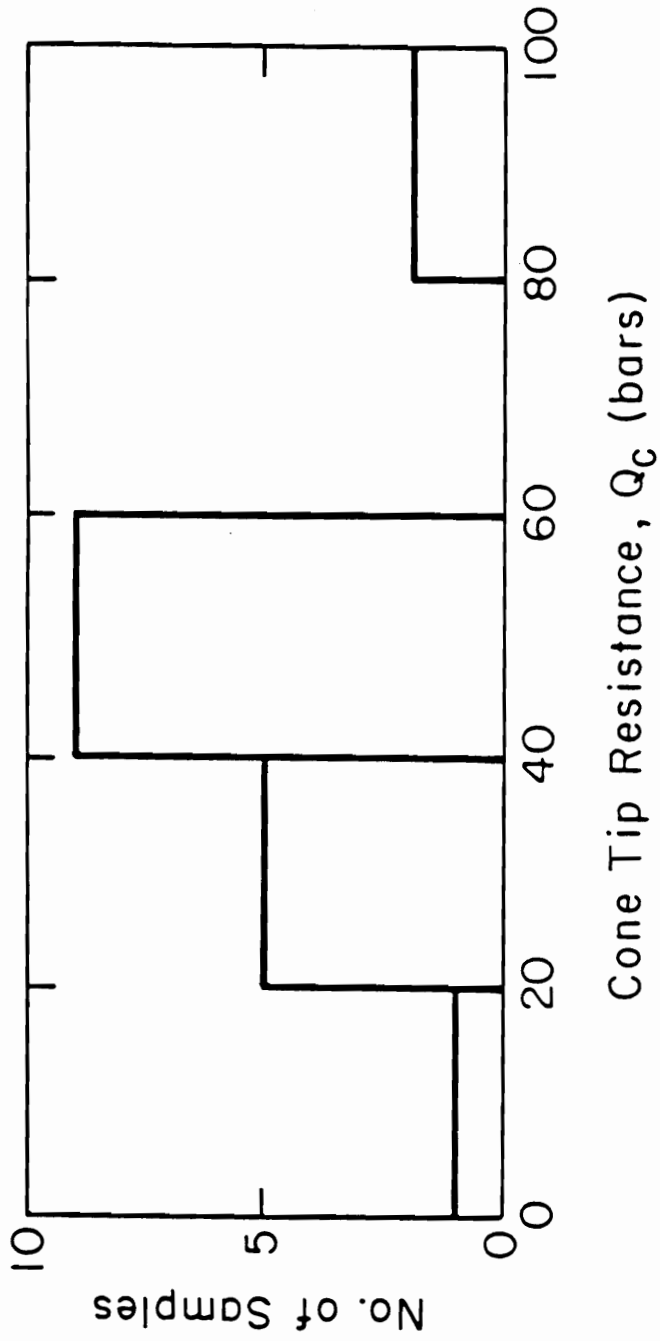


Figure 5.6 Histogram of Q_c Values for Lower Sand Layer at the Hollywood Ditch Site.

Using a relationship developed by Lunne and Christoffersen (1985), an average relative density of 50% is estimated for this material, with some values as low as 20%. The individual SPT and CPT records obtained for this site are contained in Appendix A.

Consolidated-drained (CD) static tests and undrained cyclic tests were performed on samples of reconstituted sand obtained from the bottom of the east-west trench (depth of about 8 ft.). The samples were reconstituted by air pluviation to a relative density of approximately 50%. The CD triaxial tests showed a friction angle of 38° , a value consistent with the angular nature of the sand. Stress-strain curves for the CD test series are given in Appendix C. Cyclic resistance curves for undrained cyclic tests performed by Cullen (1985) are shown in Fig. 5.7. Resistance curves for two other sands are included in the figure for comparison. It can be seen that the Hollywood sands behave similarly, but trend toward lower resistance values.

Shelby tube samples of the underlying soft clay and Cooper Marl were recovered at the site for unconsolidated-undrained (UU) testing in the laboratory. The soft clay materials were easily sampled with full recovery, but it was later discovered that the tube samples contained sand lenses and shells and were too friable to be trimmed and tested. In terms of sampling the Cooper Marl, the hardness of the

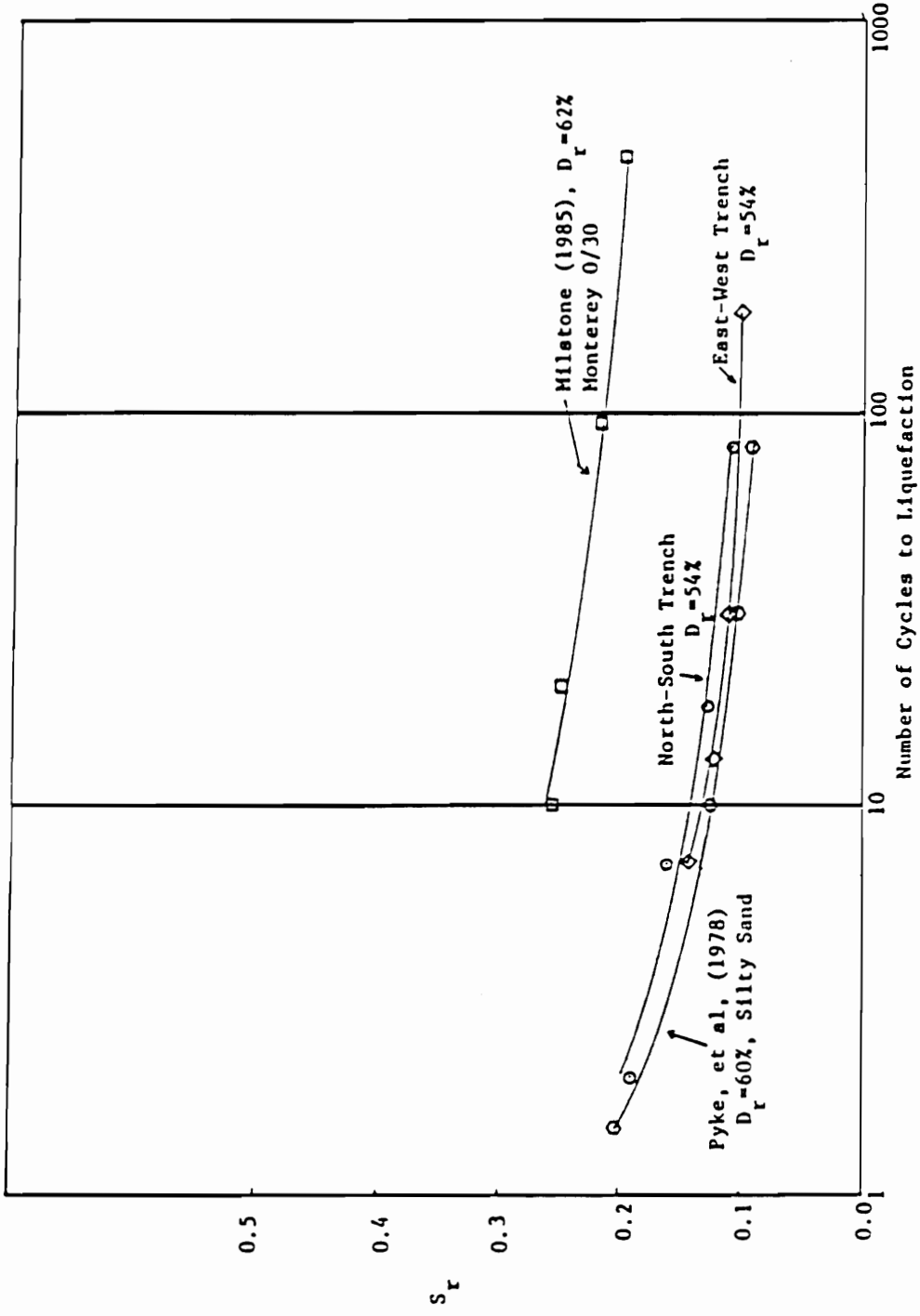


Figure 5.7 Stress Ratio Causing Liquefaction Versus Number of Cycles to Liquefaction for Charleston Sands, a Standard sand, and a Silty Sand (Cullen, 1985).

material made it difficult to sample. Pushing the sample tubes in this tough material led to crumpling of several tubes. Also, the marl material that was eventually sampled and brought back to the laboratory was too stiff to be extracted from the tubes without severe disturbance. Because of these difficulties, UU tests on neither the clay nor Cooper Marl from the Hollywood site could be performed.

5.1.2 Warren Farm Site

The Warren Farm site is located about six miles from the 1886 zone of energy release (Fig. 3.2) and is situated in the older, 200,000 year-old beach environment. The site is significant because of liquefaction that occurred there in 1886 and because of the paleoliquefaction studies that have been done there. The studied area consisted of an abandoned 30-acre pasture in which it was reported by the grandparents of the current property owners, that a large quantity of sand was expelled from a fissure which formed there during the 1886 event. The landowners pointed out the general location of this feature to Obermeier who identified an anomalous sandy area with relief, and after initial study, prompted a further investigation by Cox (1984) who trenched the site and unearthed a 30-foot long liquefaction vent. Cox investigated the morphology and stratigraphy of the feature and inferred possible source beds for the ejec-

ted material. No other liquefaction features or evidence of significant ground disruption were located at the site, indicating that liquefaction there was minor, and that soil conditions over most of the site are probably representative of 1886 conditions.

The Virginia Tech field tests were focussed around the vent feature, but were performed at locations close to and away from the liquefied area. This was done in an attempt to determine whether or not significant changes in the density patterns of the soils close to the feature had occurred as a result of the quake. The layout of the test borings is given in Fig. 5.8. Auger borings and CPT's conducted over a 600-ft. portion of the Warren site reveal a relatively uniform soil profile. As shown in Fig. 5.9, an organic soil layer covers the top 2 ft. of the profile and it is underlain by a clean fine sand stratum ranging from 2 to 8 ft. in thickness. Underlying the sand stratum is a layer of clayey sand with an average thickness of about 7 ft. A second layer of clean sand is encountered below the clay and extends for about 3 ft. before quickly grading into a 35 ft. sandy clay layer. This sandy clay layer extends to a depth of 50 ft. where Cooper Marl is encountered. The water table was located at a depth of approximately 4 ft. at the time of testing.

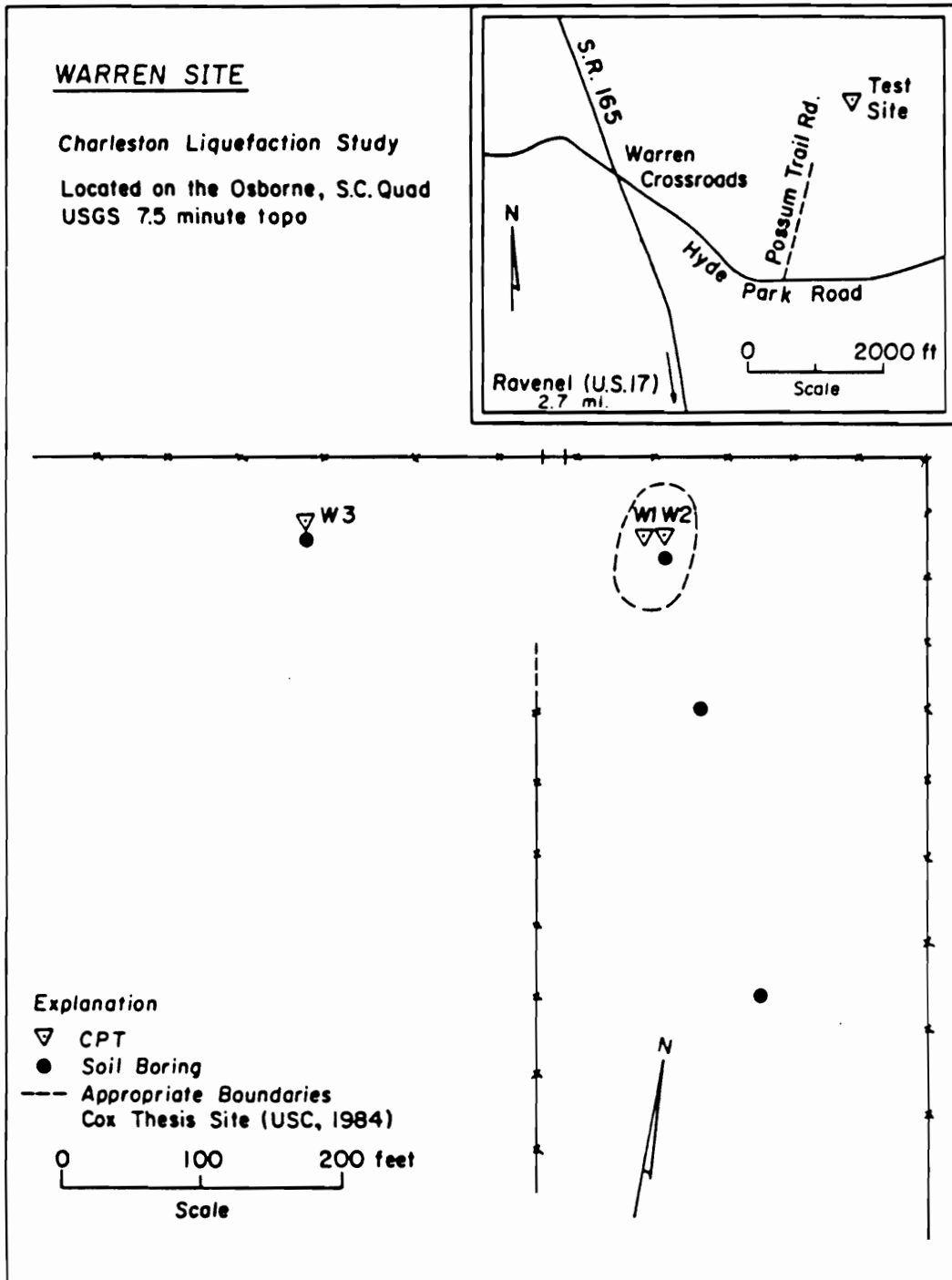


Figure 5.8 Location Map for the Warren Site.

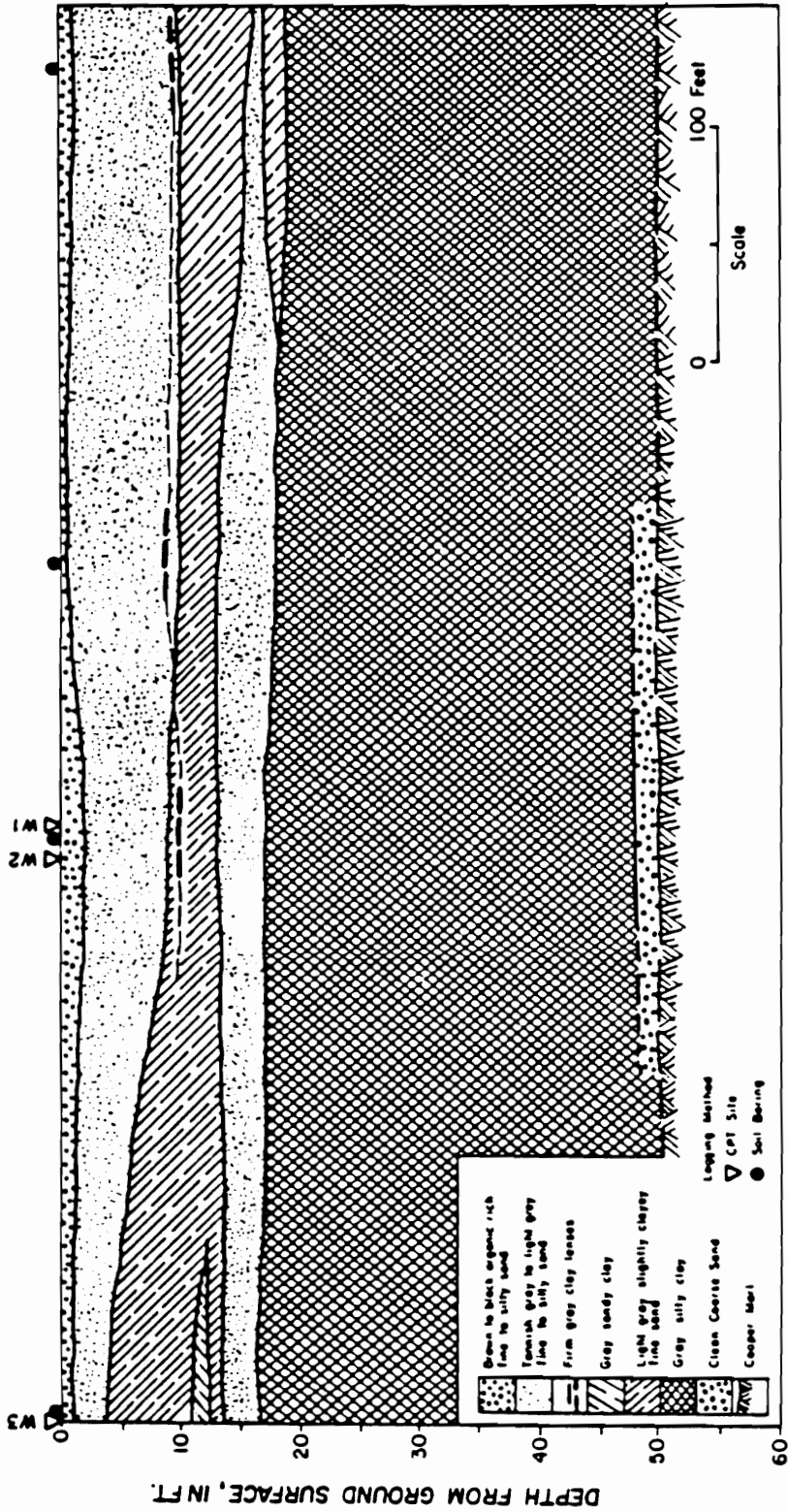


Figure 5.9 Generalized Soil Profile for the Warren Site.

CPT testing was primarily concerned with establishing the densities of the two clean sand layers, as these were the layers most susceptible to liquefaction. CPT results for three tests are illustrated in Fig. 5.10 and show the upper portion of the soil profile to be similar to that found at the Hollywood site; dense to very dense sands (85-95% Dr) in the top 5 ft. with tip resistances close to or greater than 200 bars. Below 5 ft. a gradual loosening of the sand occurs, and from 7 to 9 ft., the sand becomes very loose with densities ranging from 20-50% and Qc values as low as 30 bars. A zone of dense sand at from 9 to 10 ft. separates the loose material from the underlying clayey sand at 10 to 13 ft. Tip resistances throughout the second layer of clean sand at 13 to 16 ft. average 200 bars and correspond to an estimated relative density of 90%. Interestingly, independent investigations of the site by Cox (1984) and by Gelinis (1986) have suggested that the ejected material found in the liquefaction vent came from a depth of 14 to 17 ft. Assuming that this finding is correct, the CPT results suggest that a densification of the lower sand stratum has occurred.

The results of 35 sieve tests performed on sand taken from the vent feature and upper 5 ft. of the soil profile, are referenced from Cox (1984). A range of gradation curves is given in Fig. 5.11 which show the sand to be very fine,

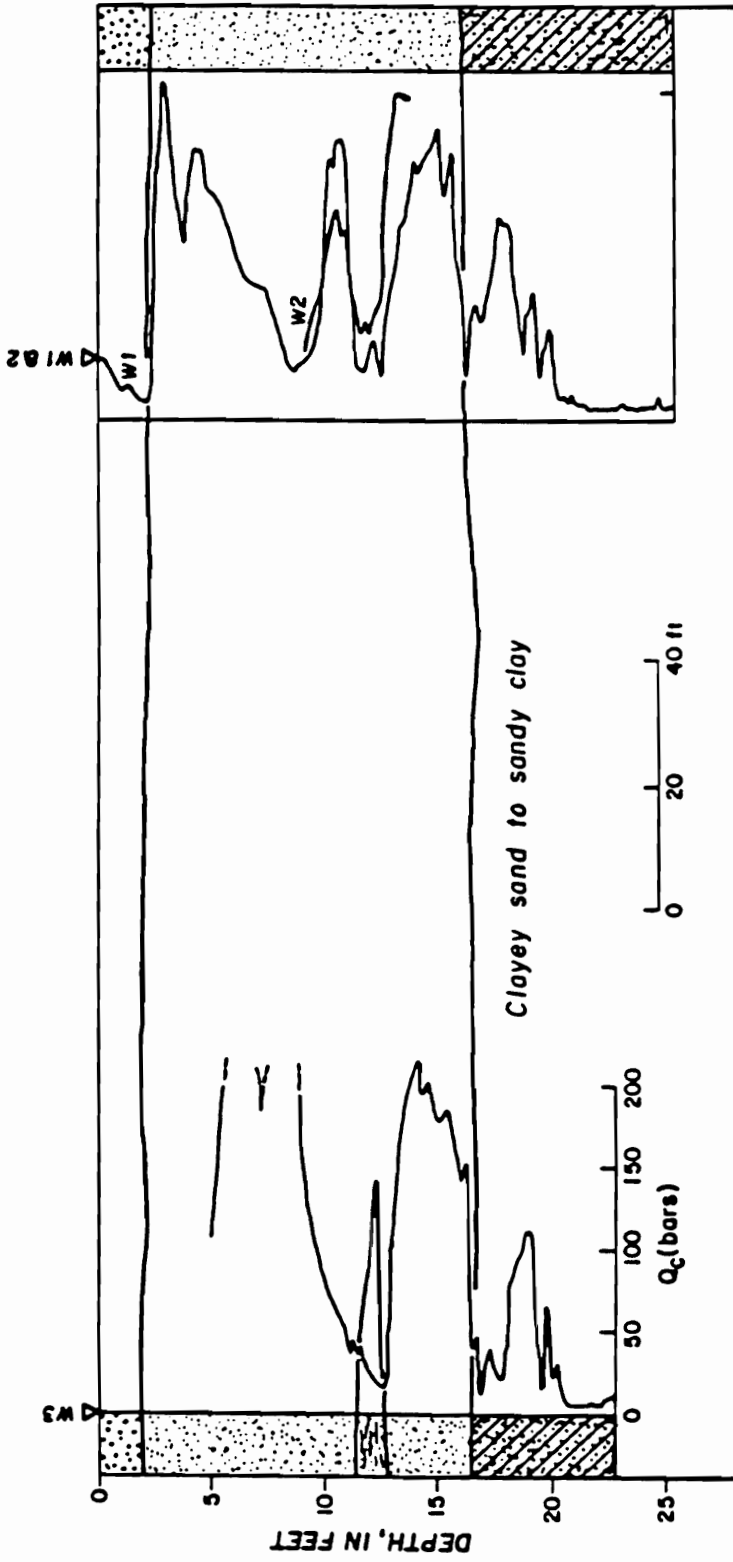


Figure 5.10 CPT Records for Stations at the Warren Site.

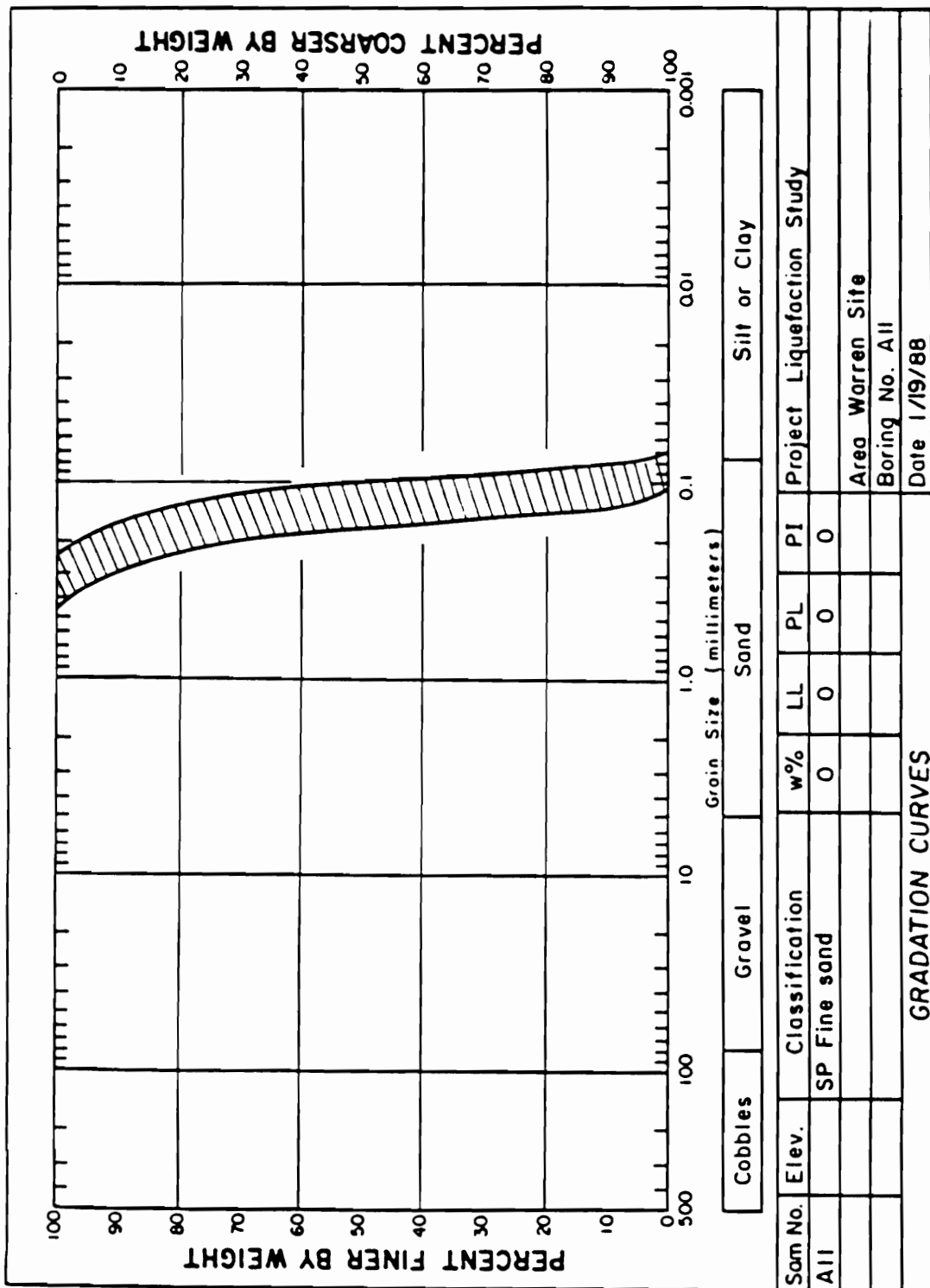


Figure 5.11 Range of Gradation for Warren Samples.

with a median grain diameter, D_{50} , of 0.10 to 0.17 mm; a uniformity coefficient, C_u , of 1.35; and, a fines content of less than 5%. All fines were found to be non-plastic silts. The sand grains in most of the sieve samples were described as sub-rounded to sub-angular. No cementation of soil particles was detected during soil logging, and tests conducted by Cox using dilute hydrochloric acid found the samples free of carbonate cement. Other soil data and individual CPT logs for this site are contained in Appendix A.

5.1.3 Sod Farm Site

The Sod Farm site lies about 4 miles from the 1886 epicentral area and is situated in the 85,000 yr.-old beach deposit. The site map for this location is given in Fig. 5.12. This site was chosen for testing because of a small liquefaction vent that was discovered there in a drainage ditch by Obermeier. Penetration testing consisted of two CPT's and one auger boring. One cone test was performed adjacent to the liquefaction feature while a second test was carried out along the ditch, approximately 200 ft. away from the feature (see Fig. 5.12). The lack of evidence for significant ground disruption at this site suggests that minor liquefaction occurred there during the 1886 quake.

During auguring, it was noted that the near-surface soils seemed to be at least as dense as those that were

SOD FARM SITE

Located on Wadmalaw Island, S.C. Quad
USGS 7.5 minute topo.

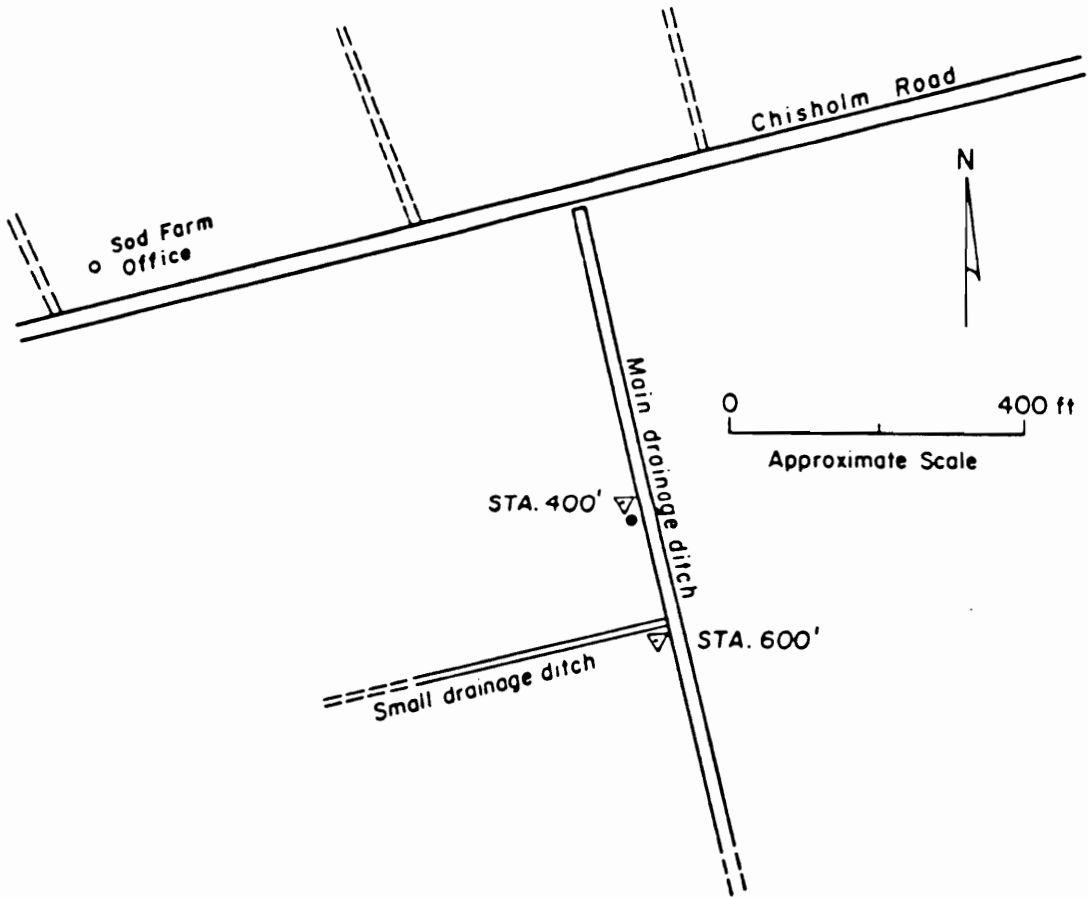
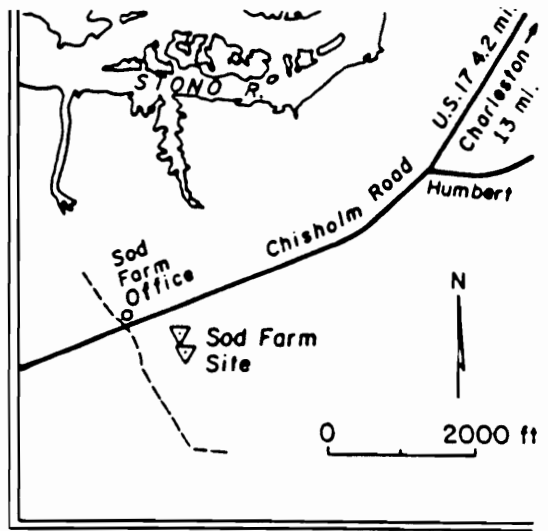


Figure 5.12 Location Map for the Sod Farm Site.

encountered at the Hollywood and Warren sites. Hence, the upper soils were pre-augured for the mini-cone CPT's, and the tests were begun at about 8 ft. Cuttings from the auger borings indicated that clean, fine sands constitute the pre-augured portion of the soil profile. The results of the CPT's are shown in Figs. 5.13 and 5.14. From the starting depth of 8 ft. down to 13 ft., the sands were found to be clean, and medium to loose in density, with tip resistances ranging from 25 to 100 bars. Corresponding relative densities range from 20 to 65%. Below 13 ft., the sands grade into a varved system of sandy clays and clayey sands which extend to a depth of 32 ft. From 32 ft. to the bottom of the boring at 35 ft., a dense sand layer is found. The water table was found at a depth of 4 ft. during auguring operations.

5.1.4 Ten Mile Hill, Eleven Mile Post, and Airport Sites

The Ten Mile Hill and Eleven Mile Post sites are located along the South Carolina Railroad (SCRR) line approximately 10 miles (hence the name) northwest of the City of Charleston and adjacent to the new Charleston International Airport/Air Force Base. Site maps are given in Figs. 5.15 and 5.16. The area lies within a 200,000 yr.-old beach environment, and the water table is within 5 ft. of the ground surface. Ten Mile Hill is important because

SITE: SOD FARM STA. 400'

CHARLESTON LIQUEFACTION STUDY

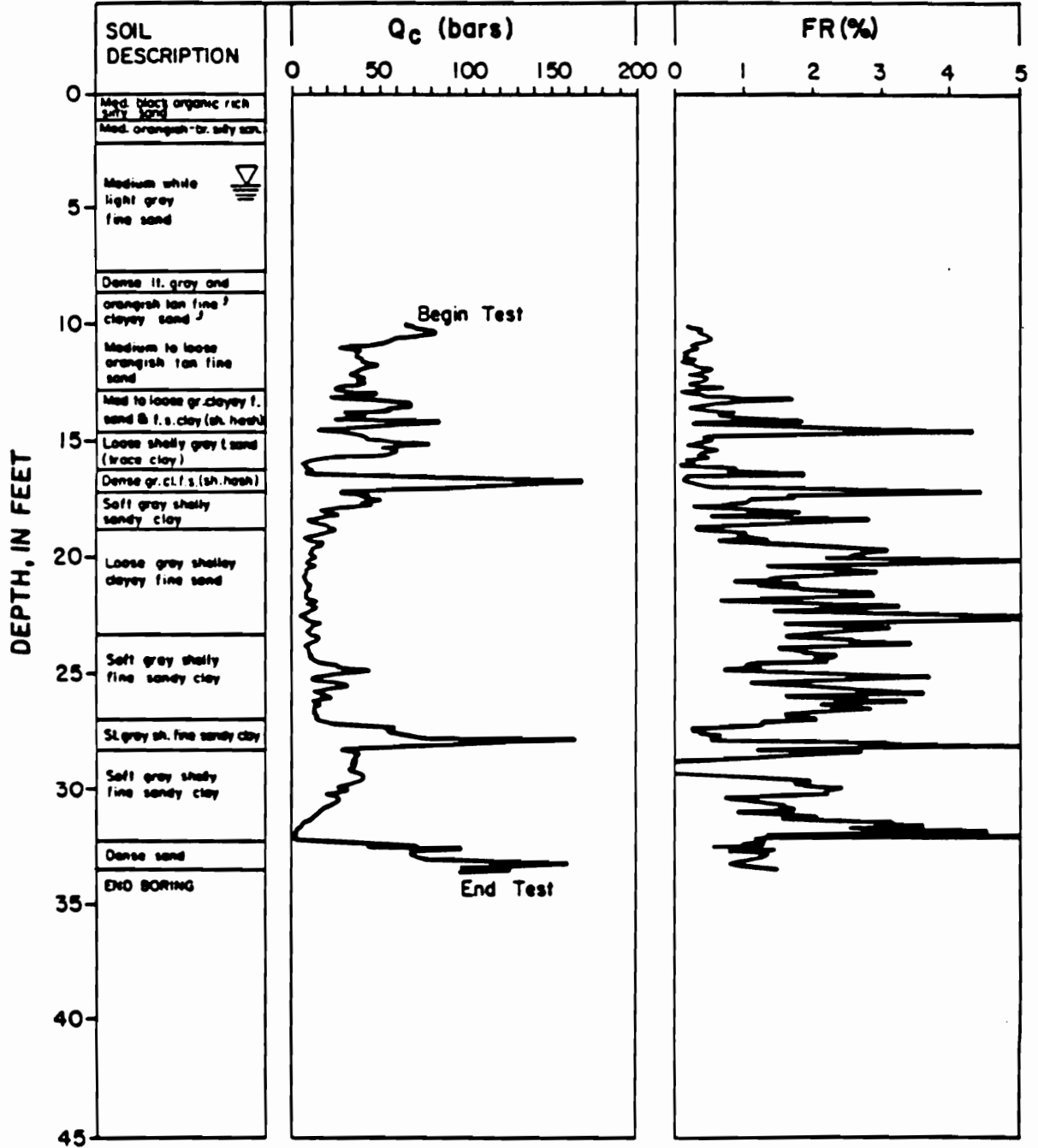


Figure 5.13 CPT Record for Sod Farm Station 400.

SITE: SOD FARM STA. 600'

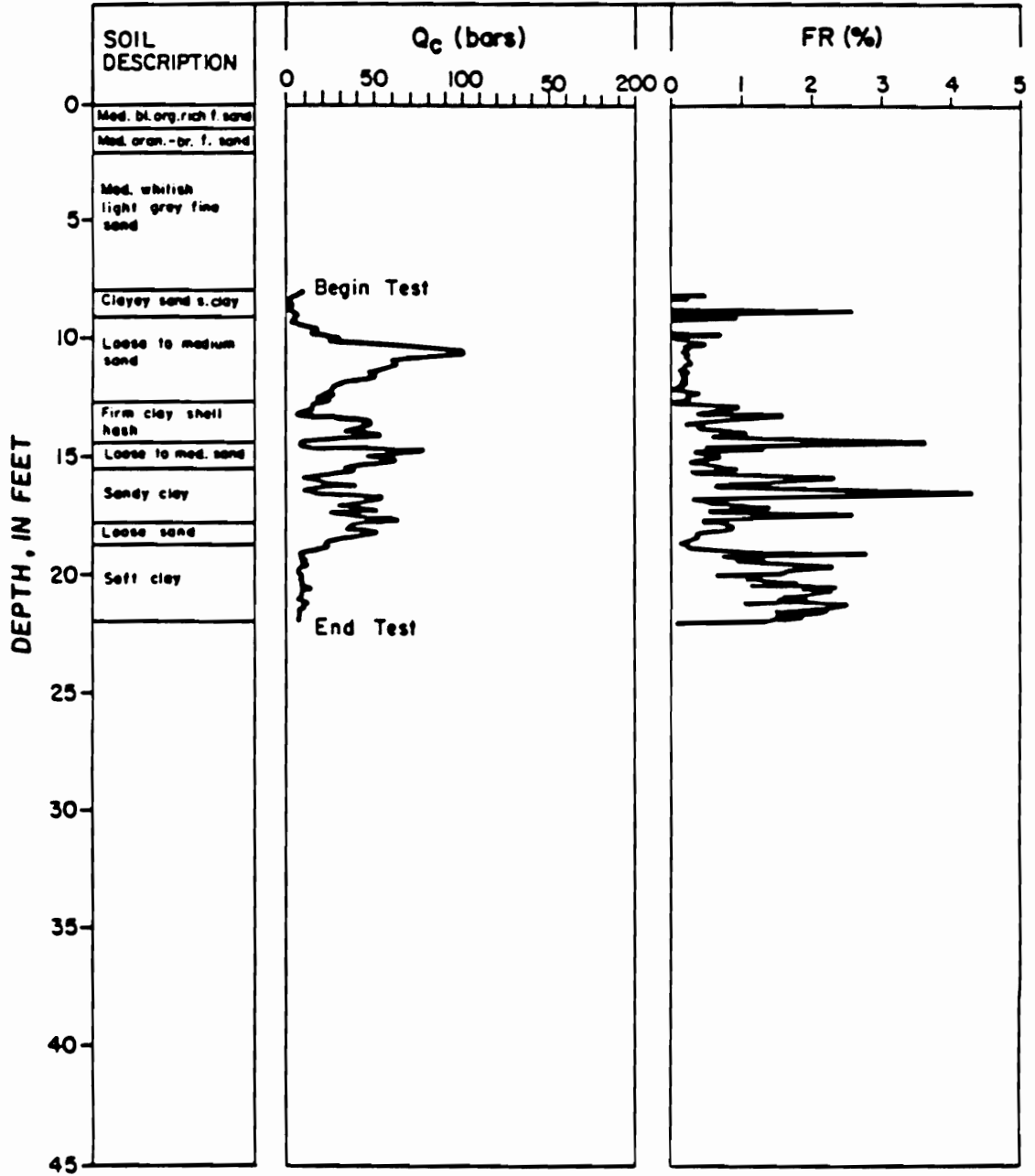


Figure 5.14 CPT Record for Sod Farm Station 600.

Ten Mile Hill Site

Located on Ladson, S.C. Quad
USGS 7.5 minute topo

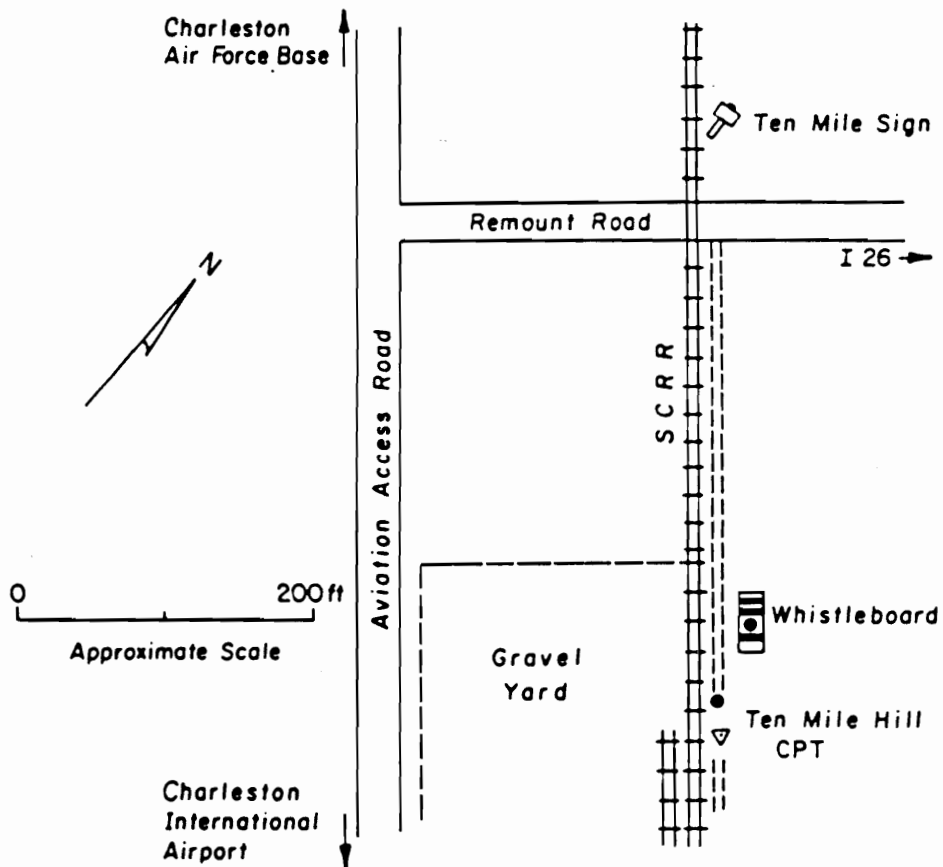
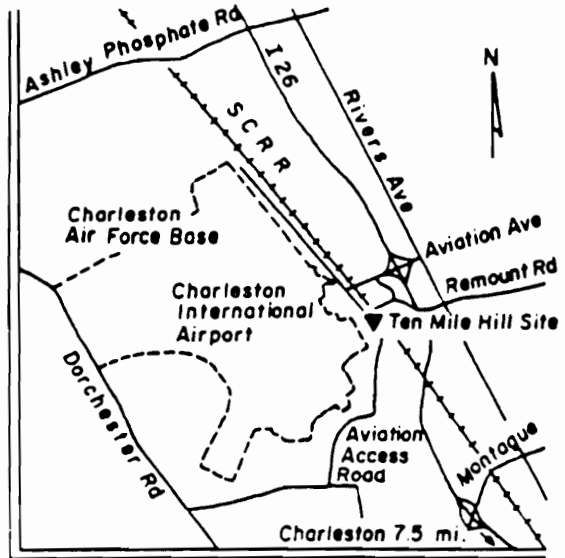


Figure 5.15 Location Map for the Ten Mile Hill Site.

Eleven Mile Post Site

Located on the Ladson, S.C. Quad
USGS 7.5 minute topo

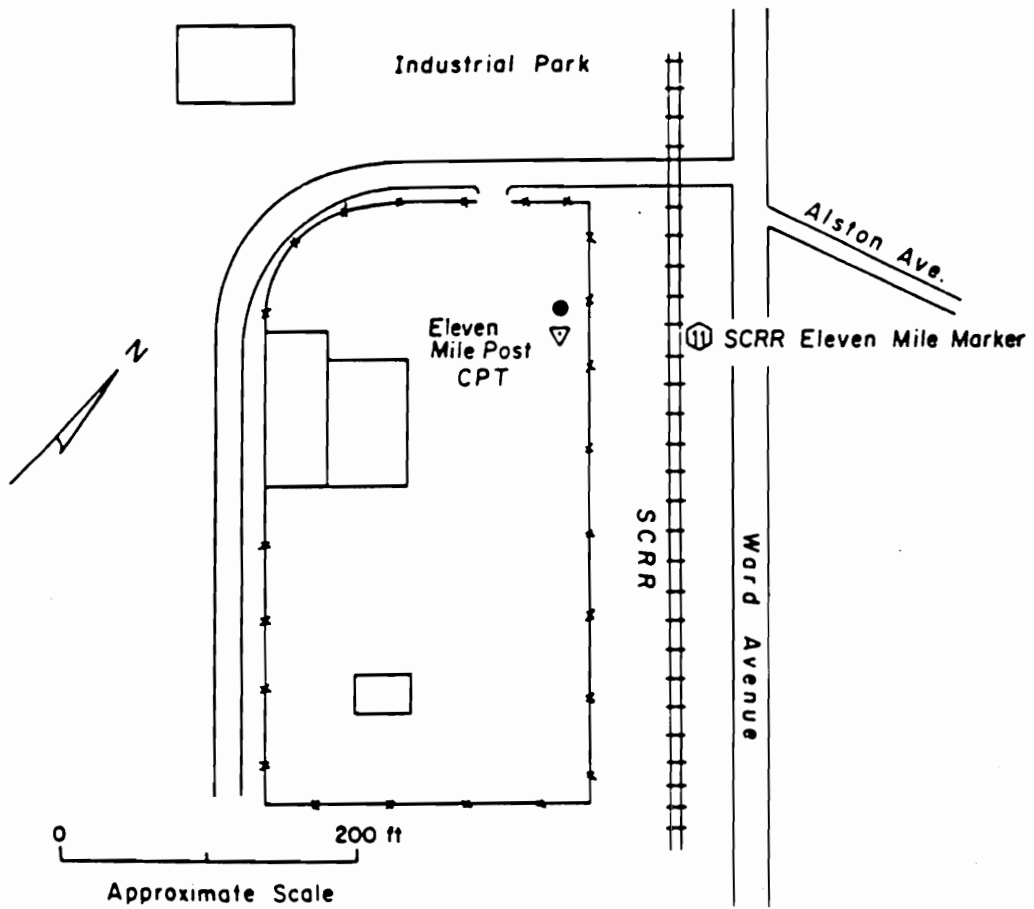
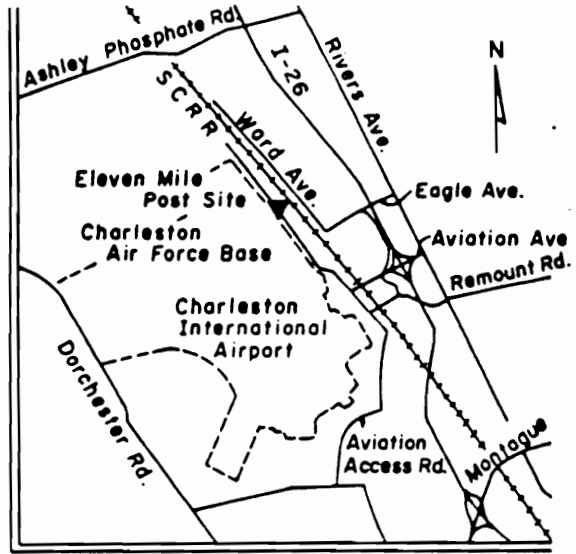


Figure 5.16 Location Map for the Eleven Mile Post Site.

of its historical significance, having attracted much attention as a result of large sand craters that were formed there during the 1886 earthquake (see photograph in Fig. 2.5). The region represents the most extensively liquefied area observed anywhere in the low country, and was described by Sloan (in Dutton, 1889) as being:

"honeycombed with craterlets affording abundant quicksand ... composed of sandy subsoil extending 8 to 12 feet below the surface and overlying bed of quicksand defying efforts (immediately after the earthquake) to sink an artesian well pipe".

Test sites in the Ten Mile Hill area were chosen on the basis of first-hand reports from field reconnaissances performed along the SCRR after the 1886 earthquake. Contained in Dutton's 1889 report is a collection of photographs from "George L. Cook's Earthquake Views" which illustrate ground disruptions and other earthquake damages near Ten Mile Hill. Included in Cook's work is a photo of the large, eroding craterlet shown in Fig. 2.5, but taken from a different angle. In the background of the photo can be seen a switch post, side track, and whistle board signifying a nearby crossing. After a review of old SCRR maps revealed the location of the side track and railroad crossing, the location of the sand crater was estimated to be within a few hundred feet and the Ten Mile test site placed there. The Eleven mile post site lies one mile fur-

ther along the SCRR tracks and was similarly selected on the basis of observations by Sloan. He reported that the area near the Eleven Mile post was conspicuous for craterlets "in belts conforming to the [beach] ridges" which trend perpendicular to the tracks.

One or two CPT's and auger borings were conducted at each of the sites. In Fig. 5.17, the CPT and soil log from the Ten Mile sign post site is shown. Due to the presence of cobbles and dense sands in the upper portion of the soil profile, the CPT hole was pre-augured to 5 ft. Below 5 ft., the CPT revealed a 2.5 ft. layer of loose sand with Q_c values close to 50 bars and an average relative density of 45%. Two layers of fine sand were encountered between 7.5 and 16 ft., with the upper layer extending from 7.5 to 11 ft. and exhibiting tip resistances ranging from 40 to 80 bars. Corresponding relative densities for this layer average 60 to 65%. The lower sand layer extends from 11 to 16 ft. and has an average Q_c and relative density of 105 bars and 75%, respectively. Below 16 ft., the sands become very dense and the CPT had to be terminated because of limited capacity. The auger boring at this location however, indicated fine sands continuing to Cooper Marl which was encountered at a depth 42 ft. During auguring, it was noted that the sands remained dense with depth. Gradation curves for sand samples taken from the lower sand layer

SITE : TEN MILE HILL

CHARLESTON LIQUEFACTION STUDY

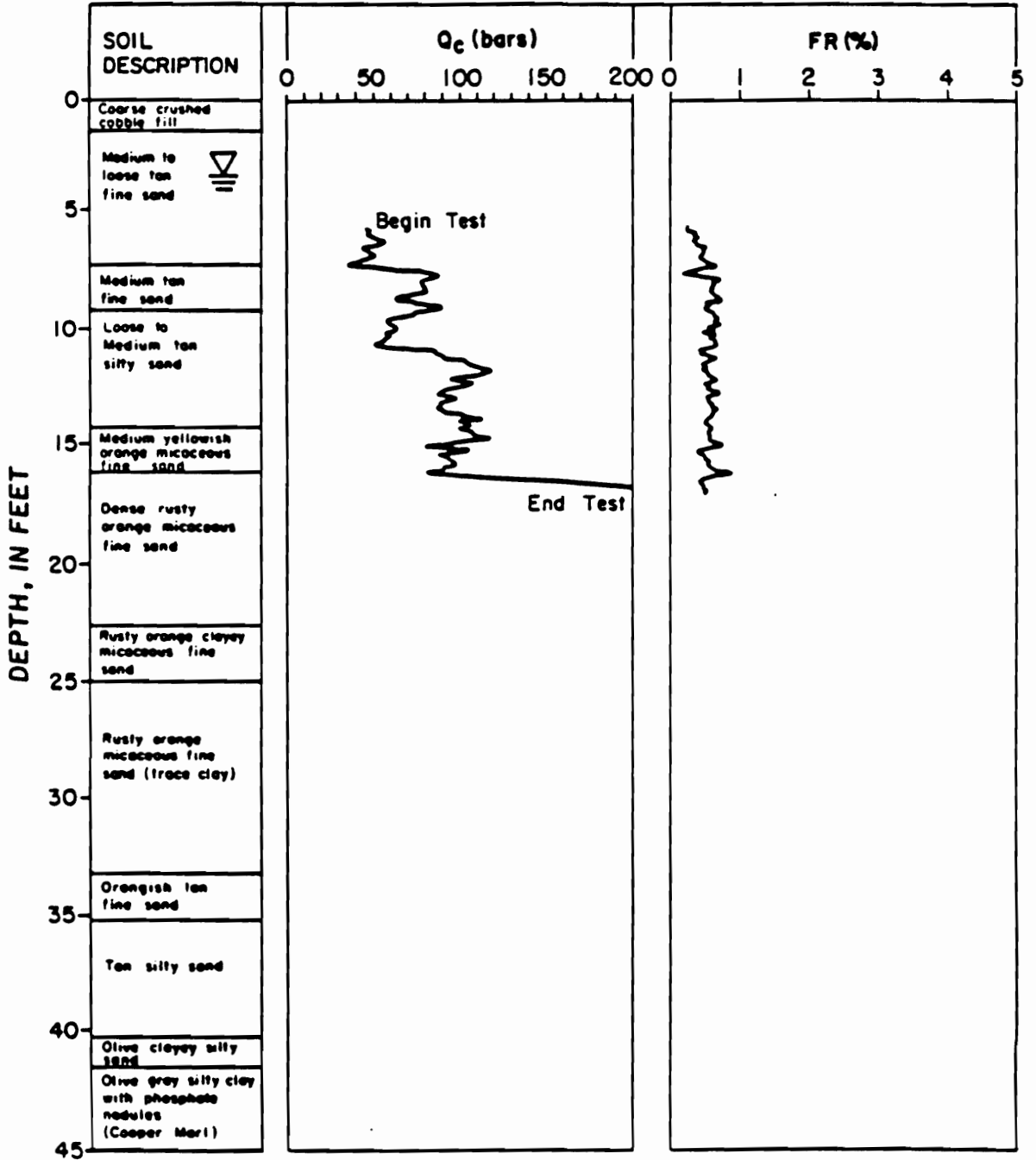


Figure 5.17 CPT Record for the Ten Mile Hill Site.

(12.5 to 15 ft.) are shown in Fig. 5.18 and indicate that the material is uniform ($C_u = 2.14-2.19$) and fine ($D_{50} = 0.14$ mm). The sieve analyses revealed a fines content of 11%, 10% of which was non-plastic silt. The sand particles were found to be sub-angular and non-cemented.

The soil profile and CPT results from the Eleven Mile sign post site are shown in Fig. 5.19. The upper soils at this location are dense fine sands and are underlain by a series of medium to loose layers of silty to fine sand. Of particular interest is a loose sand layer which extends from 10 to 17 ft. Tip resistances throughout this layer remain consistent at about 50 bars, which corresponds to an estimated relative density of 40%.

Charleston International Airport/ Air Force Base

In addition to the Virginia Tech field work, 50 SPT borings were obtained from nearby Charleston Airport (see map Fig. 5.15). The borings indicate that soil conditions are variable, consisting of clean to silty sand, clayey sand, sandy clay, and clay, underlain by Cooper Marl. The majority of the borings revealed profiles of silty fine sands interbedded with clays. Some areas were found to consist entirely of sands, although these strata are apparently

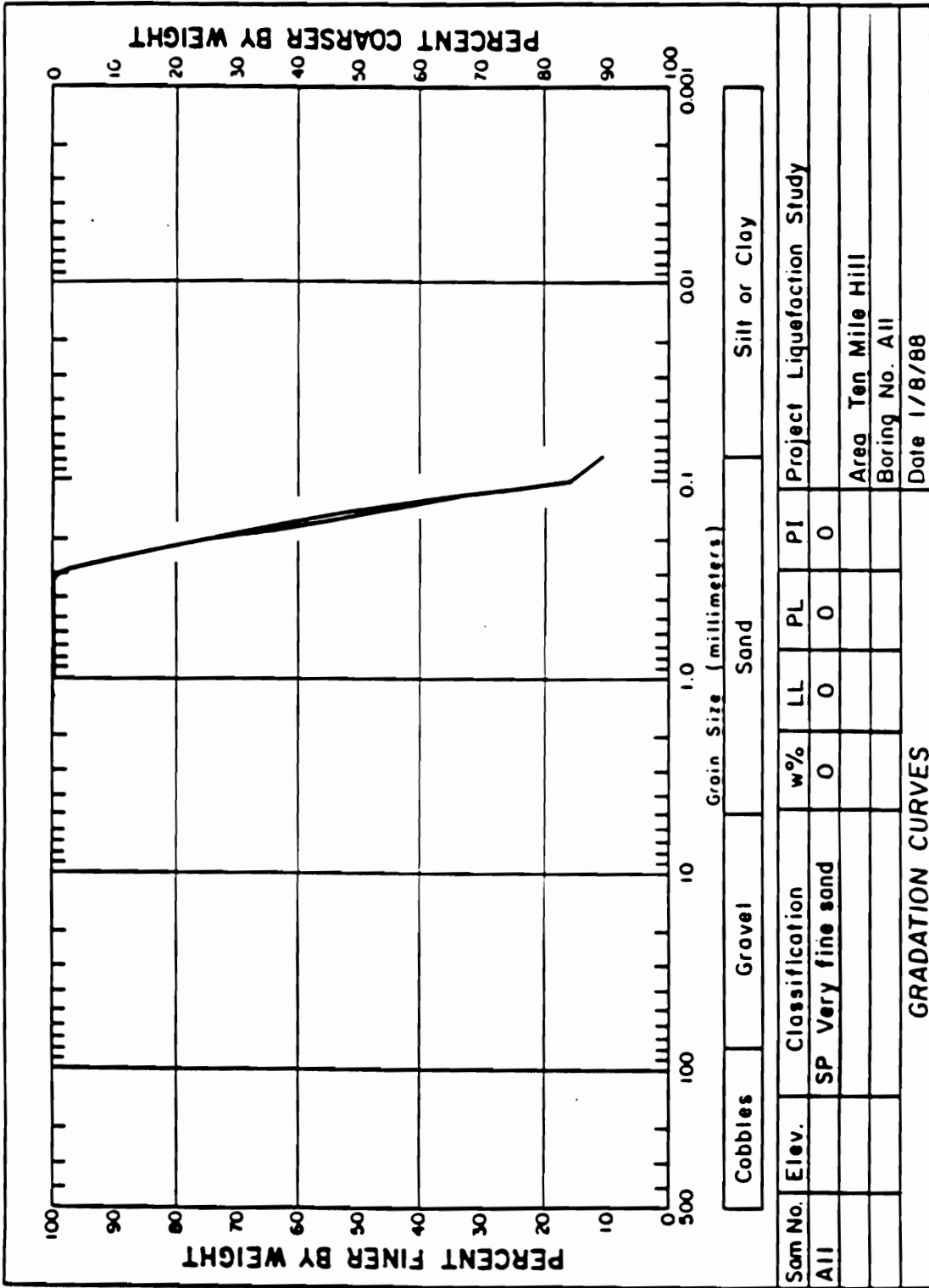


Figure 5.18 Range of Gradation for Ten Mile Hill Samples Taken at Depths Between 12.5 and 15.0 Feet.

SITE: 11 MILE POST

CHARLESTON LIQUEFACTION STUDY

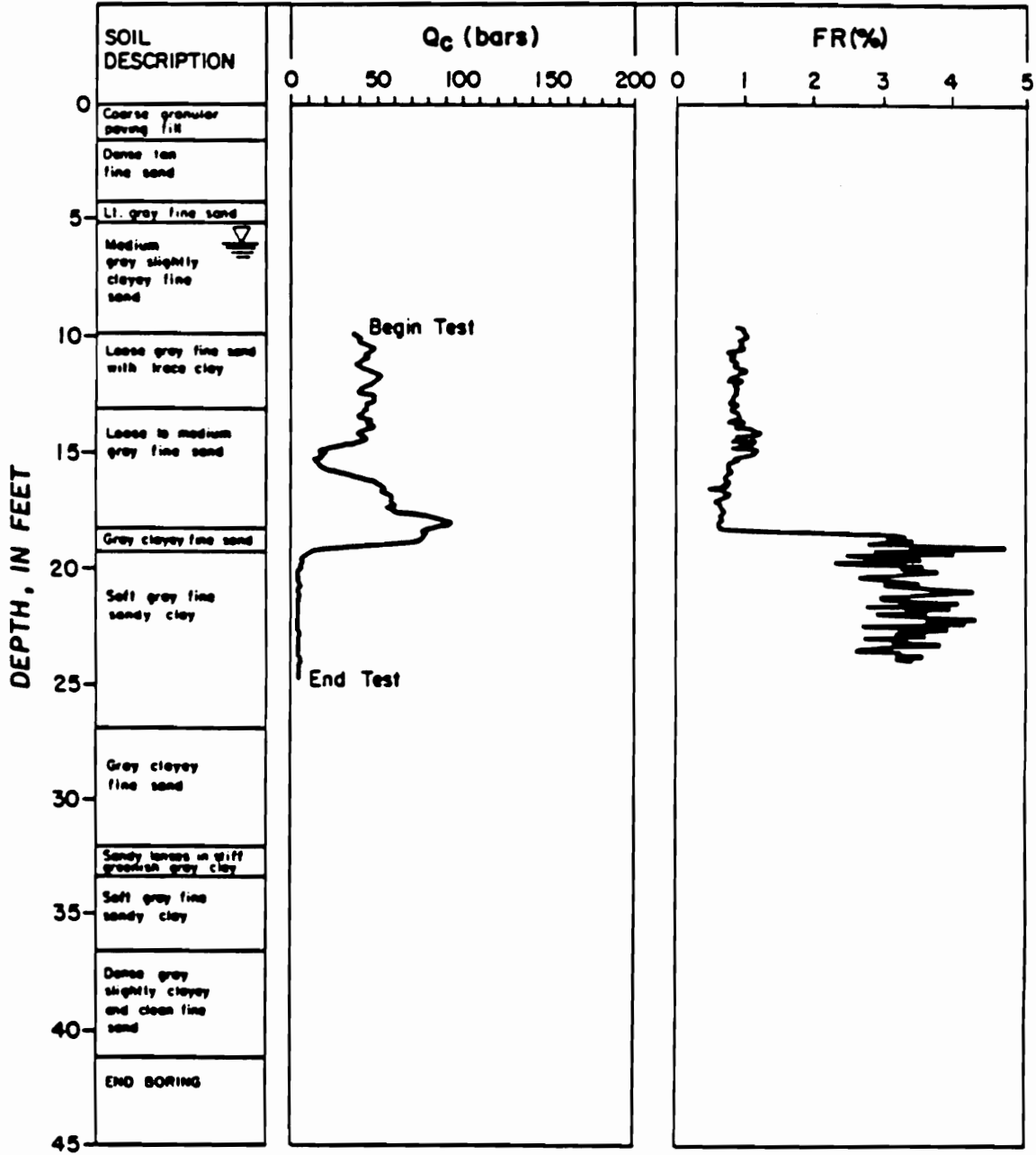


Figure 5.19 CPT Record for the Eleven Mile Post Site.

not continuous. The sediment thickness above the Cooper Marl ranged from about 20 to 40 ft., with the water table positioned at 4 to 8 ft. below the ground surface.

Fig. 5.20 shows a typical soil profile from the Airport area consisting of a silty fine sand stratum interbedded with a layer of sandy clay. Included in the figure are blowcounts from SPT's obtained in the borings. The upper sands are shown to yield blowcounts ranging from 5 to 30 blows/ft. with an average value of about 15 blows/ft. The underlying sand layer, which extends from 12 to 20 ft., is consistently lower in density and yields blowcounts ranging from 8 to 14 blows/ft. A corresponding relative density of 50% is assigned to this layer. In Chapter 6 of this report, all 50 of the SPT borings are used in a liquefaction analysis of the Airport.

5.1.5 Montague Site

The Montague site is located in the Ten Mile Hill vicinity, situated seven miles northwest of Charleston and one mile east of the South Carolina Railroad line. The site lies in the 200,000 yr.-old beach environment and the water table is within 5 ft. of the ground surface. As indicated by field reconnaissances along the railroad following the 1886 quake, this area was relatively free of significant ground disturbances (Dutton, 1889; Peters and Herrmann,

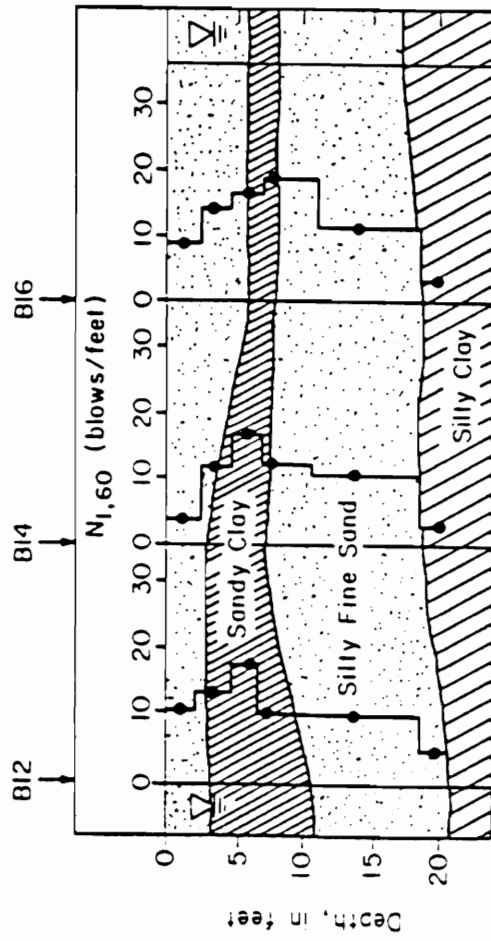
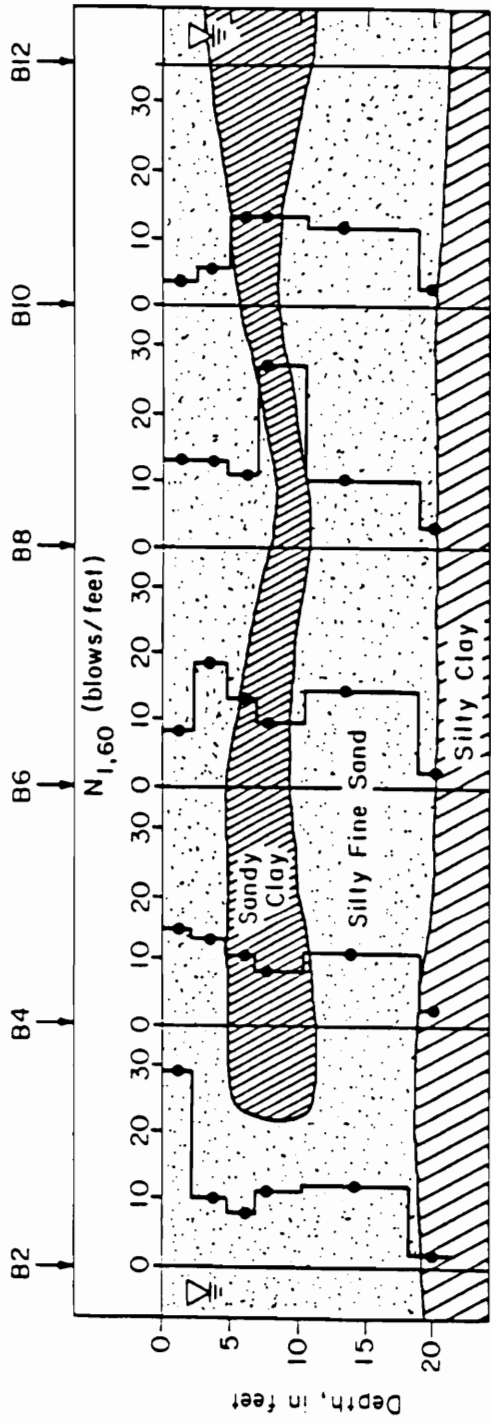


Figure 5.20 Soil Profile Showing SPT Data From Charleston Airport/AFB (Ten Mile Hill Area).

1986). Paleoliquefaction work by Obermeier revealed a small liquefaction vent located in a drainage ditch at the site which prompted field work by Virginia Tech.

Due to time constraints, only one auger boring and cone test were performed at the site. The tests holes were located adjacent to the feature as shown in Fig. 5.21. The boring log and results of the CPT are illustrated in Fig. 5.22. The upper portion of the soil profile consists of 18 feet of clean fine sand, the top 10 ft. of which was estimated during auguring to be dense. The CPT, which began at 10 ft., showed a consistent decrease in penetration resistance with depth, ranging from more than 200 bars (90% D_r) at 10 ft. to less than 100 bars (50 - 55% D_r) at 17 ft. Considering the relatively uniform nature of the old beach deposits, it would be expected that this sand stratum is extensive, although the nature of the density variations across the site cannot be determined without additional penetration data.

5.1.6 Five Mile Site

The Five Mile site is situated along the SCRR, five miles northwest of Charleston, about 9 miles from the zone of energy release, and just inside of Bollinger's meizoseismal zone (see Fig. 3.2). The geologic profile in this area is a combination of 85,000 year-old marine soils

MONTAGUE SITE

Located on John's Island, S.C. Quad
USGS 7.5 minute topo.

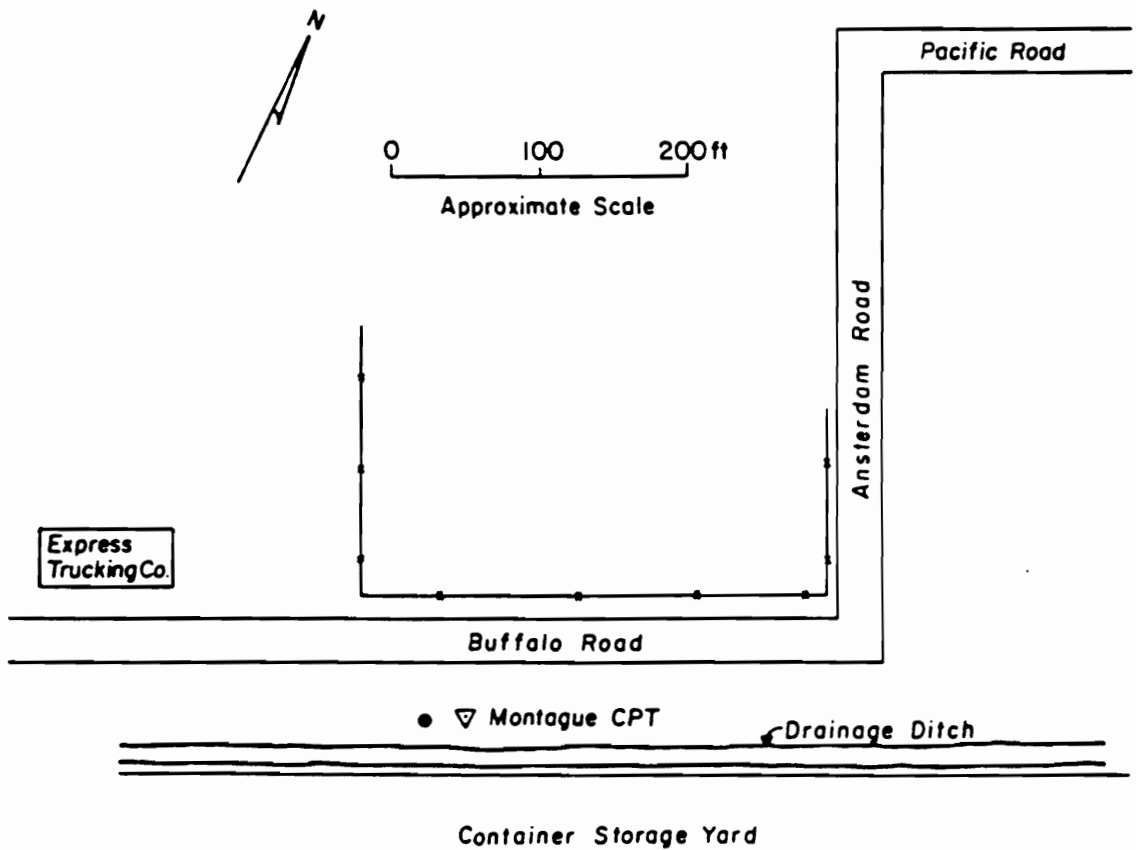
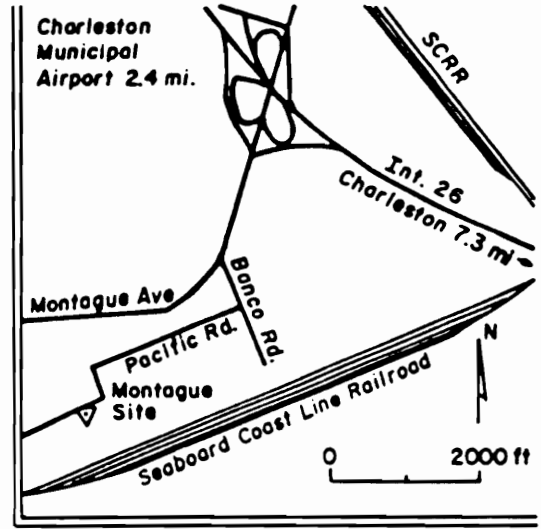


Figure 5.21 Location Map for the Montague Site.

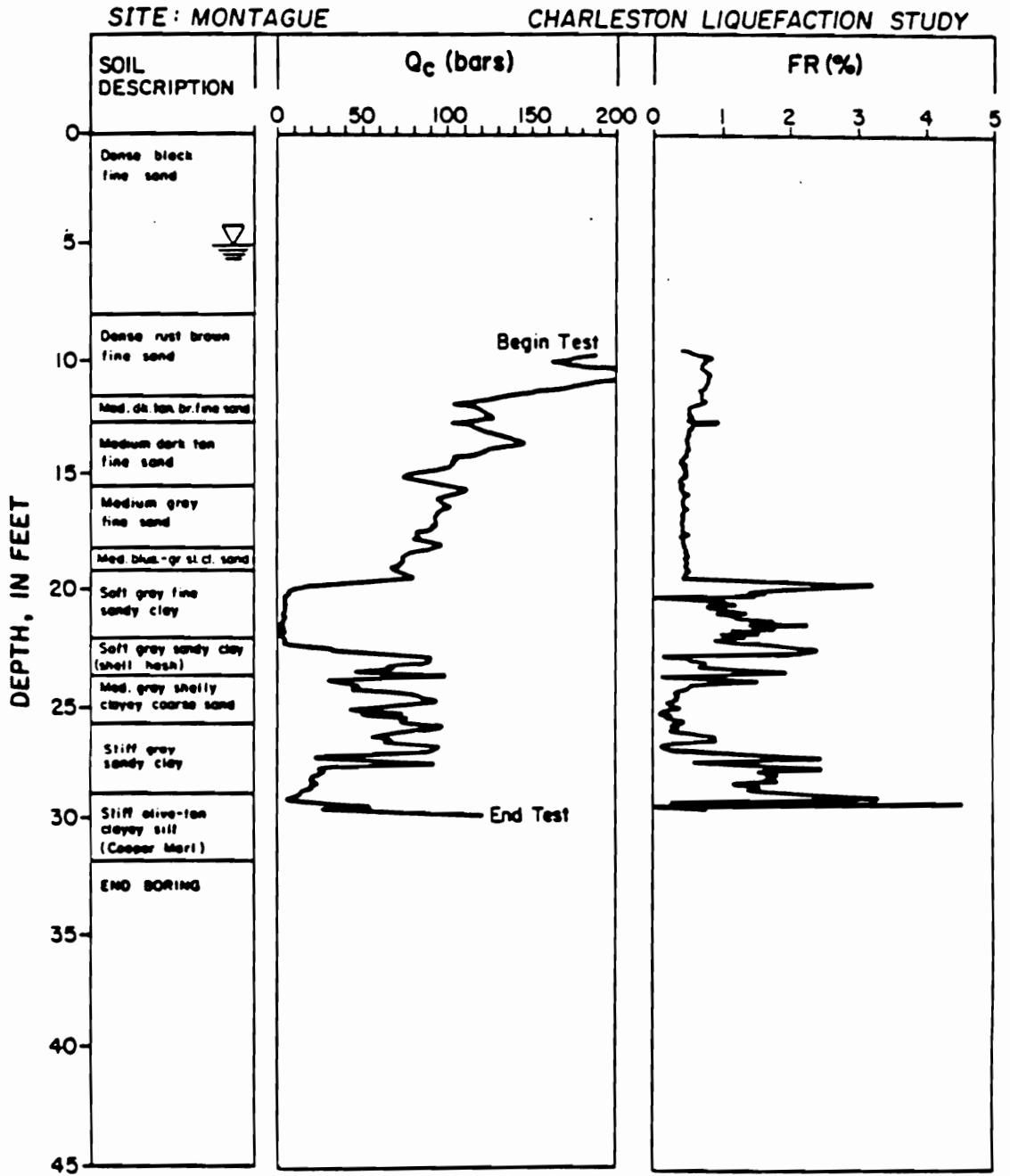


Figure 5.22 CPT Record for the Montague Site.

mixed with recent fluvial sediments laid down by stream systems. Historical accounts indicate that Five Mile was an area of minor liquefaction during the 1886 earthquake. Observations of liquefaction features and other ground disruptions at the site were reported by Sloan during his reconnaissance along the SCRR following the quake: "...Thence to near the 5-mile point no marked disturbance was found beyond the occurrence of cracks and small sand craters. At the 5-mile point the track again showed signs of great stress...".

Ten SPT Borings were obtained for a building site at the Five Mile site. A soil profile which includes the SPT's is given in Fig. 5.23. The profile consists of 10 to 18 ft. of loose silty fine sands and clayey sands underlain by soft silt and clay to 30 ft. Normalized blowcounts in the upper sands range from 1 to 45 blows/ft. and average about 17 blows/ft. The water table at this location was found at 2 ft. below the ground surface.

SOIL INFORMATION OBTAINED FROM OUTSIDE SOURCES

SPT borings and CPT data were collected on an as-available basis from commercial firms and government agencies for sites that fell within the meizoseismal zone. A major

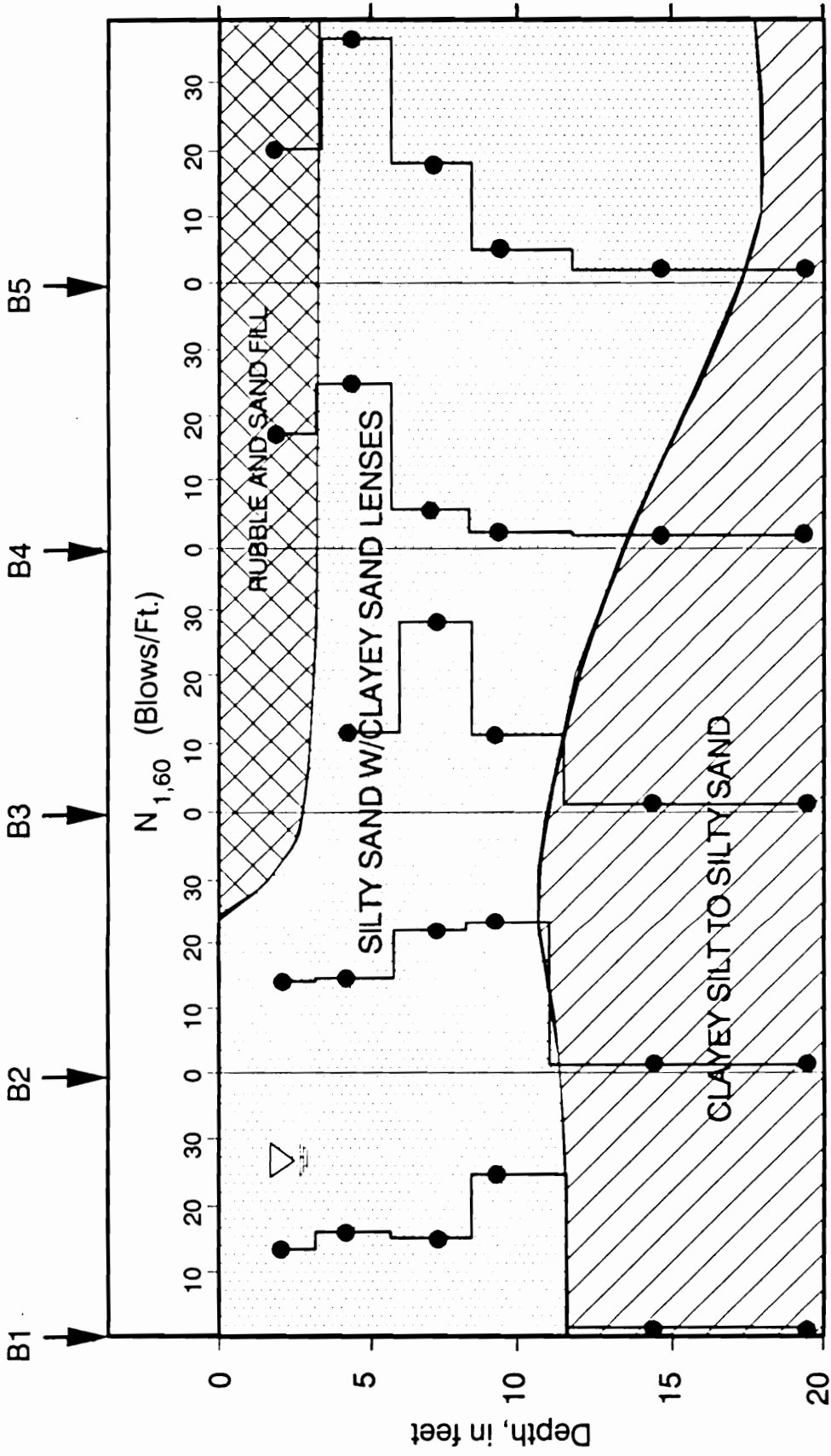


Figure 5.23 Soil Profile Showing SPT Data from the Five Mile Site.

portion of the data that was available was obtained from sites located in the Ten Mile Hill/Airport area, although some data were available for the Hollywood area. Because no detailed field studies were done at any of the sites from which these data were obtained, knowledge of the amount of liquefaction activity that occurred at each site is not clearly defined. However, these data serve to reinforce the findings of the field investigations presented earlier, and are used in the liquefaction analyses presented in the next chapter.

More than 130 borings were collected from 20 sites within the meizoseismal zone, and Table 5.1 lists these data. The borings indicated that:

- 1) Although soil conditions are variable across the meizo-seismal zone, significant zones of readily liquefiable materials are present at many locations.
- 2) Soil conditions appear to be uniquely related to the various geologic settings; Sites that were located in the 200,000 yr.-old beach deposit near Ten Mile Hill generally consisted of liquefiable fine to silty sands interbedded with non-liquefiable materials, while sites in the 120,000 yr.-old Hollywood area consisted of non-liquefiable materials near the ground surface with

Table 5.1 Summary of Soil Information Obtained From Outside Agencies for Sites Within the 1886 Meissosismal Zone

Site	Distance From Release (mi.)	Extent of(1) Liquefaction 1886	Geologic(2) Environment\ Age Classification	Number of SPT's	Avg N _{1,60} (Blows/ft.)(3)	Avg Thickness		USCS Soil(4) Classification
						Liquefiable Layer (ft.)	Capping Layer (ft.)	
MZ- 2	7	None	Beach/Interm.	4	13	17	4	SM
MZ- 6	13	Minor	Fluv. & Beach/Recent	4	18	14	8	SM
MZ- 7	12	None	Fluv. & Beach/Recent	6	19	9	8	SM
MZ- 8	8	Minor	Fluv. & Beach/Recent	23	17	12	10	SM
MZ-18	1	Minor	Backbarrier/Very Old	8	16	12	7	SM
MZ-21	5	Minor	Backbarrier/Very Old	7	23	7	3	SM
MZ-23	7	None	Backbarrier/Old	3	11	6	5	SP-SH
MZ-47	5	Minor to Mod.	Beach/Old	3	8	> 4	10	SP-SH
MZ-B	7	Very Minor	Beach/Young	4	17	9	4	SH
MZ-C	4	None	Beach/Young	5	16	8	5	SP-SH
MZ-D	4	None	Beach/Young	5	20	9	5	SH
MZ-I	5	None	Fluvial/Recent	10	17	3	6	SH
MZ-L	6	None	Beach/Very Old	8	16	5	5	SM
MZ-O	2	Minor	Backbarrier/Old	6	25	11	6	SP
MZ-P	2	Minor	Backbarrier/Old	3	18	11	7	SM
MZ-Q	0	Minor	Backbarrier/Old	4	22	7	12	SP-SC
MZ-R	7	Very Minor	Fluvial/Recent	8	18	7	12	SP
MZ-S	5	Minor	Beach/Old	7	16	> 17	3	SP
MZ-T	11	None	Fluv. & Beach/Recent	2	17	5	5	SM
MZ-V	4	Minor	Backbarrier/Young	13	21	7	5	SM

(1) Based on Historical Observations in General Vicinity of Site

(2) Age Classification: 1) Very Old = > 400,000 yrs; 2) Old = ~130,000 to 230,000 yrs; 3) Young = ~85,000 yrs; 4) Recent

(3) Includes Correction for Fines Content.

(4) Based on Visual Classification.

readily liquefiable sands at depth. Sites that fell within the fluvial deposits were more variable than the beach deposit sites, and generally consisted of coarser sands with interbedded clays and silts. Sites located in backbarrier areas were generally less liquefiable, consisting primarily of silts and clays.

5.2 SITES OUTSIDE THE 1886 MEIZOSEISMAL ZONE

Ten sites outside the meizoseismal zone were investigated by borings and penetration tests. All physical evidence and known facts about the attenuation of the earthquake motions suggest that the shaking from the 1886 event at these sites was less than that felt within the meizoseismal zone. The lack of significant ground disruptions and liquefaction-related ground failures throughout this region suggests that soil conditions are similar to those which existed at the time of the earthquake. Additional soil data were obtained from commercial firms for sites in this region.

VIRGINIA TECH SITE INVESTIGATIONS

5.2.1 Oakland Plantation Site

The Oakland Plantation is located 10 miles northeast of Charleston along U.S. Highway 17, and 7 miles from Bollinger's meizoseismal zone boundary (see Fig. 3.2). Located behind the central grounds of the historic plantation are several borrow pits which have been excavated for the mining of sand. In investigating the borrow areas, Obermeier discovered a long, thin seam of vented sand which cut through the overlying organic layer along the wall of a nine-foot-deep dewatered sand pit (see photograph in Fig. 3.1). The vent was believed to have been induced by the 1886 quake. No other relic liquefaction features or evidence for significant ground disruption were discovered, indicating that minimal liquefaction occurred at the site and that soil conditions should be essentially unchanged from 1886.

A total of 7 CPT's, 1 SPT, and 1 auger boring were performed around the edges of the sand pits. The locations of the test holes are shown in Fig. 5.24. As indicated, a portion of the drilling and testing was done at the location of the feature. The penetration tests and borings show that clean sands dominate the soil profile to depths of over 30 ft. Fig. 5.25 shows SPT and CPT results from tests

OAKLAND PLANTATION SITE

Located on Fort Moultrie, S.C.
Quad. USGS 7.5 minute topo.

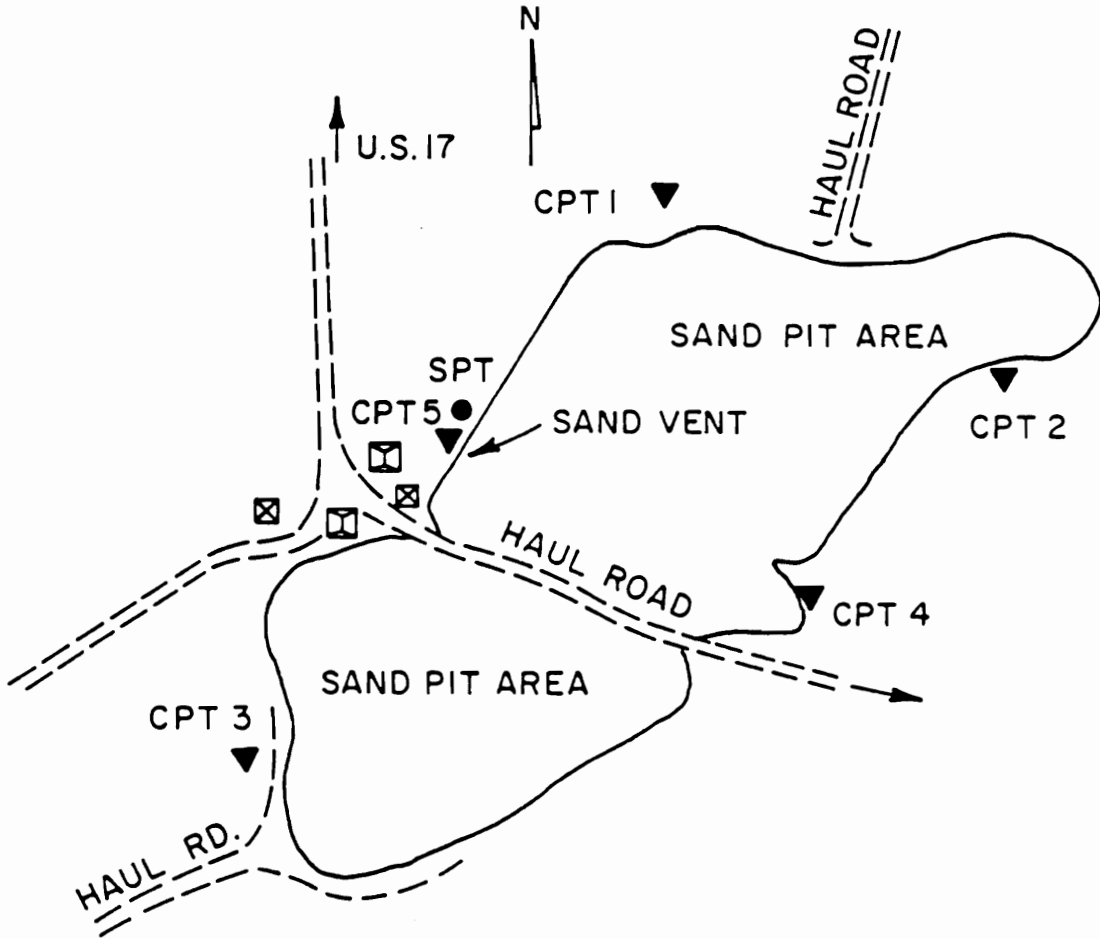
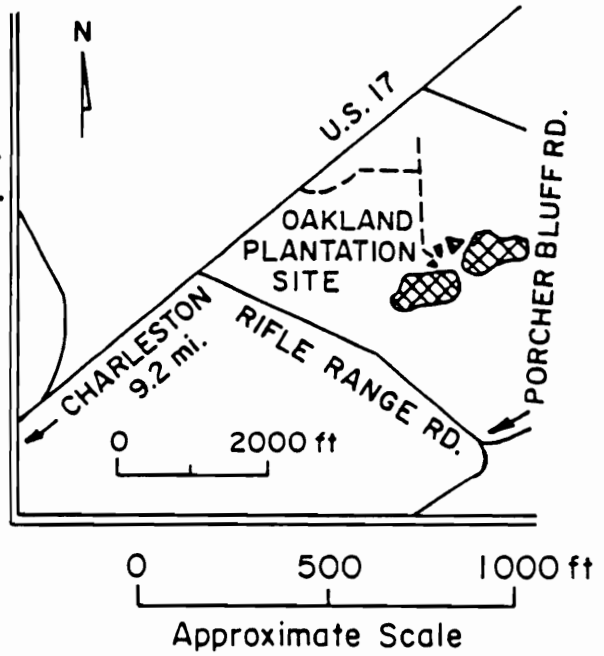


Figure 5.24 Location Map for the Oakland Plantation Site.

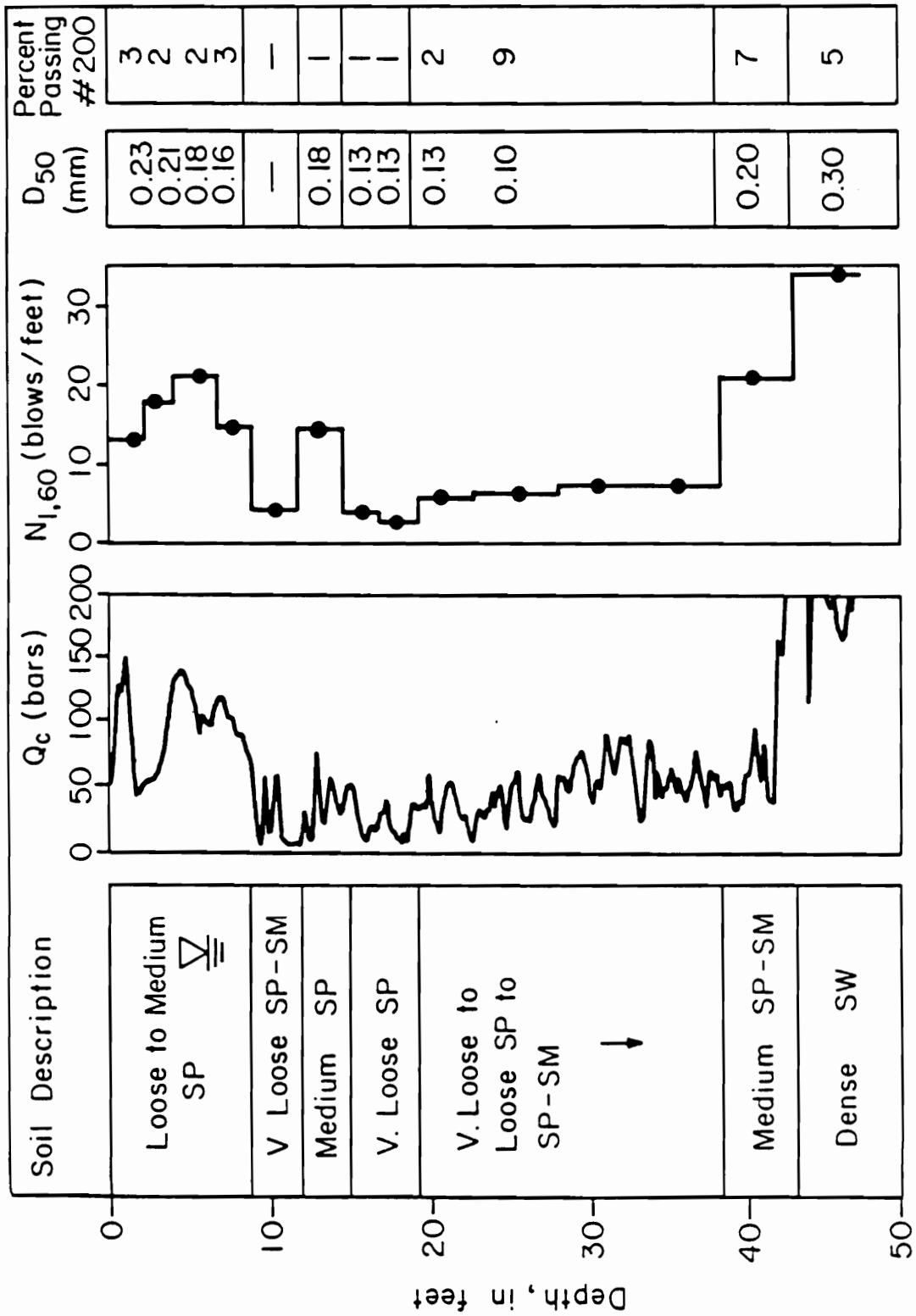


Figure 5.25 Soil Information for the Oakland Plantation Site. Data Shown is From Tests Near 1886 Liquefaction Feature.

performed near the feature. Included in the figure is information from classification tests performed on samples obtained during SPT testing. The tests reveal loose to medium sands in the upper 10 ft. of the profile, with blow-counts ranging from 12 to 20 blows/ft., and tip resistances ranging from about 50 to 150 bars. A significant zone of loose to very loose clean sand occurs from about 10 to 35 ft. with consistently low blow-counts and tip resistances. Blow counts average 6 blows/ft. throughout the 25-foot layer, while tip resistances remain close to 40 bars. A relative density of 30% is estimated for this stratum. The classification information shows the sand to be fine, with an average median grain size of 0.13 mm and a fines content of less than 5%. The gradation curves for the samples are given in Fig. 5.26 and show the sand to be uniform. The water table was located at a depth of 6 ft. at the time of testing, most likely lowered as a result of drainage into the sand pits. It seems reasonable that the water table was within a few feet of the surface before mining commenced.

Penetration tests at other locations at the site showed similar density patterns to those described to this point, although the presence of loose sands was not as common as shown in Fig. 5.25. A histogram of Q_c values taken from a depth of 8 to 24 ft. for 6 CPT's is given in Fig. 5.27. The values shown were obtained by averaging the Q_c

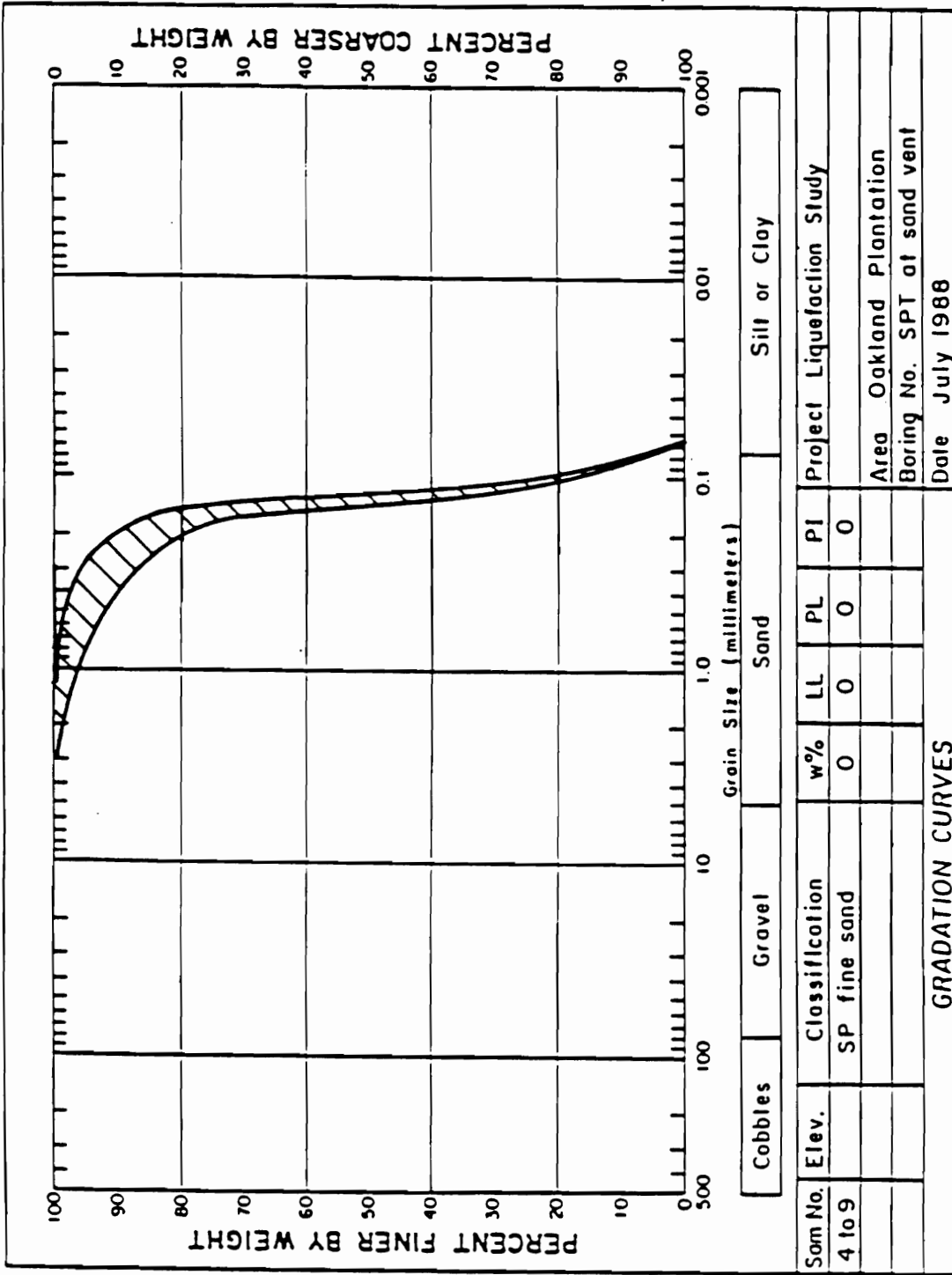


Figure 5.26 Range of Gradation For Oakland Plantation Samples Taken at Depths Between 8 and 24 Feet.

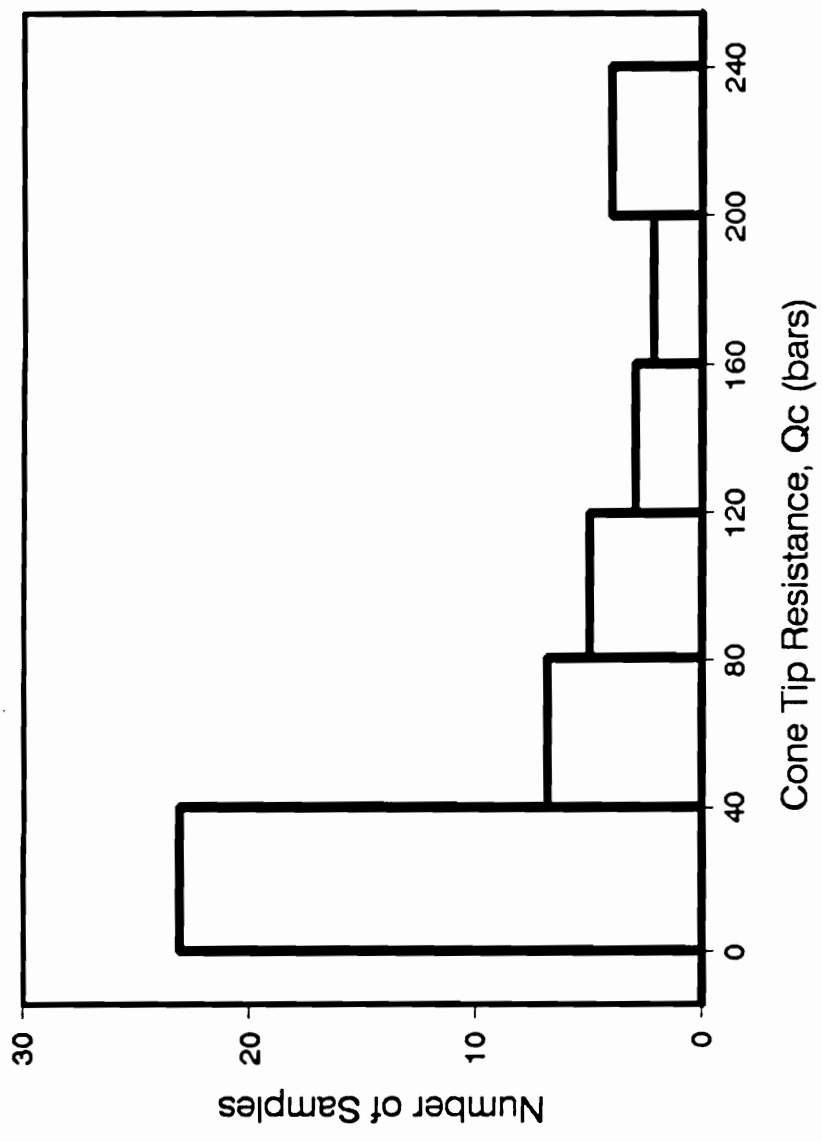


Figure 5.27 Histogram of Q_c Values From 8 to 24 Feet for 6 CPT's Performed at Oakland Plantation.

resistance in 2-foot increments from 8 to 24 ft. Examining the figure, a grouping of values occurs at less than 40 bars indicating that low-density sands are prevalent across the site. Within the framework of conventional liquefaction analyses, it is expected that relatively low levels of accelerations would liquefy these materials, yet little evidence of liquefaction during the 1886 quake was found. This observation provides insight into the upper level of ground shaking that occurred at the site during the 1886 earthquake.

5.2.2 The City of Charleston

The City of Charleston is located on a peninsula flanked by the Cooper and Ashley Rivers and bounded in the front by the Atlantic Ocean. The city is located approximately 13 miles from the zone of energy release and just outside of the 1886 meizoseismal zone. The makeup of the surficial soils is more complex than the inland sites in that the rivers have left fluvial deposits mixed with the marine sediments, and because man-made fills have been placed over large areas of the fringes of the peninsula. The first-hand accounts indicate that sand boils and other liquefaction-related ground failures occurred in both natural and "made" ground during the 1886 event, and it is believed that a significant portion of the building damage

throughout the city was caused by these failures (Robinson and Talwani, 1983). The most severe building damage, and from what is known, the most soil liquefaction, occurred in the fill areas, although it has been suggested that site period variations also played a role in the selective pattern of building damage (Elton and Martin, 1989).

No penetration testing was performed in the City of Charleston by Virginia Tech personnel, although specific site investigations were conducted. For this study, SPT logs for more than 60 building sites in the downtown area were obtained from outside sources. Some of the SPT's were performed at or near sites where liquefaction occurred during the 1886 event, and two of these cases are given here. The remaining data are presented in histogram format near the end of this chapter. Collectively, the SPT's indicate that although there is variability in the soil conditions, many areas have characteristics appropriate for liquefaction.

ST. MICHAEL'S CHURCH

One site of particular interest is that of St. Michael's Church where liquefaction took place beneath the structure during the 1886 event. The church was built

between 1751 and 1761 on one of the highest natural ground sites in the city. The distinctive steeple of the church, which served as a conspicuous landmark for incoming ships, sits atop a massive 168-foot tower located at the front entrance. No piles were driven to support the tower or any other portion of the church. During the earthquake it was reported that multi-colored sands were vented inside the church through cracks in the center aisle (A Guide to St. Michael's Church, 1979) and that the steeple tower settled eight inches (Dutton, 1889). Interestingly, the church historian indicated that several jar samples of the vented sand were recovered from the floor of the church and kept there for many years, although he was uncertain as to their current whereabouts.

Fig. 5.28 shows a sketch of St. Michael's as it appeared after the earthquake. It can be seen that the wrenching of the front portico into the steeple tower and the pattern of cracking in the side walls near the tower are consistent with a liquefaction-related ground failure beneath this portion of the church. Even today, the discerning viewer can detect a slight leaning of the tower to the northwest and the previous existence of cracks in the walls beneath the tower.

SPT's for a site directly across the street from St. Michael's are presented in Fig. 5.29. and are considered

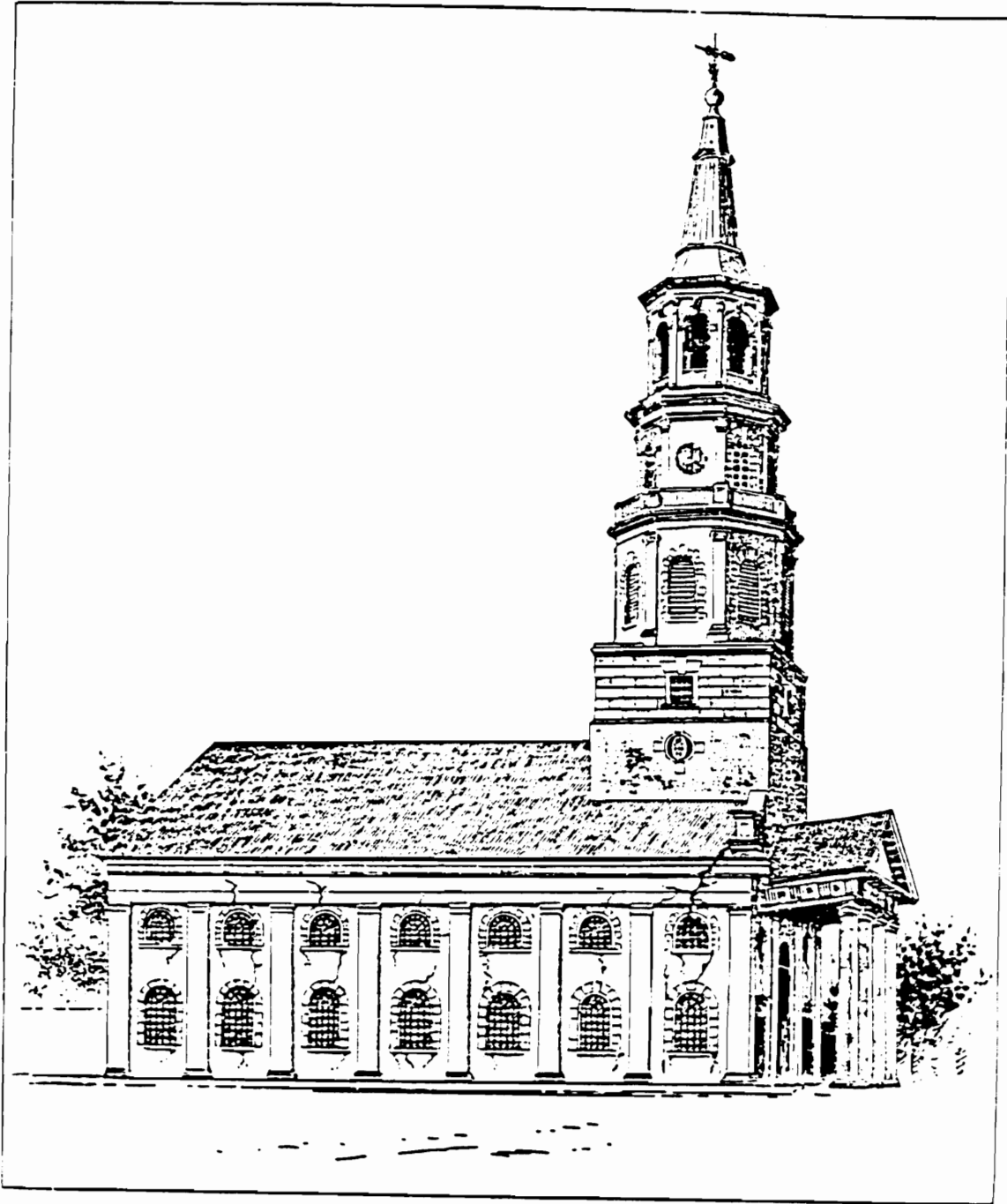


Figure 5.28 Sketch Showing Damage to Steeple Tower of St. Michael's Church (Dutton, 1889).

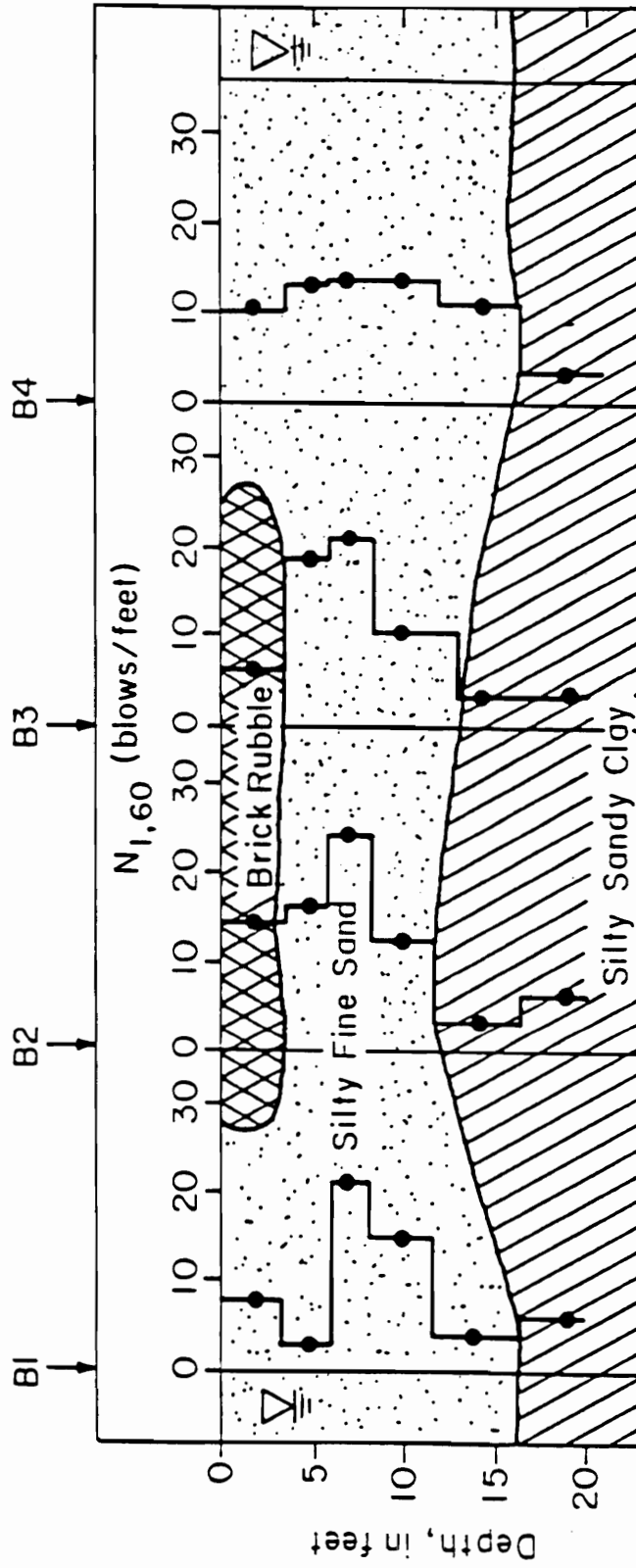


Figure 5.29 Soil Profile Showing SPT Data From Site Near St. Michael's Church in Charleston.

representative of soil conditions beneath the church. Except for the isolated presence of rubble, the upper 10 to 16 ft. of the soil profile consists of fine to medium sand in a loose to medium dense condition. Blowcounts in this layer average about 12 blows/ft. These upper sands are underlain by a 15 to 20 ft. layer of clay, and the water table is found at a depth of 4 ft. It is important to note that more damage did not occur to the church, which suggests that the level of ground accelerations necessary for the liquefaction of the upper sands was not greatly exceeded.

MANIGAULT RESIDENCE

In addition to the St. Michael's Church site, it is also instructive to cite the circumstances at the residence of Dr. G. E. Manigault, who was an official observer of the 1886 earthquake and one of the leading contributors to Dutton's report of 1889. The Manigault residence was located at the southern tip of the peninsula at the corner of what is now Gibbes and Legare Streets. Following the earthquake Dr. Manigault reported the following:

There was only one mud-spout or craterlet worth mentioning in the yard of a house opposite to my own, which I examined the next morning. The material thrown up was sand that had been slightly discolored by admixture with mud. The diameter of the mass that had overflowed was three feet, with the top of the craterlet about five inches high...There was no indica-

tion of water having escaped in any quantity from this craterlet...

An SPT boring was obtained for a site adjacent to the Manigault residence and is shown in Fig. 5.30. The soil profile in this area consists of 12 ft. of very loose silty fine to clayey sand underlain by soft clay to a depth of 63 ft. The water table is positioned at a depth of about 3 ft. With the exception of the top two feet of topsoil, normalized blowcounts in the sand layer remain below 3 blows/ft. and indicate a high susceptibility of the sands to liquefaction. The fact that only very minimal liquefaction occurred in this vicinity is important relative to the level of accelerations that occurred in this area during the 1886 event.

5.2.3 Mt. Pleasant Area Sites

The town of Mt. Pleasant is located on the east flank of Charleston harbor approximately 4 miles from the City and from the eastern boundary of Bollinger's meizoseismal zone (see map in Fig. 3.2). The town lies in an 85,000 yr.-old beach deposit characterized by clean fine sands in the upper 10 to 15 ft. of the soil profile. Local development has necessitated the mining of these sands for fill material, resulting in the excavation of numerous sand pits in the area. A sand mining operation consisting of three

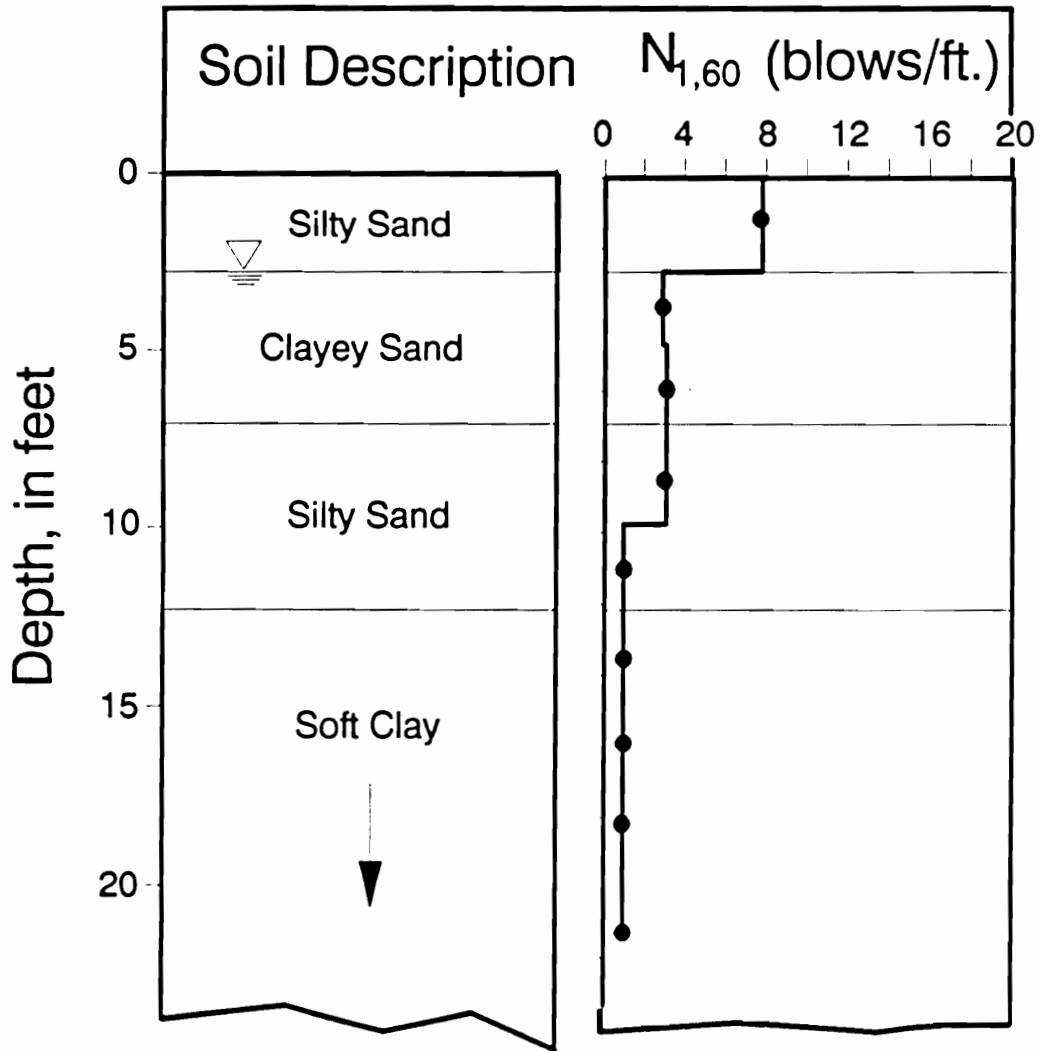


Figure 5.30 Soil Information for Site Adjacent to Manigault Residence in Downtown Charleston.

exceptionally large pits (referred to as the "Mt. Pleasant Pits" in this report) was examined by S. Obermeier for liquefaction features. Two of the pits were 11 acres in plan while a third was approximately 7 acres. All three pits were approximately 15 ft. in depth and were situated adjacent to each other as depicted in Fig. 5.31. Dewatering of the pits was maintained during mining operations which exposed the top 12 to 15 ft. of the soil profile. Obermeier searched the walls of the pits for evidence of previous liquefaction activity and found none. This suggests that ground motions from the 1886 earthquake were not sufficient to liquefy the sands in that area.

A total of 6 CPT's and 1 SPT were performed around the edges of the pits. The locations of the test borings are given in Fig. 5.31. In Fig. 5.32, a generalized soil profile which cuts across the entire site is shown along with CPT results. Soil conditions across the site are shown to be relatively consistent with the upper 20 to 25 ft. of the soil profile consisting of loose to medium fine sands. Underlying these sands is a 6-foot layer of soft clay which in turn overlies Cooper Marl at an average depth of 30 ft. Penetration resistances in the upper 10 ft. of the profile indicate the sand density to be loose to medium with an average Q_c of 150 bars and $N_{1,60}$ of 13 blows/ft. Extending from 10 ft. to the underlying clay layer at about 23 ft.,

MT. PLEASANT PITS SITE

Located on Charleston, S.C.
Quad. USGS 7.5 minute topo.

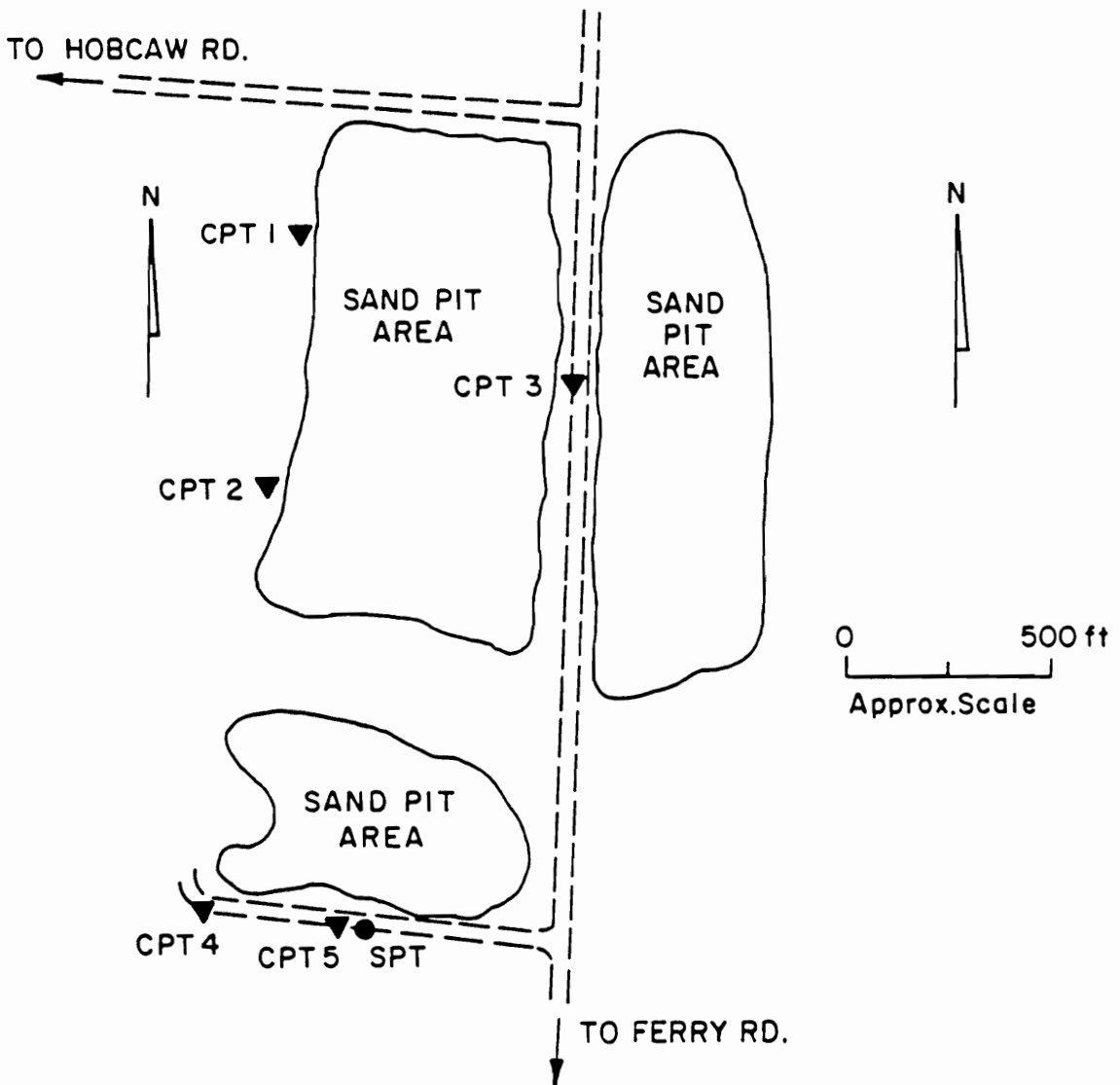
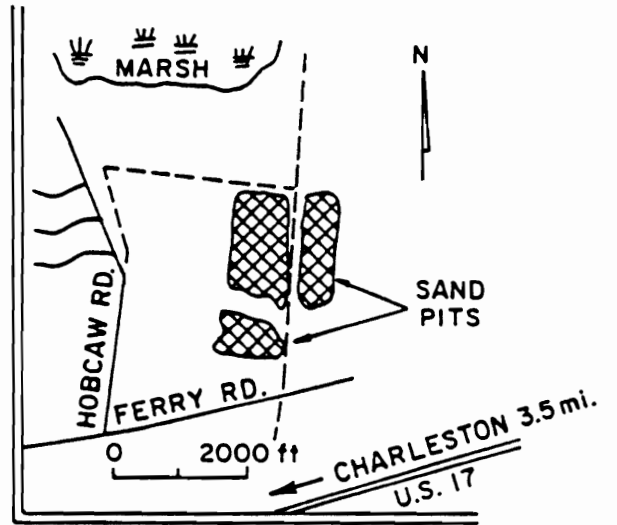


Figure 5.31 Location Map for the Mt. Pleasant Pits Site.

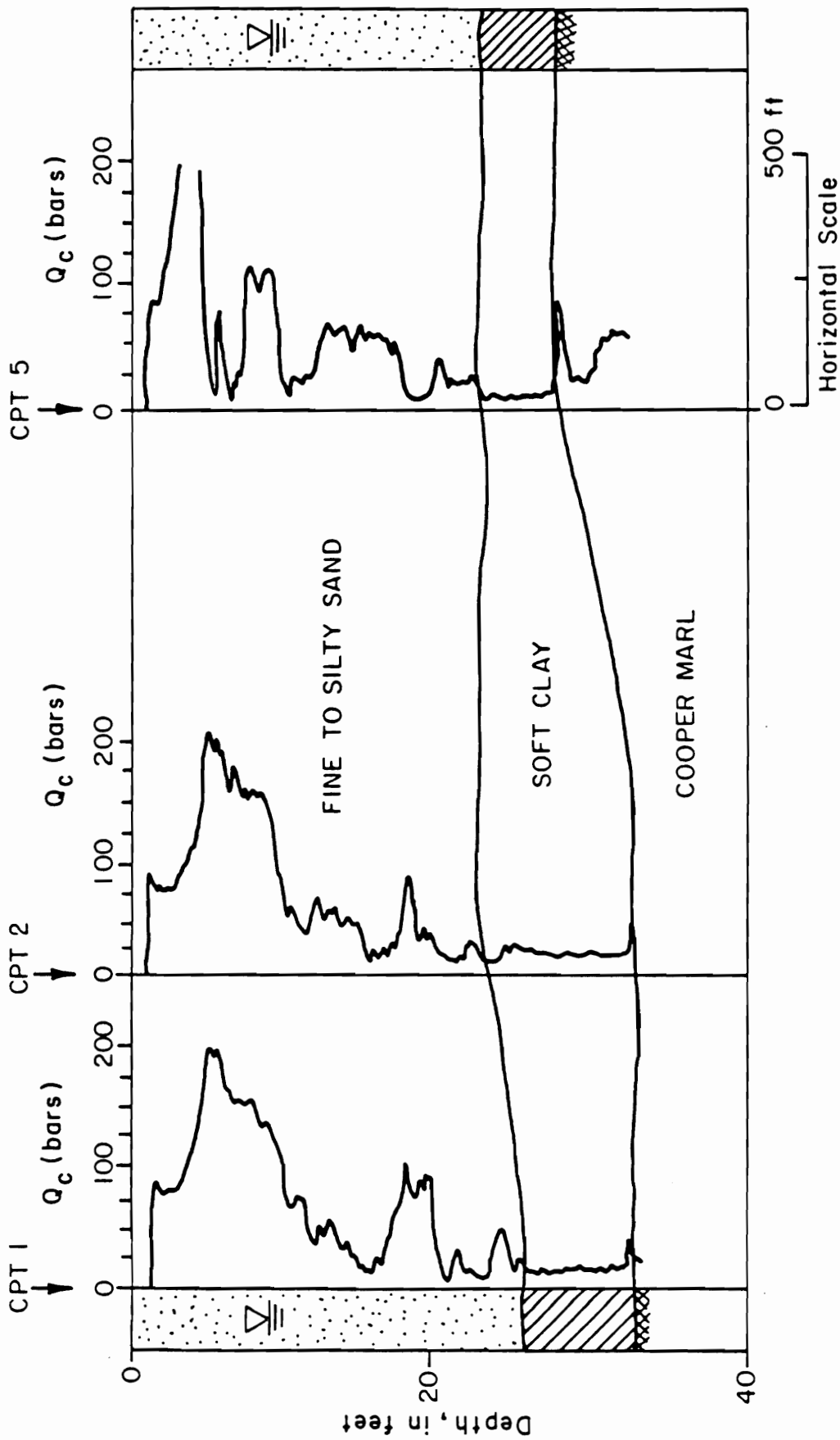


Figure 5.32 Soil Profile for the Mt. Pleasant Pits Site.

the sand densities remain consistently loose, with tip resistances ranging from 10 to 80 bars and blowcounts ranging from 3 to 8 blows/ft.

Mt. Pleasant Strip Mall and Mark Clark Bridge

To complement the Virginia Tech database in this important area, two additional Mt. Pleasant area sites were selected for investigation. These included a shopping mall site in Mt. Pleasant (the mall was under construction at the time of the present study), and a site along the cleared right-of-way for the future Isle of Palms Connector/Mark Clark Expressway Bridge. Both sites are situated adjacent to Highway 17 and lie in the 85,000 year-old beach deposit (Fig. 3.2). The shopping mall site is located approximately one mile south of the Mt. Pleasant Pits, while the bridge site is about five miles northeast of this area. The general site locations are shown on the map in Fig. 3.2.

The shopping mall site was investigated because of a small 1886 liquefaction vent that was found there in a trench by Obermeier. Obermeier, who was in the area during mall construction, searched several trenches and excavations at the site, but found only one 1886 feature. His

opinion was that only minor liquefaction occurred across the mall site which covered about 15 acres.

Four 20-ft. and two 10-ft. SPT borings were performed at the site by a local soils firm, and these data were obtained for the present study. No additional SPT or CPT testing was performed by Virginia Tech. Fig. 5.33 shows a soil profile for the site and includes results from four of the SPT's performed there. The sands are shown to be loose to medium in the upper 9 ft. of the profile, with normalized blowcounts ranging from 5 to 18 blows/ft. Underlying the upper sands is a medium to dense sand stratum which extends from 9 to 20 ft. as indicated by three of the SPT borings. Blowcounts in this layer averaged about 21 blows/ft., corresponding to an estimated relative density of about 70%. A fourth test shows consistently loose to medium sands extending from the ground surface to the bottom of the hole. Blowcounts at this hole remained less than 10 blows/ft. throughout the entire profile. The water table was found at 4 ft. below the ground surface.

The area cleared for the Isle of Palms Connector and Mark Clark Bridge cut across the 85,000 year-old beach deposit and extended for several miles from U. S. Highway 17 near Oakland Plantation, eastward to the Isle of Palms. The width of the clearing was approximately 150 ft. wide. At the time of the present study, two SPT borings were being

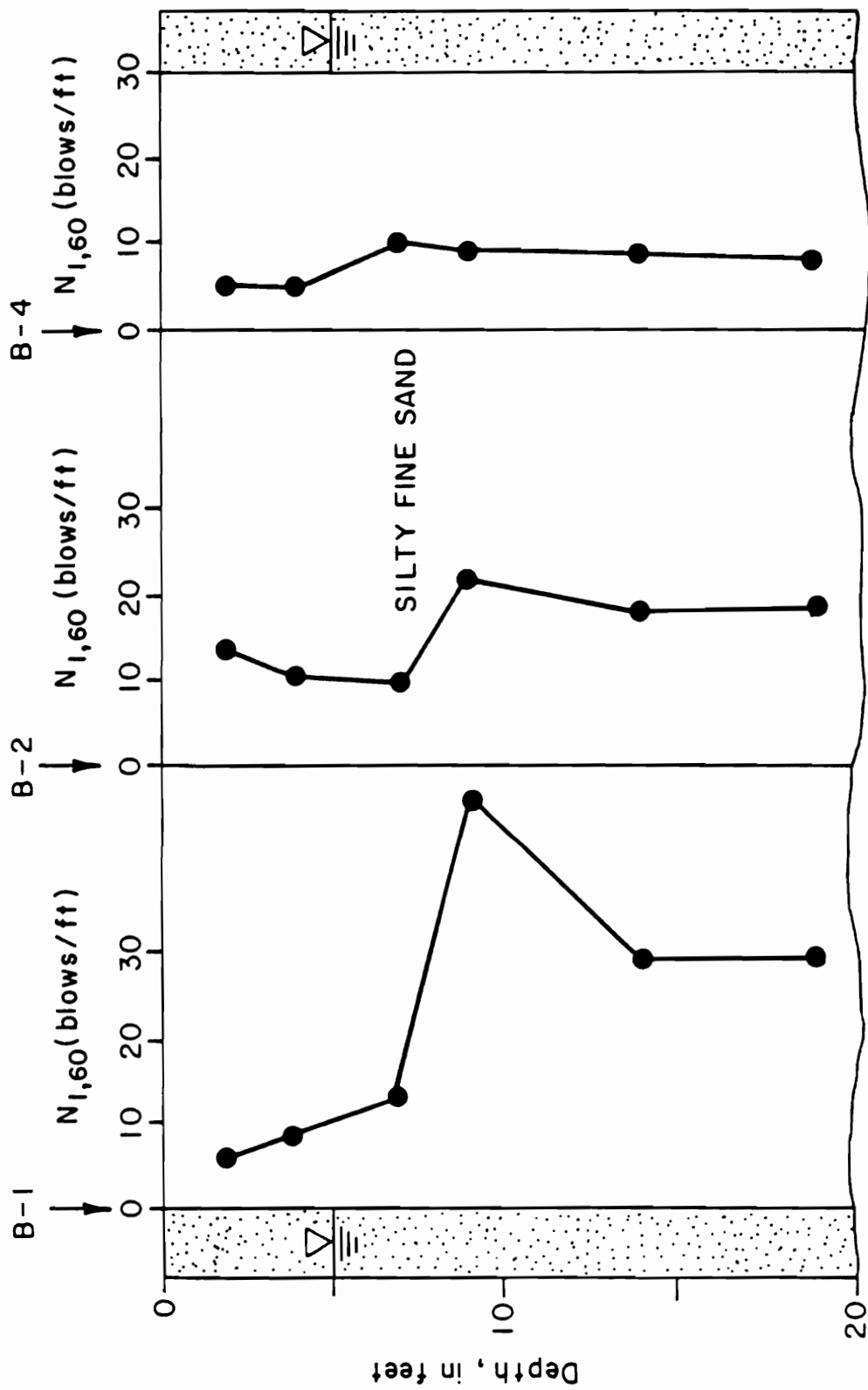


Figure 5.33 Soil Profile Showing SPT Data From the Mt. Pleasant Shopping Mall site. NOT TO HORIZONTAL SCALE

performed along a portion of the project alignment by a local soils firm. These borings were subsequently made available to Virginia Tech.

Four CPT's were performed by Virginia Tech along a three-quarter mile section of the site as shown in Fig. 5.34. Three of the CPT's were located close to SPT holes, and were used for CPT-SPT calibration purposes. The borings indicated loose to medium silty fine sands in the upper portions of the soil profile and medium to dense silty sands at depth. The sediment thickness overlying the Cooper Marl ranged from 38 to 44 ft. The position of the water table was found at a depth of 3 ft. at the time of SPT testing.

As indicated in Fig. 5.34, CPT's 1 and 2 were located close to SPT 1 (at STA 26+40), although the CPT's and the SPT revealed different density patterns in the upper 30 ft. of the soil profile. SPT 1 (see Fig. 5.35) indicated the presence of very loose to loose sands from the ground surface to a depth of more than 30 ft. The CPT's in this area also indicated loose to medium sands in the top 7.5 ft., but except for a 2-ft. layer of very loose sand at 16 ft., revealed the sands below 7.5 ft. to be consistently dense, with Q_c values remaining higher than 200 bars. The reason for these differences is not obvious, but is attributed to an abrupt change in soil conditions. Underlying the upper sands, the SPT revealed a medium sand

**MARK CLARK EXPRESSWAY BRIDGE*
SITE**

Located on Charleston, S.C. Quad.
USGS 7.5 minute topo.

*(Isle of Palms - to - Charleston
connector)

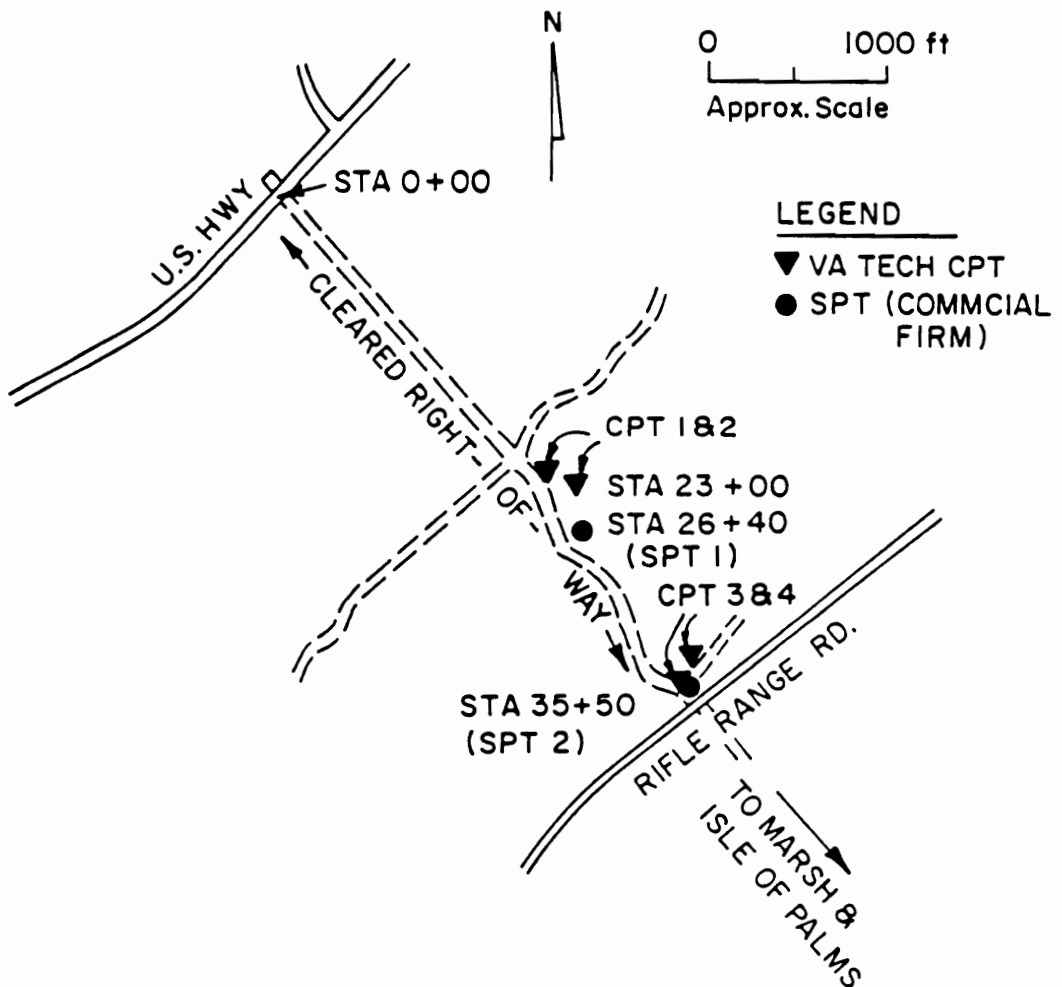
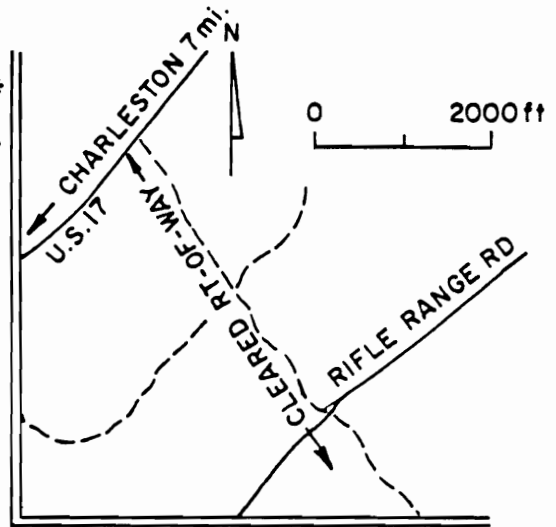


Figure 5.34 Location Map for the Mark Clark Bridge/Isle of Palms Connector Test Site.

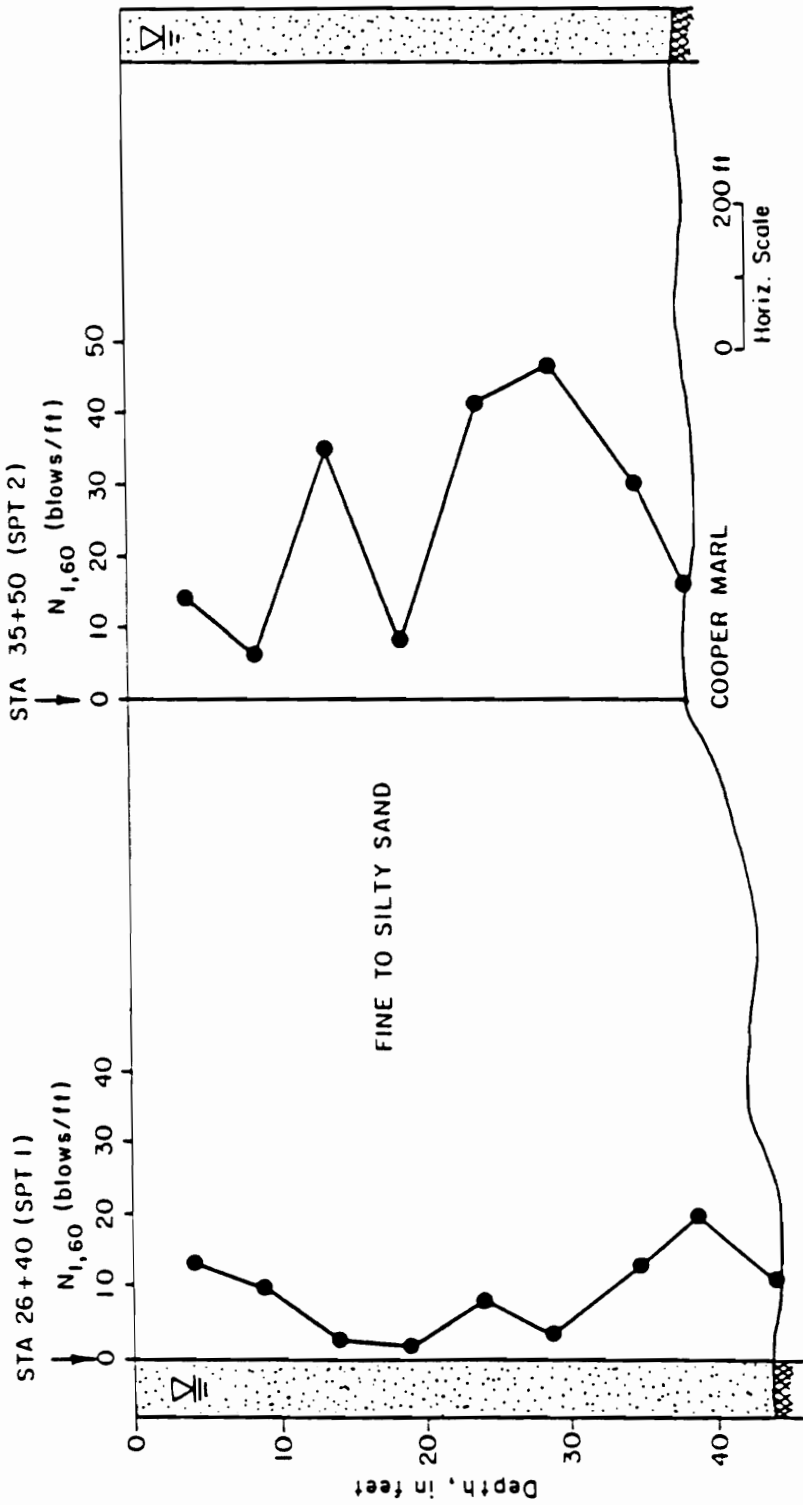


Figure 5.35 Soil Profile Showing SPT Data from Mark Clark Bridge/Isle of Palms Connector Test Site.

layer at 33 ft. which extended to Cooper Marl at 44 ft. The two CPT's and SPT near station 35+50 showed similar density patterns, indicating the sand density to be loose to medium in the upper 10 ft; medium to dense from 10 to 18 ft.; loose from 18 to 21 ft.; and dense from 21 to a depth of 38 ft. where Cooper Marl was encountered.

5.2.4 South Tibwin and St. James-Santee School Sites

The town of South Tibwin is located along U.S. Highway 17 about half-way between Charleston and Georgetown and falls approximately 35 miles from the 1886 epicentral area. Two sites in the South Tibwin area were selected for investigation. The primary test location was a several hundred-acre tract of timberland owned by the International Paper Company. The site was located by S. Obermeier who discovered a drainage ditch which extended for some several thousand feet along the main entrance road into the property. The ditch was similar in size and depth to those found at Hollywood and offered good exposure for signs of 1886 liquefaction activity. Much like the Mt. Pleasant area sites, no evidence for liquefaction was found over any lengths of the ditch and this suggests that ground motions were insufficient to induce liquefaction.

Soil testing at this site consisted of three CPT's and one SPT along the ditch (Fig. 5.36). The SPT along with the

SOUTH TIBWIN SITE
 Located on Awendaw, S.C.
 Quad. USGS 7.5 minute topo.

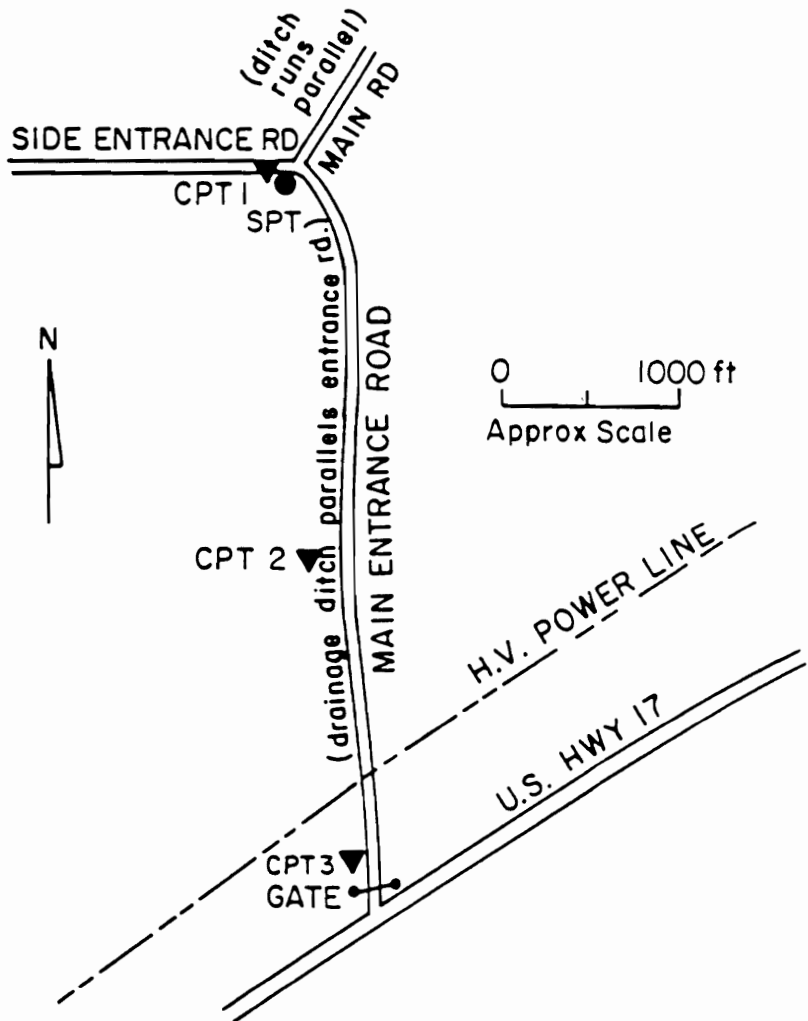
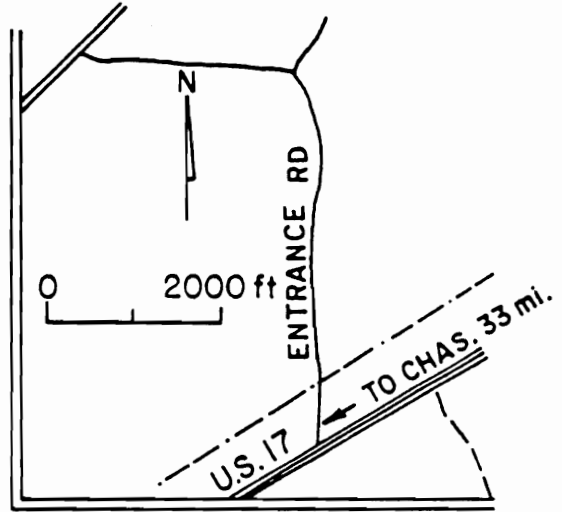


Figure 5.36 Location Map for the South Tibwin Site.

CPT's shown in Fig. 5.37, revealed fine sands of medium density from the ground surface to a depth of about 12 ft., and loose sands from about 12 to 17 ft. Below 17 ft. a soft clay layer is encountered which extends to Cooper Marl at 36 ft. The water table was at a depth of 5 ft. at the time of SPT testing.

ST. JAMES-SANTEE SCHOOL

Saint James-Santee School, a newly constructed building, was located adjacent to the South Tibwin tract approximately 400 yards from the drainage ditch. Nine SPT borings done at the site were obtained from a local soils firms. In addition to these data, two CPT's were performed and several Shelby tube samples of clay and marl were obtained at the site by Virginia Tech personnel. The CPT's were performed adjacent to previous SPT holes (see Fig. 5.38). The penetration resistances indicate soil densities similar to those encountered at the drainage ditch. Q_c values in the upper sands average more than 200 bars from the ground surface down to 10 ft., while normalized blow-counts in this layer average 15 blows/ft (see Fig. 5.39). Below 10 ft., a 2.5 ft. loose sand layer is encountered, and soft clay is found from 12.5 to 23 ft. A second stratum of

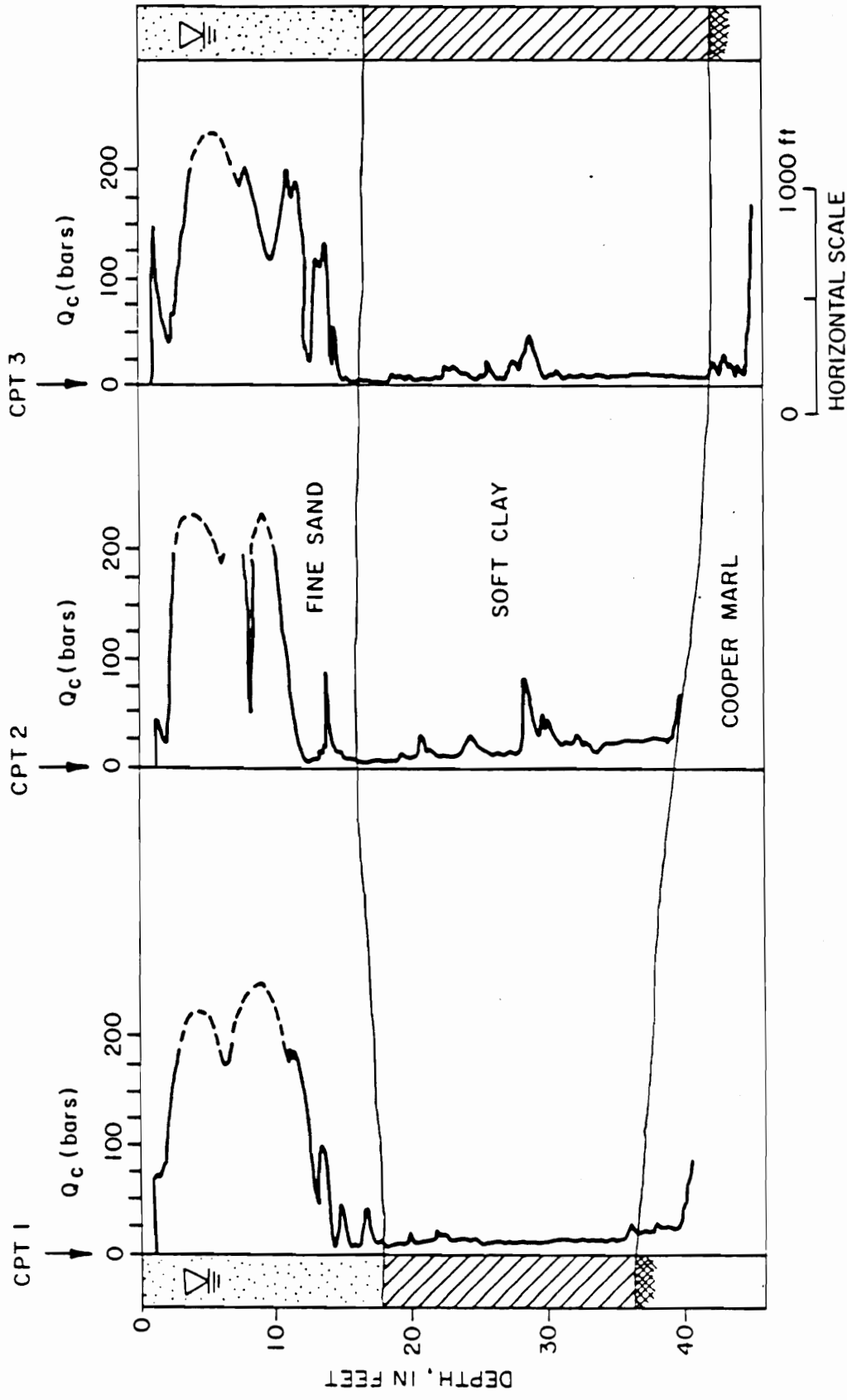


Figure 5.37 Soil Profiles for the South Tibwin Site.

ST. JAMES - SANTEE SCHOOL SITE

Located on Anendaw, S.C.
Quad. USGS. 7.5 minute Topo.

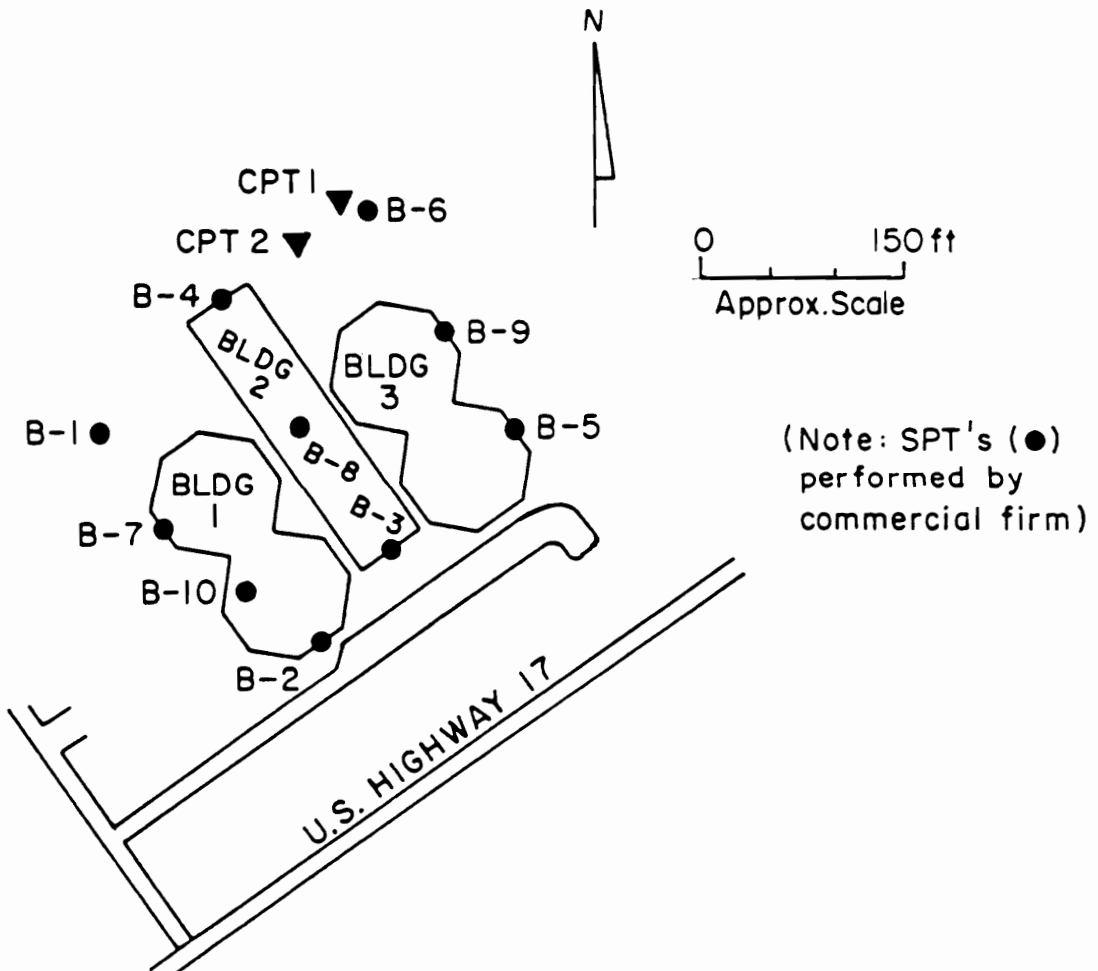
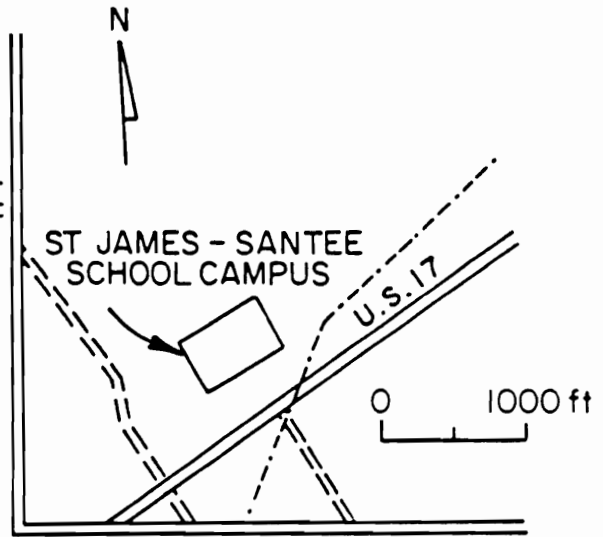


Figure 8.38 Location Map for the St. James-Santee School Site.

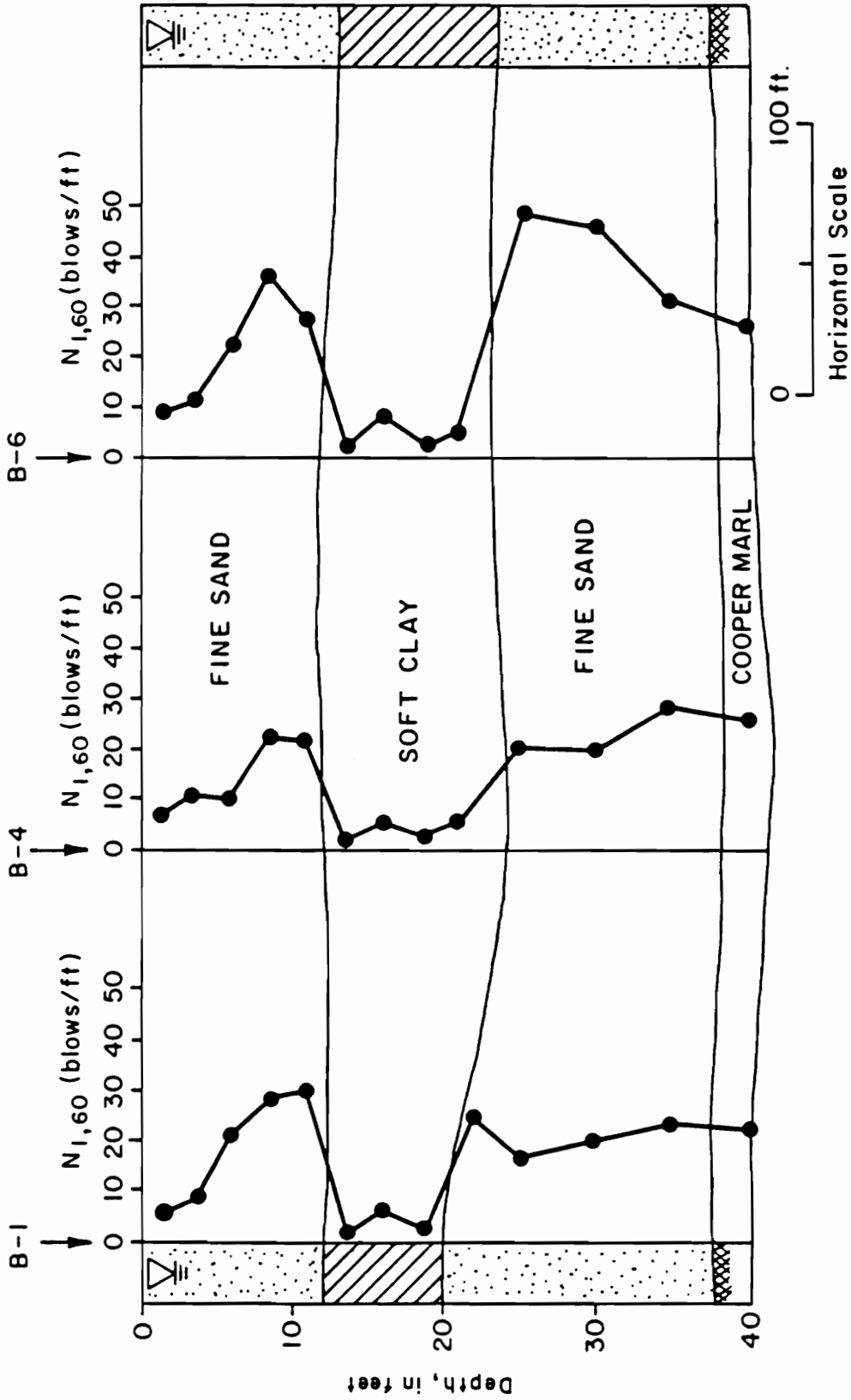


Figure 5.39 Soil Profile for the St. James-Santee School Site.

medium to dense sand extends from 23 to Cooper Marl at 36 ft. The water table at this site was found at 2 ft. below the ground surface.

5.2.5 Georgetown Area Sites

The City of Georgetown, SC lies along U.S. Highway 17 about 55 miles northeast of Charleston and the epicentral area. As illustrated in Fig. 5.40, Georgetown is a peninsular city flanked by the Pee Dee and Sampit Rivers and fronted by Winyah Bay. The geologic environment is composed of a mixture of marine and fluvial deposits, although most of the downtown area consists exclusively of beach sediments close to 200,000 yrs. old. Area soil conditions are variable, and consist of medium to coarse sands with interbedded clays, clayey sands, sandy clays, clays, and clean fine sands. The entire area is underlain by Santee Limestone, and the water table is found within two to three feet of the ground surface.

The Georgetown area is of interest because it arguably represents the farthest site from Charleston at which liquefaction phenomena was reported by 1886 observers. Investigations were performed in the Georgetown area because of newspaper accounts that appeared in the Georgetown Enquirer a few days following the 1886 event. The newspaper reported that "three marked depressions had been observed in

KING and PRINCE SITE
 Located on Georgetown, S.C.
 USGS Quad. 7.5 minute topo.

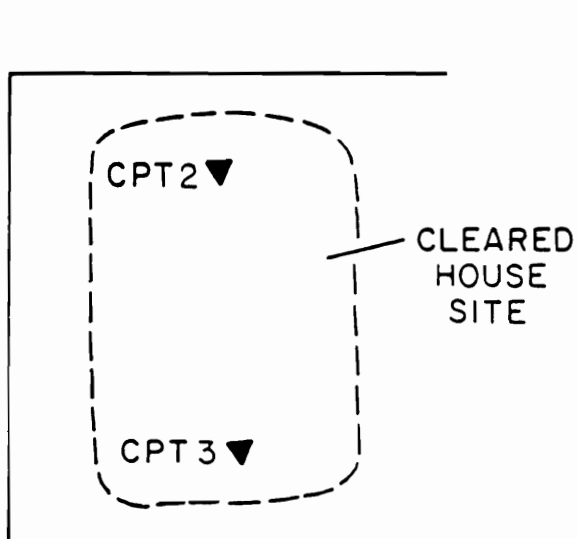
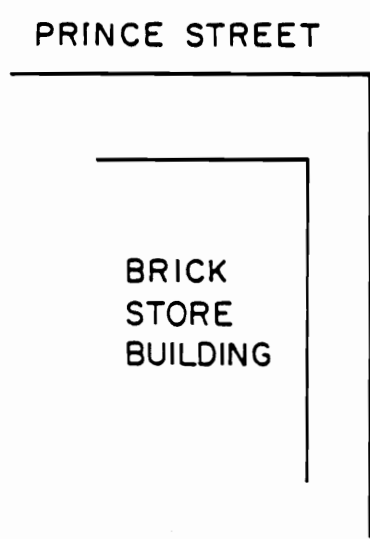
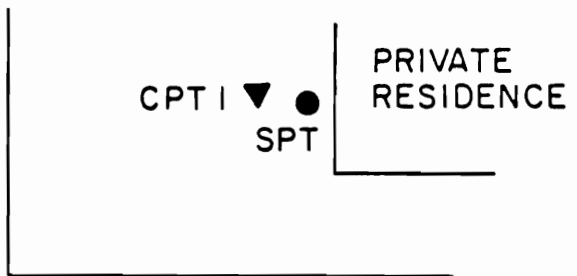
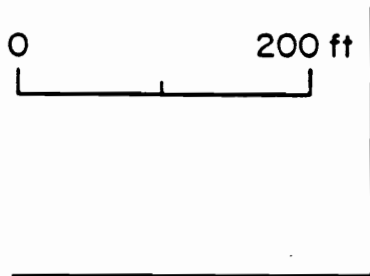
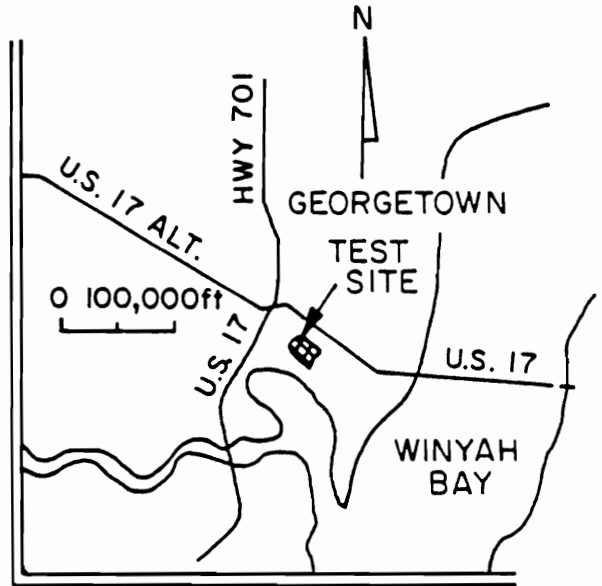
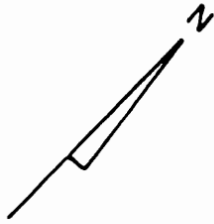


Figure 5.40 Georgetown Site Map.

different locations of the town...". The newspaper's description of two of these features is given below:

- 1) At the corner of King and Prince Streets near the residence of Mr. S. Rembert was seen a small circular spot, about two feet in diameter, which presented a disturbed appearance, as though someone had been grubbing with a hoe and had turned up a lot of fresh earth. There seemed to be nothing peculiar about the sand, except that it was slightly darker than that in the street.

- 2) A similar spot about six feet in diameter was observed on the Sampit Road. It looked as though someone had thrown fresh earth into an excavation to fill it up. The top was on a level with the ground. It was reported that this spot had contained water on Wednesday morning.

Although these accounts describe what appear to be sand craters, the physical evidence for the Georgetown area indicates otherwise: 1) No reports of liquefaction in the Georgetown area appear in Dutton's Report of 1889; 2) there is an absence of any other 1886 features at locations closer to the epicenter; especially along the 35-mile reach from Oakland Plantation to Georgetown; 3) U.S. Geological Survey workers report that no 1886 features were unearthed in the Georgetown area; and, 4) it must be considered that the accounts above were not given by "official" observers; they generally described such features in greater detail and related their observations to a particular soil formation or geologic setting, etc. It is postulated that the

observations do not describe liquefaction craters, but instead describe areas of localized ground subsidence caused by the densification of near-surface loose sand pockets. With the shallow water table, minor ground subsidence would, in many cases, result in the submergence of the ground surface and the formation of muddy pools of water similar to those described above. The outward appearance of these features would be much the same as that of the "explosion" sand craters observed within the meizoseismal zone. Assuming that this explanation is valid, the physical evidence would appear to indicate that Georgetown experienced no liquefaction, or at best, extremely minor liquefaction during the 1886 event, and that the upper level of ground shaking in the area was not sufficient to effect the onset of liquefaction.

The penetration tests performed in the Georgetown area seemed to support the idea that the liquefaction potential of sediments there (to the 1886 event) was minimal. Three CPT's and one SPT were performed in the yards of two private residences at the corner of King and Prince Streets (location of one feature given in newspaper accounts above). As indicated in Fig. 5.40, the tests were performed on opposite sides of the street corner to characterize soil conditions over a broader area. Fig. 5.41 shows a soil profile developed from the CPT and SPT test holes. The

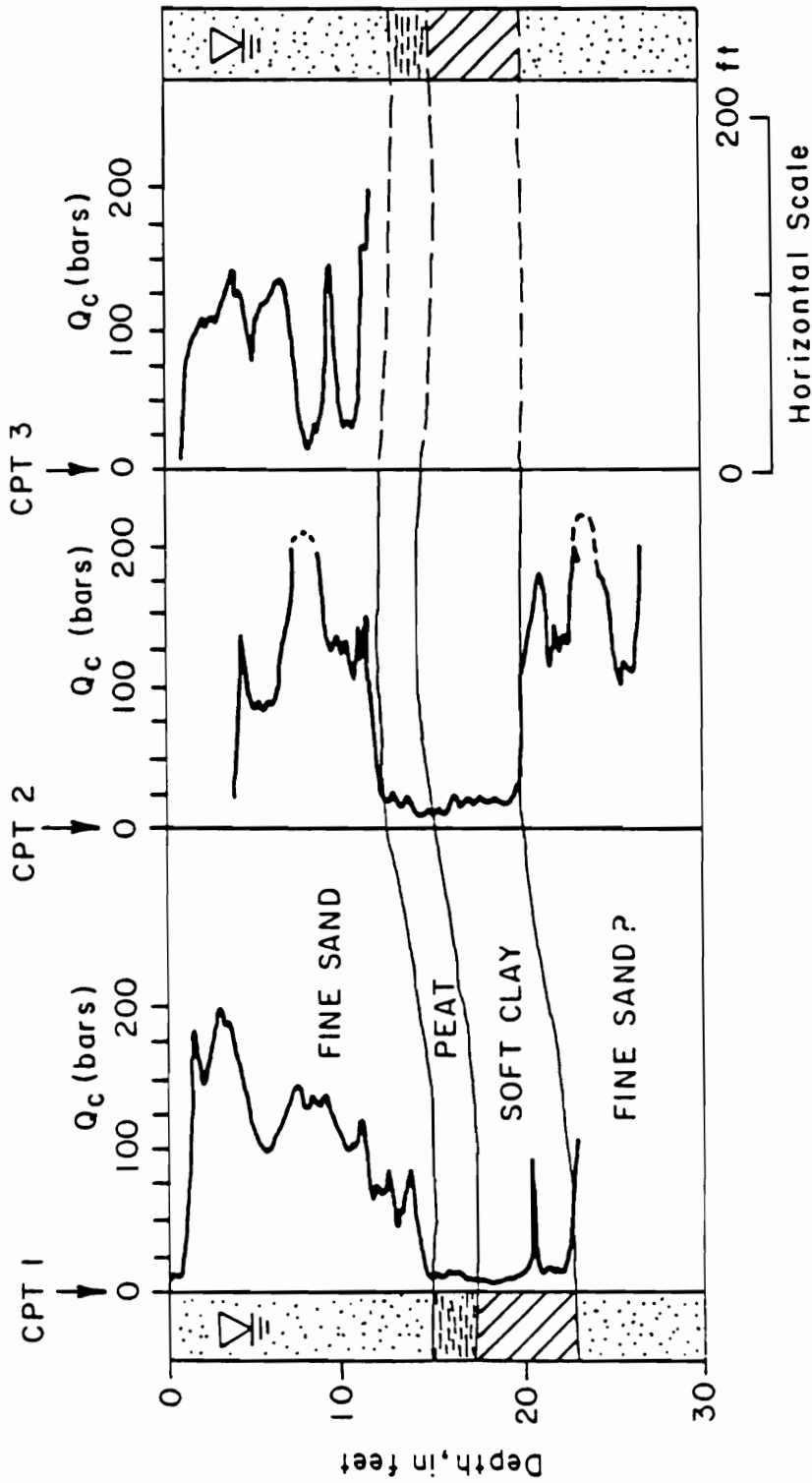


Figure 5.41 Soil Profile for the Georgetown Site (King and Prince Streets).

upper 5 ft. of the profile is shown to consist of dense fine sands with a trace of clay. From 5 ft. to 15 ft., the sands are shown to be fine and medium in density. Below 15 ft. a 2-ft. layer of peat is encountered, and from 17 ft. to 24 ft. a soft clay layer is found. No SPT sampling or CPT testing was done in the depth range from 24 to 36 ft., although the SPT was augured down to firm material at 36 ft. The cuttings from the auger boring indicated that medium to dense sands were present from 24 to 36 ft., and that hard marl-like material was present at 36 ft. The water table was located at about 5 ft. below the ground surface 24 hrs. after SPT testing.

Parkersville/Pawley's Island Site

Parkersville, SC is located ten miles north of Georgetown along U.S. Highway 17 near Pawley's Island, SC. The town lies approximately 65 miles northeast of Charleston and 55 miles from the northeastern limit of Bollinger's estimated meizo-seismal zone. This area is of significance because of the first-hand accounts that appeared in the Georgetown Enquirer following the 1886 earthquake:

Near the residence of Senator B. H. Williams on the west side of the street was a sink of at least one foot in depth and perfectly circular in shape. Its diameter

was 8 ft. Three or four heaps of gray sand stood at irregular intervals on the edges of the sink. They were probably thrown up through the fissure by the internal convulsion which preceded the subsidence of the ground.

As with the other Georgetown area sites, the interpretation of this account is not clear-cut, although the site warranted an investigation by Virginia Tech personnel. The location of the former Williams residence, a large plantation, was obtained from an 1886 tax map at the Georgetown Rice Museum. It was learned that the property was located in the Parkersville community along the old Georgetown Highway (present-day U.S. Highway 17). The exact whereabouts of the house and the reported feature is uncertain, but was located to within one-quarter of a mile. The geologic setting of this location is somewhat different from that at Georgetown, consisting entirely of beach sediments dating to approximately 200,000 yrs. (see Fig. 3.2).

A total of 4 CPT's were conducted to characterize the nature and variability of soils in the vicinity of the plantation (see map in Fig. 5.42). Test holes ranged from 12 to 45 ft. in depth, as dictated by the density of the sands and the capacity of the rig. No SPT borings were performed at the site; friction ratio measurements from the CPT's were used to identify soil types. The water table was found at a depth of 4 ft. during testing operations. The results of the CPT's are shown individually in Appendix B.

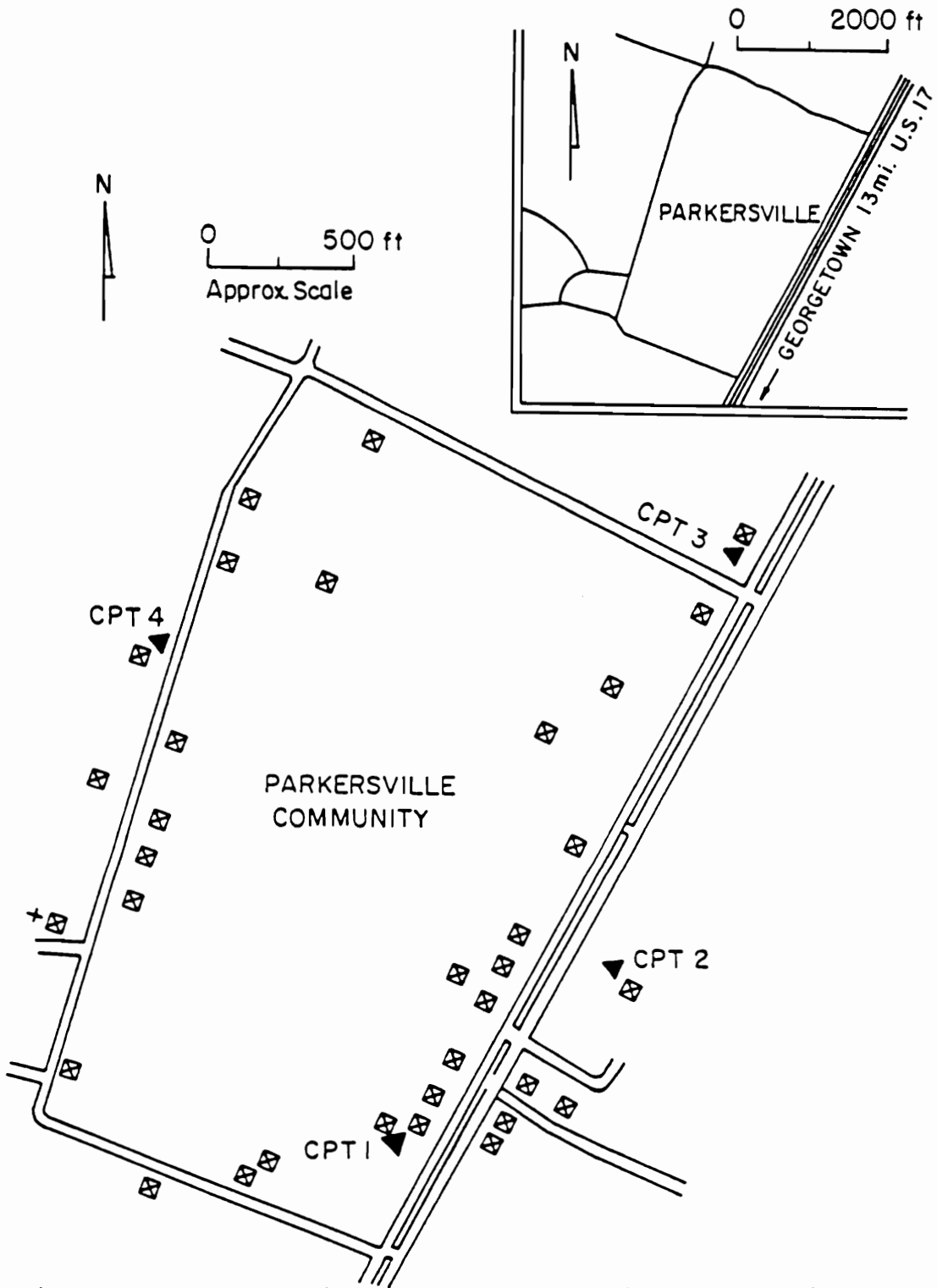


Figure 5.42 Location Map for CPT's in Parkersville (Near Former Williams Plantation).

Three of the CPT's indicated that medium to dense sands dominate the soil profile to depths of 30 ft. or more, although a fourth CPT revealed the presence of loose to medium sands over this depth range.

5.2.6 Edisto Island Site

Edisto Island is located approximately 25 miles southwest of Charleston and about 19 miles south of Dutton's southwestern epicentrum. Liquefaction features at this site were described as a "bounding line of craterlets coursing to S [south] passing through the island about 2 miles East of South Edisto River." The area referred to is near Edingville Beach on Edisto Island. At the time of the present investigation, several hundred acres of the Edingville beach community were under development which afforded access for penetration testing over a broad area.

Seven CPT's and 1 SPT were performed at Seaside Plantation, the site of a future housing development on Edingville Beach. The tests were performed at various sites across the plantation which covered about 300 acres. The location of the penetration tests are shown in Fig. 5.43. The test data showed relatively consistent density patterns among the test holes. The penetration data illustrated in Fig. 5.44 shows the profile to consist of medium sands in the top 10 ft. of the soil profile; loose to very loose

EDISTO ISLAND SITE
 Located on Edisto Island, S.C.,
 USGS Quad 7.5 minute topo.

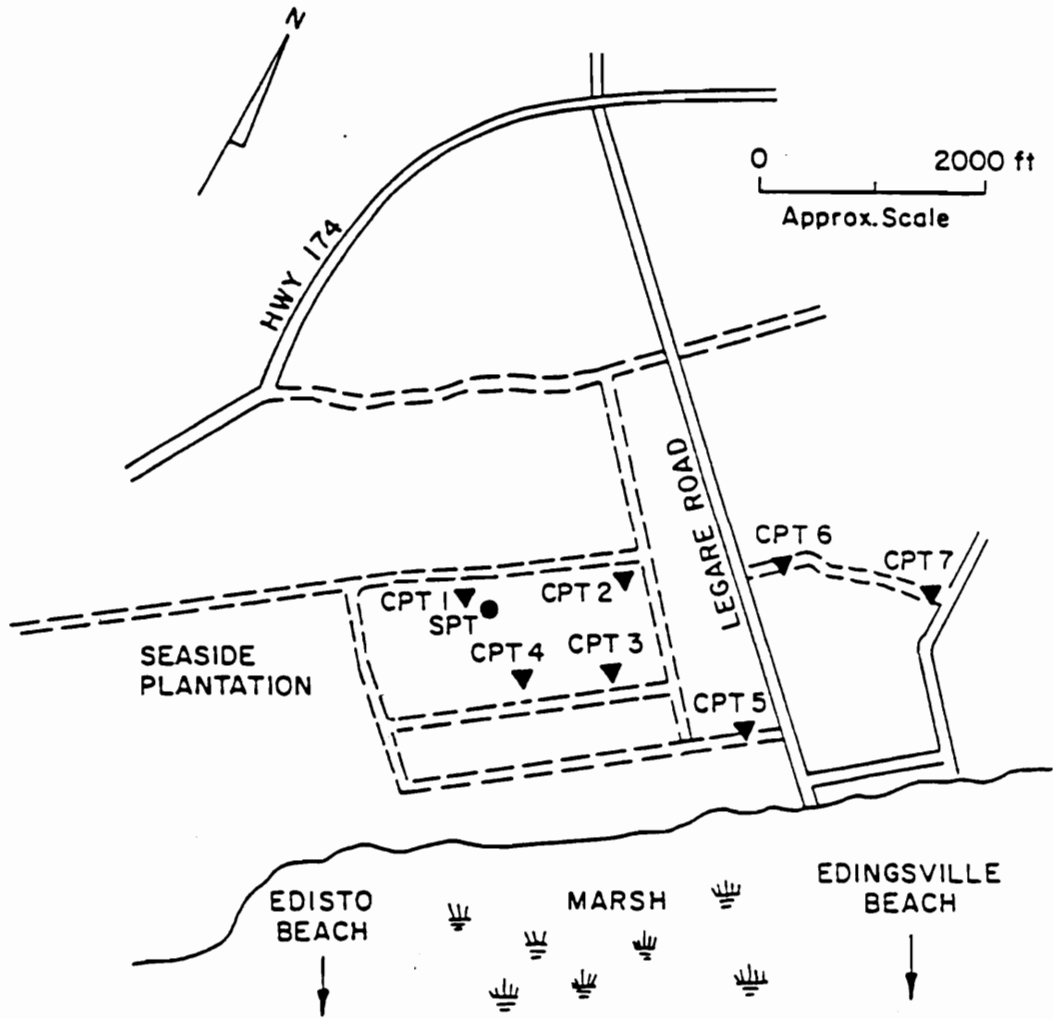
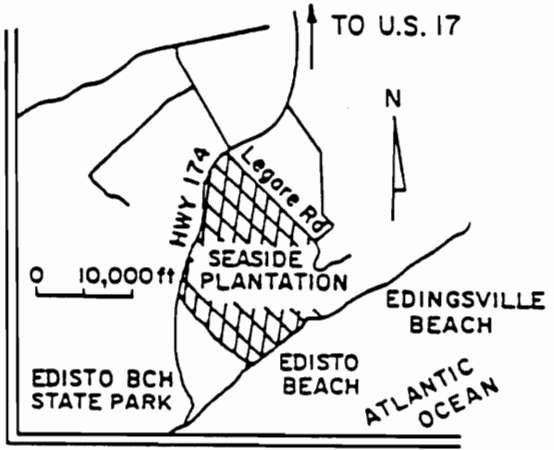


Figure 5.43 Location Map for Edisto Island Site.

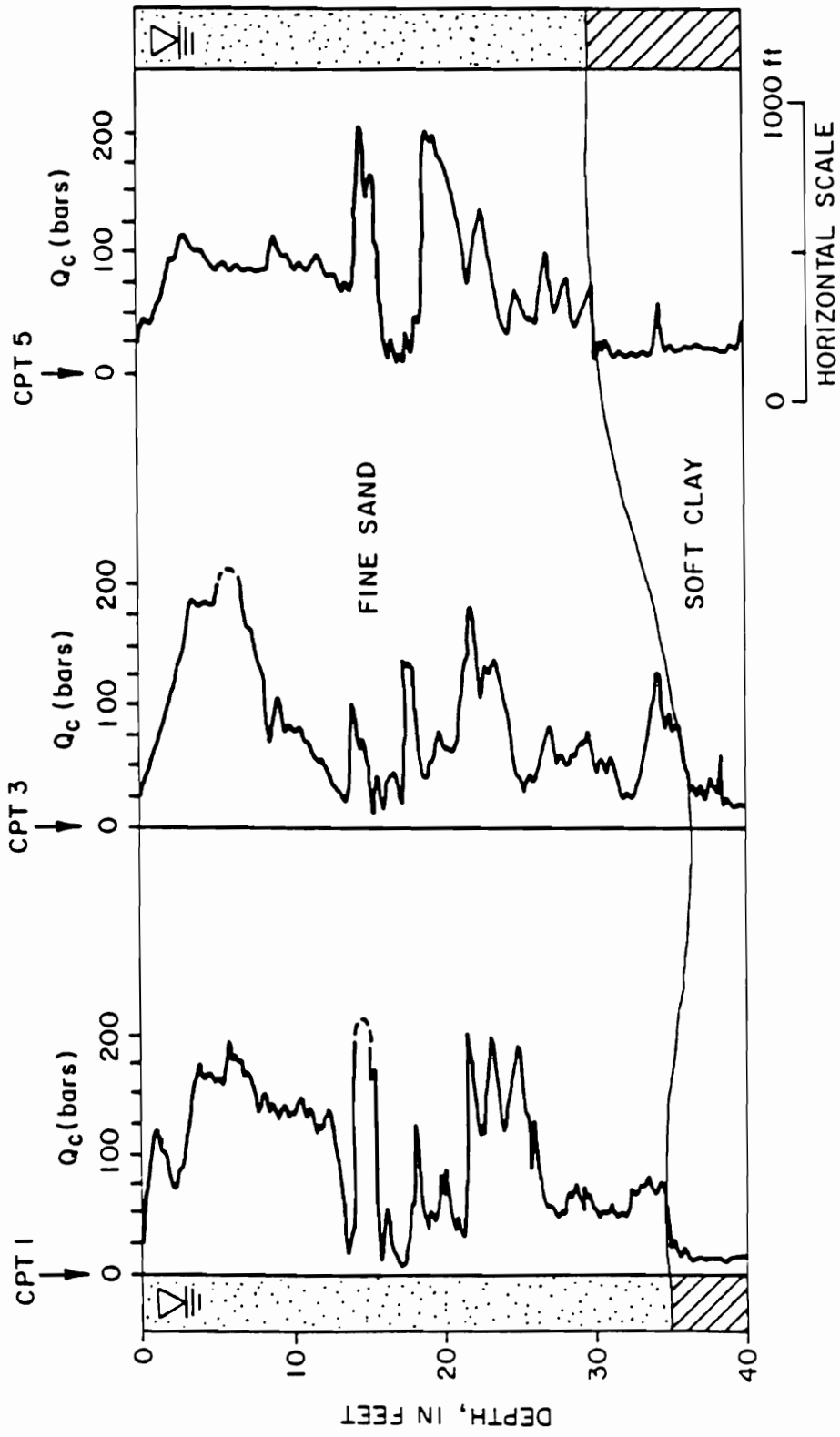


Figure 5.44 Soil Profile for the Edisto Island Site.

sands from 10 to 20 ft.; loose to medium sands from 20 to 24 ft.; loose to very loose sands from 24 to 36 ft.; soft clay from 36 to 42 ft.; and medium sand from 42 to 48 ft. Cooper Marl underlies the entire site at an average depth of about 48 ft., and the water table is located at a depth of about 3 ft. The presence of the loose sands from 10 to 20 ft. affords a layer susceptible to liquefaction.

INFORMATION OBTAINED FROM OUTSIDE SOURCES

SPT and CPT data were obtained from outside agencies for sites outside of the meizoseismal zone. These data are used to supplement the Virginia Tech data base in areas where detailed field studies were not performed. The majority of the data obtained was located along U.S. Highway 17 in the intermediate 85,000 year-old beach deposit and along the present beach areas. Data were also obtained for the City of Charleston and sites in the Georgetown, Myrtle Beach, and Savannah, GA areas.

More than 1500 borings were collected for sites outside the meizoseismal zone. Because the data are voluminous, and because the data are consistent in terms of their implications to this study, only a portion of these data are

utilized in the Chapter 6 analyses; these data are listed in Table 5.2. Collectively, the overall indications were that:

- 1) Many areas have loose sands that are obviously liquefiable at low levels of seismic shaking.
- 2) Soil conditions throughout the intermediate and present beach ridges appear more susceptible to liquefaction than the older beach ridges.
- 3) Soil conditions in the younger deposits are more uniform, and consist of more continuous sand strata with fewer interbedded non-liquefiable materials, than in the older beach ridges.
- 4) The thickness of the non-liquefiable overburden is less in the younger beach deposits than in the older units.
- 5) The water table is positioned closer to the ground surface in the younger beach deposits.

5.3 DISCUSSION OF SOIL CONDITIONS

DISCUSSION OF FIELD TESTING RESULTS

The geotechnical investigations presented thus far were performed to evaluate variations in soil conditions which

Table 5.2 Summary of Soil Information Obtained From Outside Agencies for Sites Outside the 1886 Meizoseismal Zone

Site	Distance From Energy Release (mi.)	Extent of Liquefaction 1886	Geologic Environment Age Classification	Number of SPT's	Avg N _{1,60} (Blows/ft.)	Avg Thickness		USCS Soil Classification
						Liquefiable Layer (ft.)	Capping Layer (ft.)	
John's Isl.	10	Minor	Beach/Young	16	14	14	5	SP-SH
James Isl.	13	Minor	Beach/Young	11	9	16	3	SP-SH
Kiawah & Seabrook Isl.	14	None	Beach/Recent	14	12	21	3	SP-SH
Edisto Isl.	18	Very Minor	Beach/Young	29	18	19	5	SP-SH
Isle of Palms	20	None	Beach/Recent	3	26	13	5	SH
Cape Romain	26	None	Beach/Young	8	19	10	4	SP-SH
St. Helena Isl.	38	None	Beach/Young	16	24	17	3	SH
Beaufort	40	None	Beach/Young	31	17	15	4	SP-SH
Hilton Head	53	None	Beach/Young	49	27	21	3	SH
Bluffton	55	Minor (5)	Beach/Young	14	42	17	3	SP-SH
Debidue	61	None	Beach/Old	16	24	17	4	SP-SH

(1) Based on Historical Observations in General Vicinity of Site.

(2) Age Classification: 1) Very Old = > 400,000 yrs; 2) Old = ~130,000 to 230,000 yrs; 3) Young = ~85,000 yrs; 4) Recent

(3) Includes Correction for Fine Content.

(4) Based on Visual Classification.

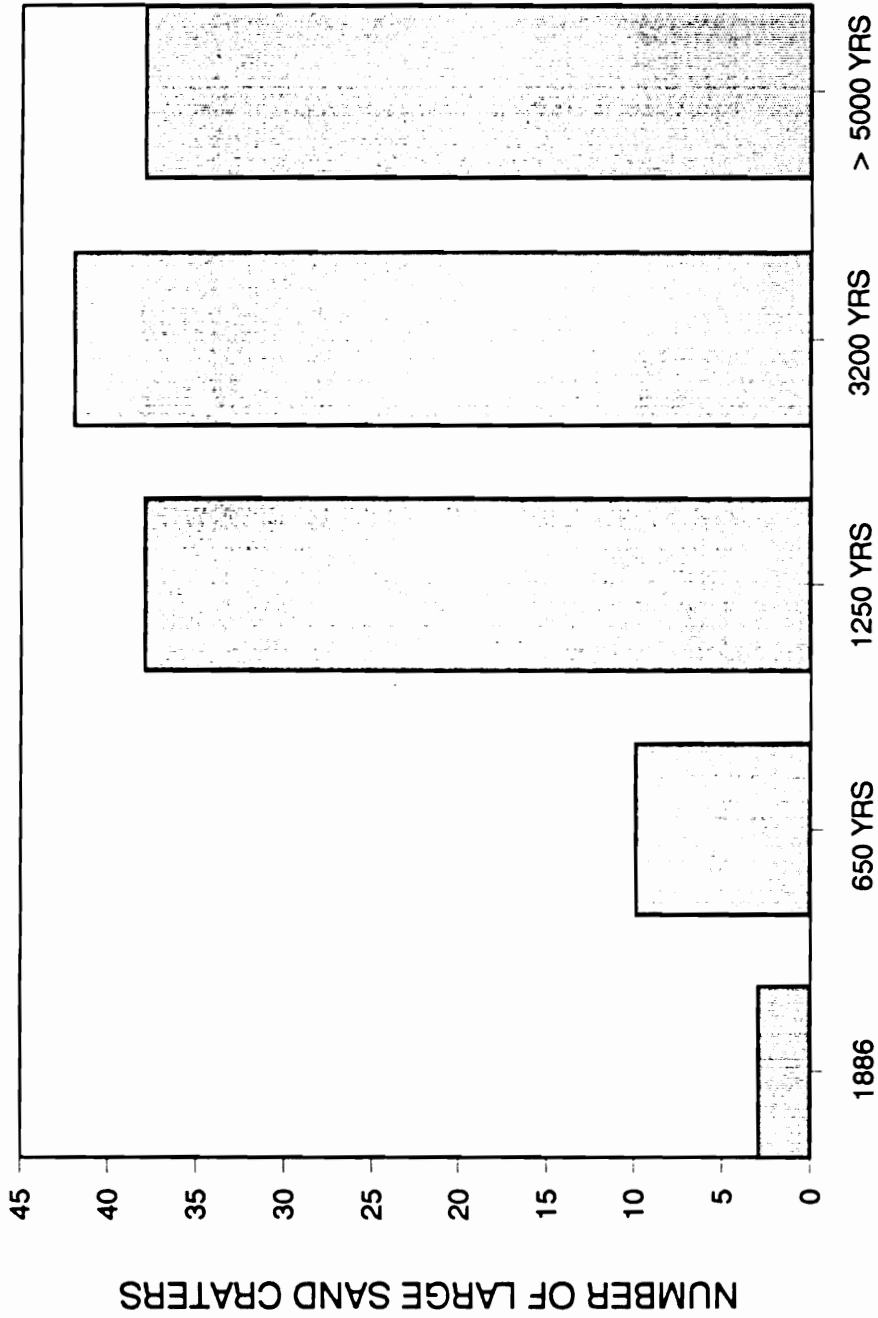
(5) Amick et al. in NUREG/CR-5613, 1990.

may suggest that the initiation and surficial manifestation of liquefaction would be different from site to site. The investigations indicated that soil conditions at many of the sites appear to be consistent with the liquefaction activity documented in historical references and evidenced in paleoseismic studies. The results also reveal findings which are unique, and circumstances that are not clear-cut.

The investigations showed a consistent pattern for the sites within the meizo-seismal zone where soil layers near the top of the profile were dense, and those near the bottom looser and subject to liquefaction. Soil profiles at sites outside of this region were generally looser and more consistent in density with depth. For sites within the meizo-seismal zone, the dense-over-loose pattern was most pronounced at Hollywood, and is partially attributed to progressive liquefaction and densification of the upper sands. It is expected that with time, the lower sands will also be densified, although the densification process is likely to be slow since the low-permeability dense overburden would impede the escape of pore pressures. The very uniform gradation of the Charleston sands and/or the silt content of the sands might also play a role in impeding the densification process.

In studying the Hollywood site, the plot shown in Fig. 5.45 was developed from the findings of Obermeier (written

communication, 1990). The plot shows that at least four episodes of liquefaction have occurred at Hollywood prior to the 1886 event. The figure indicates that both the number and size of liquefaction features have steadily decreased from the earlier earthquakes to the later events. The amount of liquefaction that occurred during the 1886 earthquake is shown to be greatly reduced from that which occurred during the pre-historic events. The reason for the decrease in the level of liquefaction is not obvious, but seems to suggest that conditions have become less favorable for the development of liquefaction following each event; especially if it is assumed that the ground motions of each event were comparable. In this case, the decreasing liquefaction activity could be due to progressive densification of the sands. This idea would be consistent with the top-down densification hypothesis mentioned earlier. An interesting twist however, is that the soil profiles in the 85,000 year-old and present beach deposits do not support the top-down density pattern. This is not completely understood, but is thought to be due to the fact that the majority of these deposits are situated outside of the meizoseismal zone and the development of liquefaction within them has never been extensive enough to cause significant densification of the upper sands.



APPROXIMATE AGE OF EARTHQUAKE

Figure 5.45 Plot Showing Decreasing Number of Large (> 3 ft. Diameter) Liquefaction Features With Successive Seismic Events for Hollywood Ditch Site (Obermeier, Written Communication, 1988).

The idea of progressive densification at Hollywood is further extended to the test sites near Ten Mile Hill which indicate that although liquefiable soils are present, extensive areas with very loose sands are not as common as those areas outside of the meizoseismal zone (i.e. Charleston, Edisto, Mt. Pleasant, Oakland Plantation, etc). The historical accounts appear to indicate that significant densification occurred at the Ten Mile Hill and Warren Farm sites. It may be remembered from the soil profiles for the Ten Mile Hill and Airport sites that the soils below 12 ft. are presently dense and resistant to liquefaction. As discussed in the next chapter however, the historical accounts indicate that the source sands for the sand boils in the Ten Mile vicinity generally came from depths of 12 ft. or more. This would suggest that significant densification of the lower layers has taken place as a result of the liquefaction. Also, at the Warren site the historical accounts and recent paleoseismic investigations indicate that only one liquefaction vent occurred over the entire site during the 1886 event. Interestingly, the field testing there revealed that soils conditions across the site today are relatively uniform as well as dense and resistant to liquefaction. This would suggest that the vent occurred in a localized area of loose material which liquefied and subsequently densified. This possibility seems reasonable

in that soil conditions across the site today are uniformly dense, and it would be expected that if these conditions had existed at the time of the earthquake, then liquefaction would have occurred in other locations as well.

In discussing the occurrence of dense soils in the top of the profile, it has also been suggested that their presence could be related to weathering/aging effects or other geologic processes (Obermeier, 1989). At several of the sites, the sands in the top soil horizons were often found to be weakly cemented and appeared to be over-consolidated. It is apparent that these upper soils have a greatly reduced permeability relative to the underlying loose sands. The cohesive nature of the upper layers gives rise to a "brittle-but-weak" material behavior and serves to explain the formation of the large explosion craters at the Ten Mile Hill and Hollywood areas. The authors postulate that the craters were formed due to rapid localized escape of pore pressures from the underlying loose material through the capping layer, followed by further erosion of the capping layer by outward water flow. Because the development of these capping layers is related to aging, their occurrence in the Charleston area becomes more pronounced in the older beach deposits (Obermeier, 1989). The relation of these or other geologic processes to the dense-over-loose pattern is unclear, but their effect appears less dramatic

than the progressive densification effect in that the presence of the dense overburden is generally absent from the 85,000 year-old deposits, although these deposits are of sufficient age to reflect weathering and other geologic processes of the upper soils.

The Charleston environment is unique in that liquefaction has occurred repeatedly in relatively old soil deposits. Much of our knowledge concerning the influence of sediment age upon liquefaction susceptibility has been derived from locations of relatively high seismicity, where multiple episodes of significant shaking can occur over intervals of 100 years or less. The paleoliquefaction evidence suggests that liquefaction-inducing earthquakes have occurred in Charleston at most every 500 to 600 years, and more likely every 1000 to 1500 years. In other regions, where the rate of seismicity is high, it has been noted that liquefaction does not occur in soils older than about 10,000 years, and seems to be most common in clean sands (Youd, 1978). In the Charleston environment however, recent liquefaction has occurred in soils as old as 230,000 years, including silty sands. The ability of the older soils to liquefy may be related to the silt content of the sands which prevents densification after liquefaction (as mentioned earlier), or it could be due to the lack of repetition of large shakes that would cause densification.

The latter idea is more consistent with the overall evidence since there are significant zones of loose clean sands as well as loose silty sands within the older deposits.

One objective of the field investigations was to qualify the various geologic units in terms of their susceptibility to liquefaction. Based on the soil information gathered from the test sites and outside agencies, it appears that the liquefaction susceptibility of the Charleston soils is related to geological environment. The beach deposits are shown to have a high susceptibility to liquefaction, as compared to the relatively low liquefaction potential of the backbarrier and fluvial deposits. Also, it appears that within the individual soil units, soil conditions and density patterns are relatively consistent. The soils in the older beach deposits are shown to have less liquefiable material with more fine-grained soils at depth, and the presence of dense sands at the top of the profile. The soils within the younger beach deposits have thicker zones of liquefiable material and less variability in density with depth or from site to site. Fig. 5.46 shows a simplified profile of the upper 30 ft. of soil along the intermediate, 85,000 year old inland beach moving from 3 to 30 miles northeast of Charleston along U.S. Highway 17 (Section A-A on Fig. 3.2). The profile was prepared using averages of many boring logs. It can be seen that fine

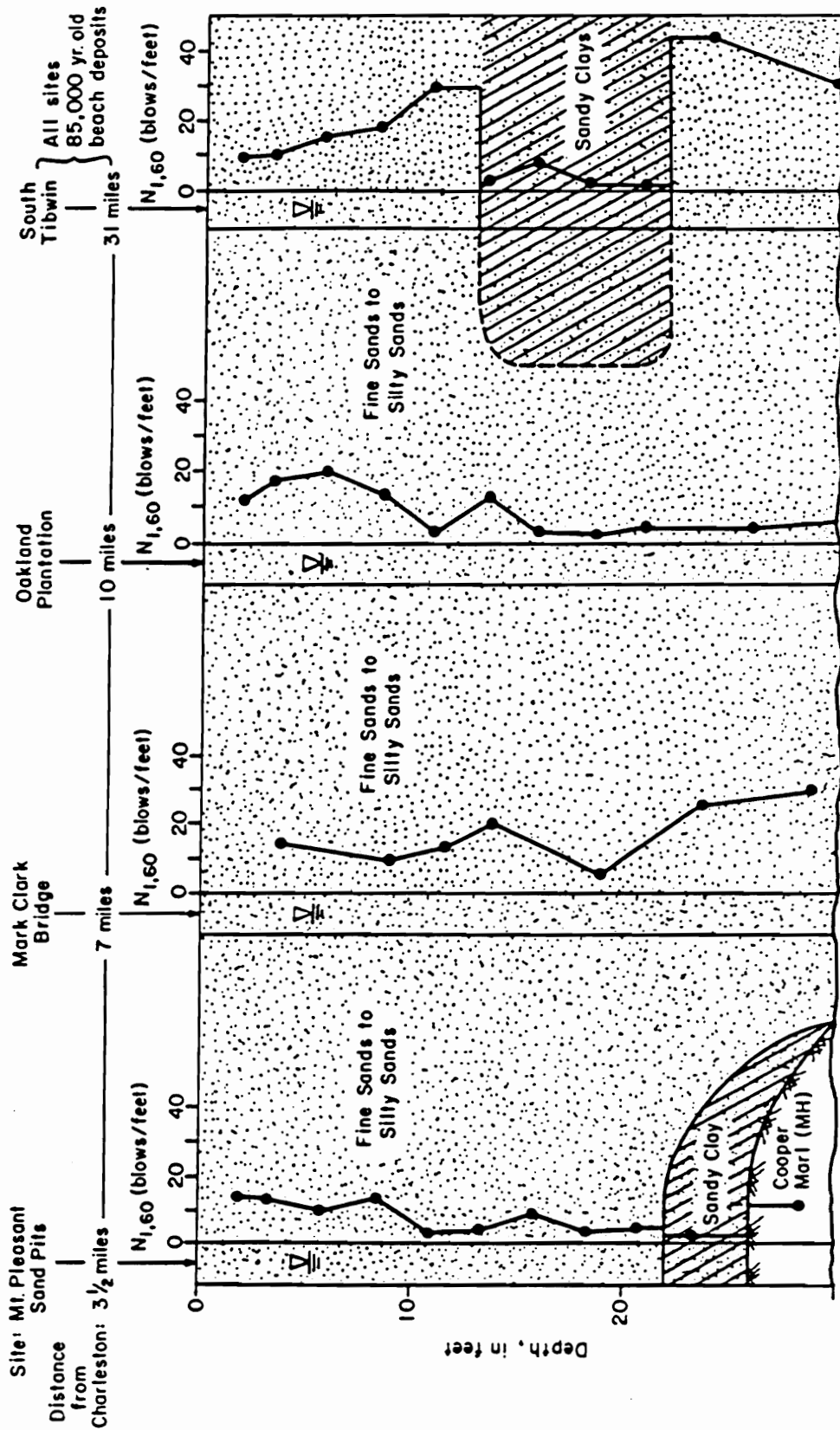


Figure 5.46 Average Soil Conditions Along U.S. Hwy 17 From Charleston to Moore's Corner (Section A-A', Fig. 3.2).

sands and sandy silts are found consistently in the profile, with blow counts often below 10 blows/ft. The water table is located in the upper 5 ft. An averaged profile of the upper 30 ft. of soil that cuts across the beach ridge deposits is shown in Fig. 5.47 (Section B-B on Fig. 3.2). While the sandy soils in the present beach and the intermediate beach show blowcounts below 10 blows/ft., the blow counts in the older beach are typically above 10 blows/ft. Notably the loosest layer of sand in the older beach is at depth, just above the underlying clayey sands. Also, the percent of the profile occupied by sandy soils and clayey sands under the sands increases as the age of the beach deposit increases. The conditions for the older beach deposits shown in Fig. 5.47 are highly simplified in that the soil profile in these deposits can be complex, and locally the sands in them can be looser than indicated in the figure. However, Fig. 5.47 is consistent in that the older beach ridges on the average tend to show denser sands and the presence of more cohesive soils than the younger beach ridges. The uniform nature of soil conditions within the individual deposits gives rise to an almost equal susceptibility of the sediments to liquefaction with distance away from the epicentral area. This suggests that the observed decaying pattern of liquefaction activity with distance from the epicenter during the 1886 event was due

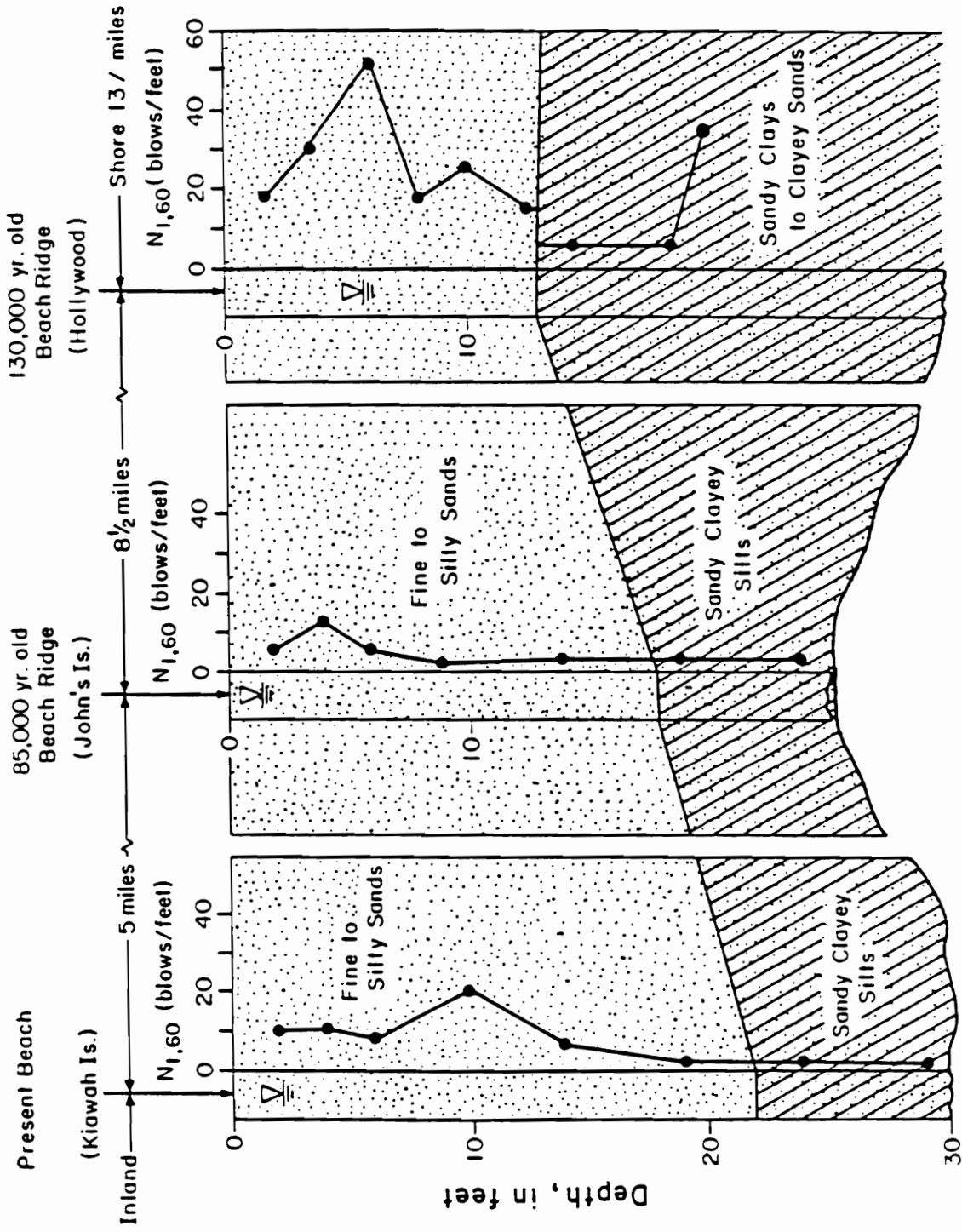


Figure 5.47 Average Soil Conditions From Kiawah Island Inland to Hollywood (Section B-B', Fig. 3.2).

primarily to attenuation of the earthquake energy as opposed to local changes in soil conditions and liquefaction susceptibility.

Another objective of the field testing program was to determine an appropriate Q_c/N (CPT-to-SPT) conversion factor for use in the liquefaction analyses in Chapter 6. At sites where feasible, CPT holes were located adjacent to SPT holes for the purpose of calibrating the penetration resistances. A summary of the results of this effort is given in Table 5.3 which lists the Q_c/N factors obtained from the calibration tests. The table shows the Q_c/N factor for the Charleston sands to range as high as 13.0, with an average close to 8.5. The values published in the literature however, range from 4 to 5 for sands with similar median grain sizes. The reason for this difference is not completely understood, but may be related to the extreme uniformity of the Charleston sands.

DISCUSSION OF LABORATORY TESTING RESULTS

Sieve analyses and classification tests were performed on sands from the Hollywood, Warren, Ten Mile Hill, Oakland Plantation, South Tibwin, Edisto Island, and Georgetown sites to determine possible variations in grain size or

Table 5.3 Comparison of Cone-to-Blowcount (Qc/N) Conversion Factor From Literature to Those Obtained at Virginia Tech Test Sites.

Site	Number of Available QC to N Comparisons	Avg Median Grain Size, D ₅₀ (mm)	Published(1) Qc/N	Actual(2) Qc/N	Standard Deviation
Hollywood	18	0.19	5.0	10.0	3.5
Mt. Pleas. Pits	10	~ 0.20	5.0	8.0	2.6
Oak. Plantation	13	0.20	5.0	8.0	2.8
Edisto Island	12	0.15	4.5	7.5	3.0
M. Clark Br.	9	~ 0.20	5.0	13.0	2.8
S. Tibwin/	13	0.22	5.0	7.0	2.9
St. James Sch.					
Georgetown	6	0.19	5.0	5.0	0.8
Mean Values:	-	0.19	5.0	8.4	2.6

 (1) From Robertson and Campanella (1983).
 (2) Based on Penetration Tests by Virginia Tech and Commercial Firms.

gradation that could result in differing liquefaction susceptibilities. The gradation curves showed the sands to be uniform ($C_u = 1.24-2.29$) and fine ($D_{50} = 0.12-0.20$ mm) and to remain so with depth. In Fig. 5.48, the gradation curves for all seven of the test sites are shown with respect to a bounding zone of gradation curves which, as proposed by Tsuchida (1970), represent the range for the "most liquefiable soils". It can be seen that the gradation curves for the Charleston sands plot in the range considered to be most susceptible to liquefaction. It should be noted that Tsuchida's gradation boundaries were determined on the basis of gradation curves for soils which did or did not liquefy during recent earthquakes. Data from laboratory testing (Castro and Poulos, 1977; Seed and Idriss, 1971) support this proposed boundary by demonstrating that sands with a D_{50} in the range of 0.10 mm and constant coefficient of uniformity have a low resistance to cyclic loading. Although sieve tests were not conducted on material at the other test sites, visual field classifications suggest that the sands at these sites also fall on or very close to the composite curve. Further, several of the SPT data sets collected from outside sources contained gradation curves from sites within the study region, and in all cases the curves were shown to fall within the range of the composite curve obtained by Virginia Tech personnel. Friction ratio

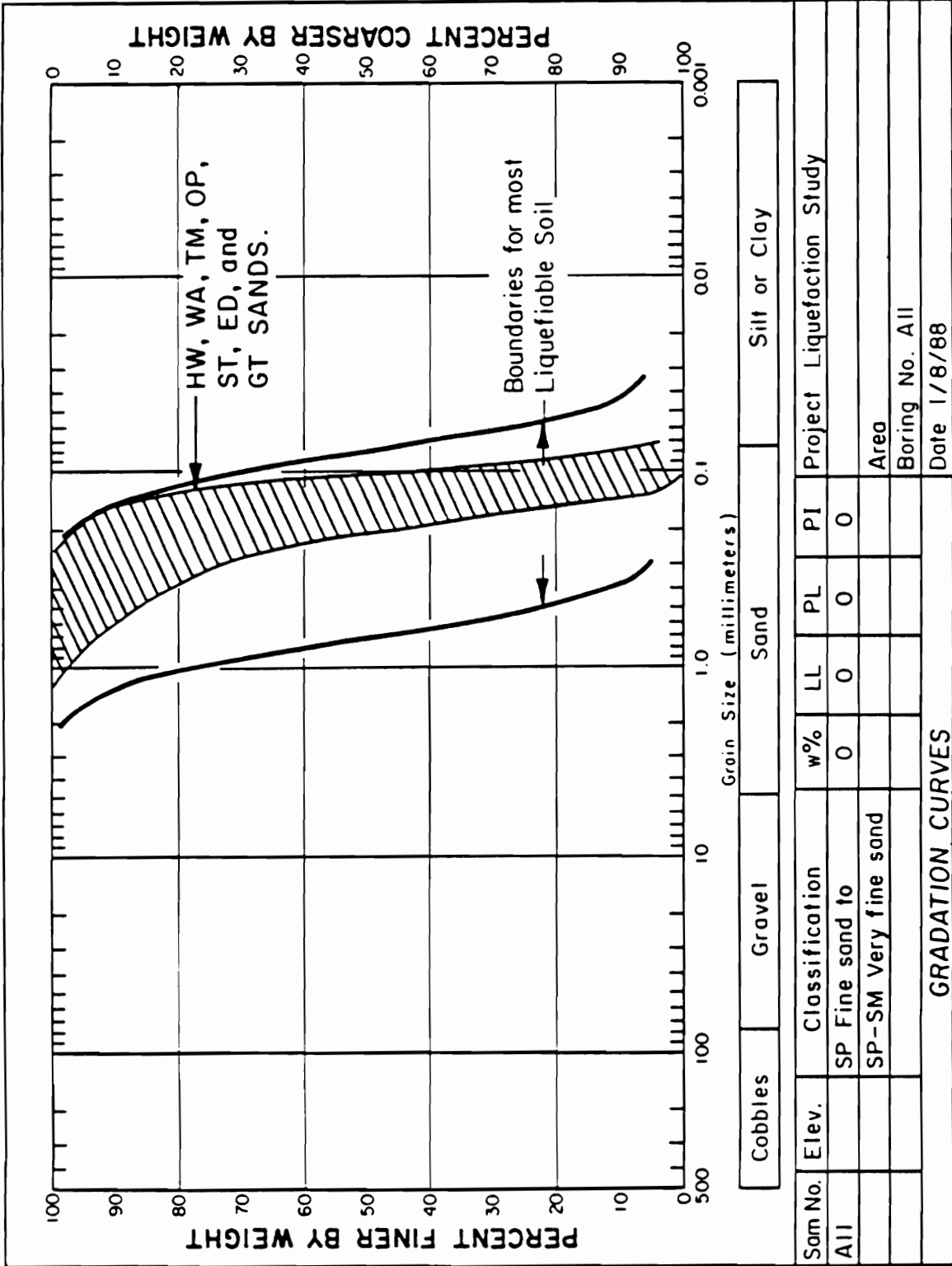
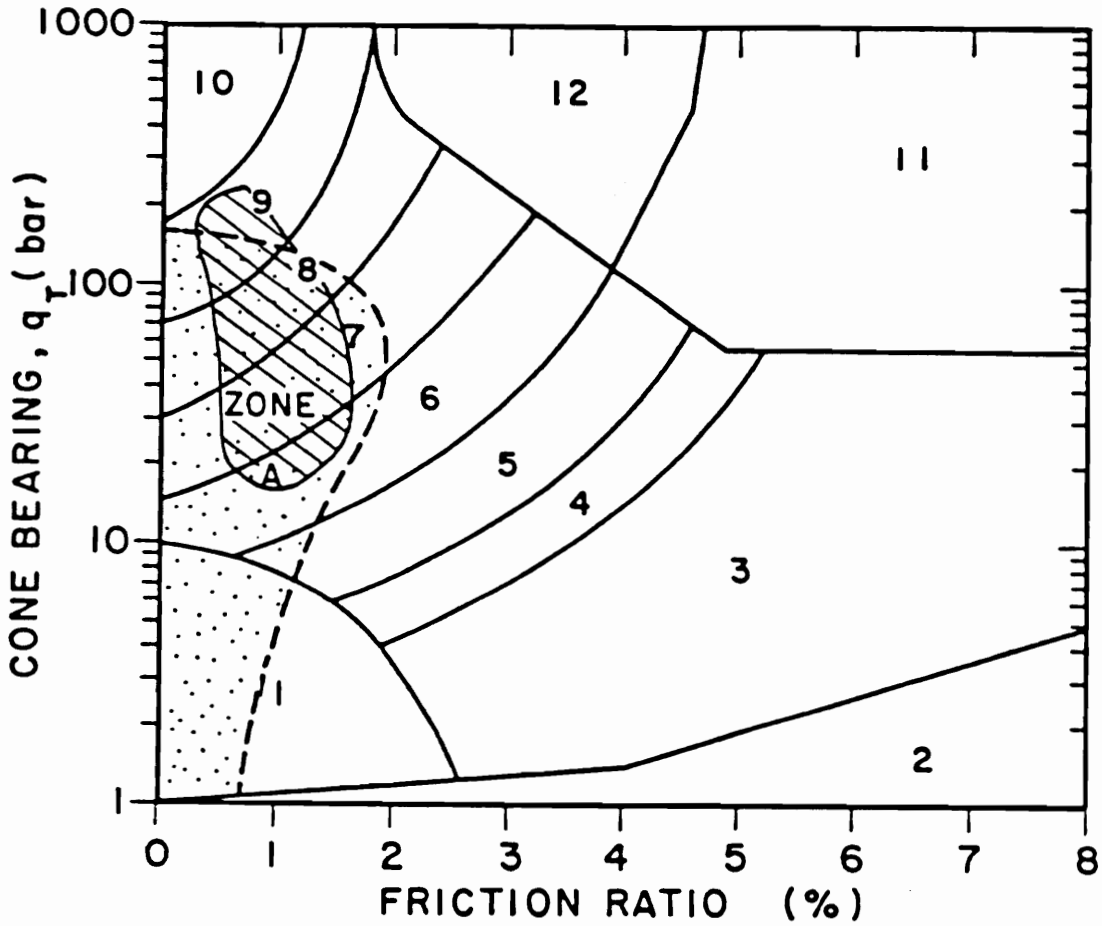


Figure 5.48 Range of Gratation Curves for Hollywood, Warren, Ten Mile Hill, Oakland Plantation, Edisto Island, and Georgetown Sites.

measurements from the CPT's were used to supplement the sieve tests utilizing a simplified soil classification chart. The cone data for the upper sands at all sites plots almost entirely in the shaded zone of Fig. 5.49. This is well within the stippled zone A which has been proposed by Robertson and Campanella (1985) to be the most liquefiable soils.

The cyclic behavior of a sand can be influenced by its angularity, degree of cementation, and prior strain history. The effects of grain angularity on the liquefaction potential of sands has been investigated by Vaid et al. (1985). Their results indicate that other things being equal, angular sands are more resistant to liquefaction than less angular sands, especially at low confining stresses. The angularity of the sands investigated for this study were classified as sub-angular to angular. With the exception of the top organic layer, the sandy soils retrieved from auger cuttings, SPT's, and trench walls, all showed negligible cohesion due to grain cementation. The effects of strain history on the liquefaction susceptibility of sands are dependent upon the magnitude of strain imparted to the soil during seismic loading. It is assumed that since all test sites are located in approximately the same seismic regime, that the strain histories of these deposits are similar. All other factors being equal, the uniform, fine-grained



<u>Zone</u>	<u>Soil Behavior Type</u>
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained*
12	sand to clayey sand*

* overconsolidated or cemented.

Figure 5.49 Soil Classification Chart Showing Proposed Zone of Liquefiability Soils (Stippled Zone) and the Range of CPT Data From All Test Sites (Hatched Zone A) (Chart From Robertson and Campanella, 1985).

character of the near-surface sands increases the likelihood of liquefaction during seismic loading (Ishihara, 1985; Seed and Idriss, 1971; Lee and Finton, 1969). This is supported by findings of Cullen (1985) who based on cyclic triaxial tests on sands from the Hollywood area, concluded that the Charleston sands have a relatively low resistance to liquefaction when compared to coarser grained sands. The lack of cementation of the soils (below the organic horizon) also contributes to an increased liquefaction susceptibility. It thus appears that with regard to grain-size, angularity, cementation, and strain history, the soils of interest at the test sites within the epicentral region are very similar in character.

CHAPTER 6

ANALYSIS

It is the intent of this study to use the findings of the field and laboratory work to help understand the levels of seismic shaking which occurred during the 1886 Charleston earthquake. The methods that are used for the definition of seismic shaking levels are discussed in the first sections of this chapter and followed in the latter sections by formal analyses of the data.

In the absence of seismic records for the 1886 earthquake, strong motion estimates have been postulated in previous work on the basis of knowledge of attenuation of intensity with distance from the epicenter and scaling relationships for simulated eastern United States earthquakes (Bollinger, 1977; Bollinger, 1983; Campbell, 1976; Rizzo et al., 1986). Based on first-hand accounts of earthquake effects and damages, and the duration of ground shaking, it

is generally accepted that a MM intensity of X was experienced in the meizoseismal zone during the 1886 event, while a maximum intensity of IX was felt in the City of Charleston and other nearby regions. In their study of intensity data from the 1886 event, Bollinger (1977) and Nuttli (1976) have estimated that the earthquake magnitudes were $M_s = 7.7$ and $m_b = 6.6 - 6.9$ on the basis of Nuttli et al. (1979).

Using this information and a variety of procedures, different estimates of the peak ground acceleration levels that occurred during the 1886 earthquake have been obtained. Based on scaling relationships for eastern U.S. earthquakes of $M_s = 7.65$, $m_b = 6.7$, a recent study has estimated that peak horizontal accelerations of approximately 0.6g were experienced within the 1886 meizoseismal zone. This estimate was made for the near surface peak acceleration at a "soft" site composed of unlithified sediments (Rizzo et al., 1986). Chapman et al. (1989) placed the range of peak 1886 accelerations at from 0.4 to 1.0g. Other estimates have been made by Campbell (1976) who used empirical procedures to estimate ground motion intensities for an eastern U.S. earthquake similar to the 1886 event. In his analyses, the use of magnitudes $M_s = 7$ and $m_b = 6.8$ resulted in median acceleration values close to 0.55g for soft sites located along the surface projection of the fault rupture, and 0.3g for a site located 9.3 miles from this fault projection.

These estimates for near field accelerations approximately agree with a more recent study by Krinitzsky and Chang (1987) who compiled world-wide earthquake intensity data for hard and soft sites. The use of their curve shown in Fig. 6.1, suggests that for a MM intensity of X, a peak horizontal acceleration of about 0.5g would be experienced in the near field (< 25 miles from fault zone) at a soft site. Thus, the seismological evidence would appear to indicate that peak horizontal accelerations within the 1886 meizoseismal zone were in the range of 0.5g to 0.6g. It is important however, to mention that there are still open questions about these conclusions because of our lack of hard knowledge about the 1886 event and about some of the assumptions upon which these estimates are based.

6.1 FUNDAMENTALS IMPORTANT TO THE METHODS USED TO DETERMINE SEISMIC SHAKING LEVELS

The information that is available to the present study which related to the determination of seismic shaking levels included:

1. Sand Densities - SPT and CPT data were directly related to the in-situ density of the sands.
2. Sand grain sizes and grain size distribution - sieve test on samples from the SPT spoons and from the auger cuttings were performed in large numbers. Additional

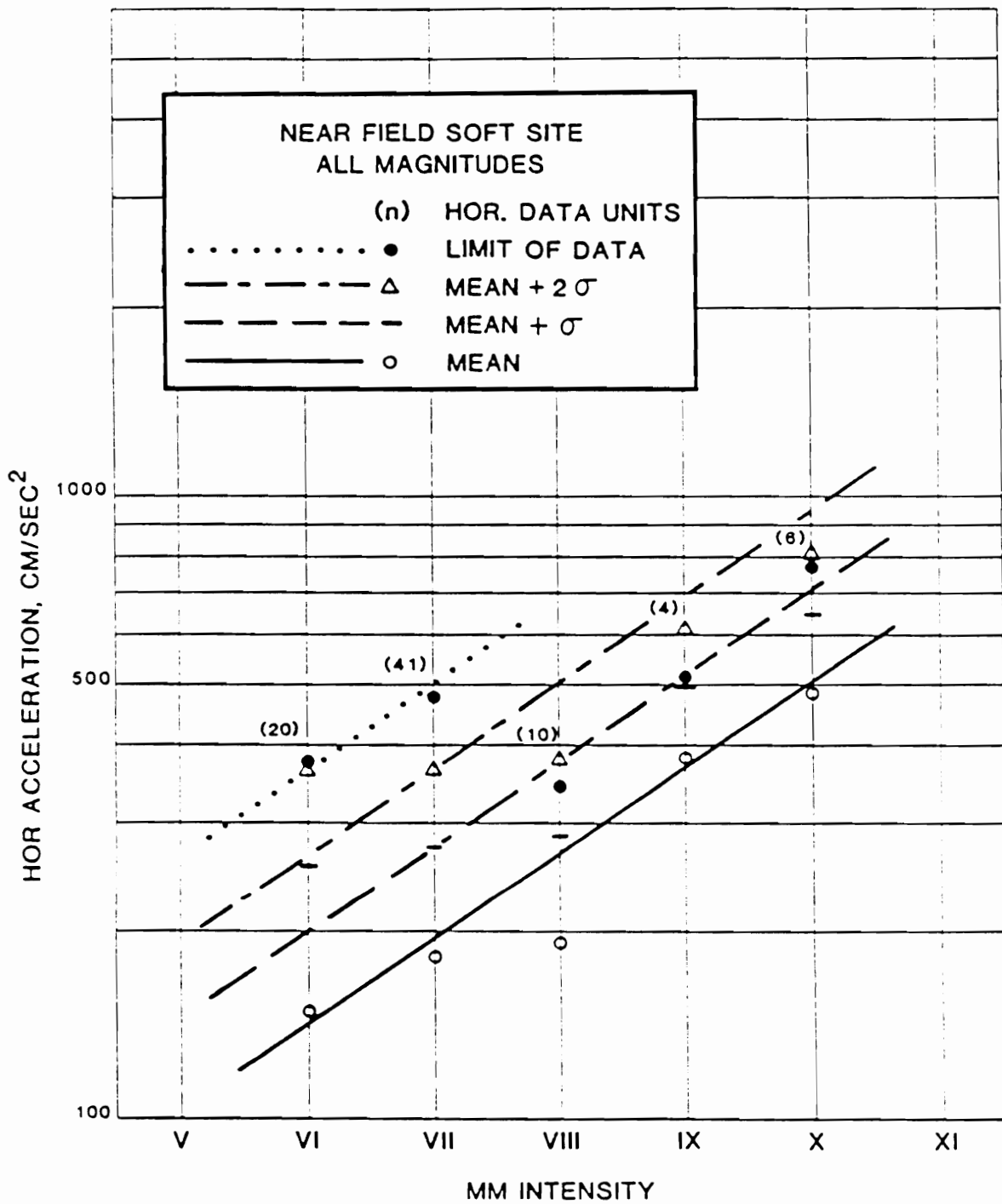


Figure 6.1 Peak Horizontal Acceleration Versus Modified Mercalli Intensity (Krinitzky and Chang, 1987).

SPT and sieve data were made available by local soils firms and government agencies.

3. Site Stratigraphy - Soil layering profiles were determined from the soil classification logs of the augers and SPT holes as well as from available friction ratio-soil type correlations for cone penetration tests. The position of the water table was known from the testing operations.
4. Historical and/or paleoseismic records of liquefaction occurrences at each site. In some cases this was important in demonstrating a large extent of 1886 liquefaction, while in others it was important in indicating the relative lack of liquefaction.

To translate this type of information into likely earthquake shaking levels requires interpretation and back-calculation. In this process ground rules need to be established, and the first step involved the definition of certain basic assumptions. Based on the best available information, the following assumptions appeared to be reasonable:

- 1.) For practical purposes, all of the sands in a deposit of the same age have had an equal stress history;
- 2.) None of the samples (below the organic soil horizon) showed any signs of cementation, thus it was assumed that no differences in field performance were caused by this factor; and

3.) The historical record and the coincidence of the surficial organic horizon with the present water levels suggested that the ground water table has been at the same depth since the 1886 event, except in localized areas where drainage ditches were excavated in the past or dewatering operations were underway.

In these areas, the water table was lowered close to the trench or dewatered area, but it is apparent from nearby swampy areas where the water table was previously. It is thus assumed that the water table for the 1886 earthquake was the same as its present position unless there were obvious recent local drainages.

Second, the possible reasons for the presence or lack of liquefaction features on or near the ground surface had to be considered. Liquefaction features are known likely to occur where cyclic loading of sufficient strength and duration is applied to shallow saturated sands and non-plastic silts. They will be most prevalent in areas with a high ground water table and loose sands. In addition, the occurrence of liquefaction features is enhanced if the liquefiable sands are present in thick layers and the overlying non-liquefiable soil thickness is small; if this situation is reversed, the presence of liquefaction features is suppressed.

Third, there is the question of the present density of the sands versus that of the sands before the 1886 event.

This relates to two issues: 1.) Can the sand densities today be used to infer the response of the sands in 1886?; and, 2.) If the densities today are thought to be different than those in 1886, what is the implication relative to progressive densification as a result of prior liquefaction? In regard to the first of these points, it would be expected that if a site is out of the meizoseismal zone, and if little evidence exists that any significant liquefaction occurred there, that the sand densities today should be representative of conditions before the 1886 earthquake. This rationale is extended to other sites within the meizoseismal zone where the paleoseismic and/or historical evidence shows that few or minor liquefaction features developed. If, on the other hand, a site is known to have exhibited prominent liquefaction features (e.g., Ten Mile Hill), it is assumed that the sand densities may have been significantly altered as a result of the earthquake. In this case, it would be expected that some densification of the sands within the source zone has taken place as a result of the liquefaction. The approach that is used in this chapter is to first draw conclusions based on the present densities, and then later discuss the implications of the possibility that the present densities may be different from those which existed at the time of the 1886 event.

6.2 ANALYSIS OF LIQUEFACTION IN SANDY SOILS WITH LEVEL GROUND CONDITIONS

The liquefaction analyses that allow estimation of ground shaking levels consisted of two parts. First, the possibility that liquefaction can occur in the sandy soils at the site was evaluated. Second, the possibility of development of liquefaction features as a result of the liquefaction was determined. The first of these tasks involved testing the site conditions against the effects of different possible levels of shaking following the procedures of Seed and others (Robertson et al., 1983; Seed and Idriss, 1971; Seed et al., 1983). The second was done using the layer effects chart of Ishihara (1985) which relates the presence of liquefaction features to the thickness of the liquefiable soil layer and the overlying non-liquefiable soil layers. It should be recognized that neither of the evaluation procedures is based on exact science, and thus there is some subjectivity in the analysis outcome.

The process of liquefaction is induced by shear stresses that are transmitted to the saturated sand layers during seismic shaking, as described in Chapter 2. The shear stresses developed in the soil layer during an earthquake are due primarily to the upward propagation of shear waves in the deposit and can be directly related to

the peak acceleration induced at the site as described by Seed and Idriss (1982). In this way, the development of liquefaction can be related to acceleration. Although levels of observable liquefaction phenomena have also been related to MM intensity values, this relation can only be used in a subjective sense.

The procedure for assessing the liquefaction potential of a soil deposit subjected to seismic loading using the results of in-situ tests involved two steps: 1.) Estimation of the cyclic stress or strain condition developed in the field due to the design earthquake; and, 2.) Estimation of the cyclic resistance of the soil. For soil under level ground conditions, the cyclic behavior is represented by the average cyclic stress ratio, i.e., the ratio of the average cyclic shear stress, τ_{av} , developed on horizontal planes in the soil as a result of ground motions, to the initial effective stress, σ'_o . This parameter has the advantage of taking into account the depth of the soil layer involved, the depth of the water table, and intensity of earthquake shaking (Seed and Idriss, 1982). The cyclic stress ratio experienced in the field due to an earthquake can be computed from the equation:

$$\tau_{av}/\sigma'_o = 0.65 \cdot a_{max}/g \cdot \sigma_o/\sigma'_o \cdot r_d \quad [\text{eq. 1}]$$

in which a_{\max} = peak acceleration at the ground surface; g = gravitational acceleration; σ'_o = initial effective stress in the sand layer under consideration; σ_o = the total stress at the same point; and r_d = stress reduction factor varying from a value of 1.0 at the ground surface to 0.9 at a depth of 30 feet (Seed and Idriss, 1971). Values of the cyclic shear stress ratio have been determined for areas which have or have not exhibited surface evidence of seismically induced liquefaction. These values are then correlated with soil properties indicative of cyclic shear resistance (Seed and De Alba, 1986).

Estimation of liquefaction potential is obtained using either laboratory testing methods on representative field samples or through correlations to in-situ testing results. The inherent difficulties in obtaining undisturbed field samples of loose, saturated cohesionless soils has led to the prominence of in-situ testing methods for liquefaction potential evaluation, with the SPT and CPT most commonly used. The first in-situ evaluation method was developed for use with the SPT because of its predominant usage in geotechnical practice. For this method, Seed and his co-workers (1984) developed charts such as that given in Fig. 6.2, which relate cyclic stress ratios (τ_{av}/σ'_o) causing liquefaction to modified SPT blowcounts ($N_{1,60}$). His work was based on world-wide field performance data from sites which did or did not liquefy during large earthquakes. The

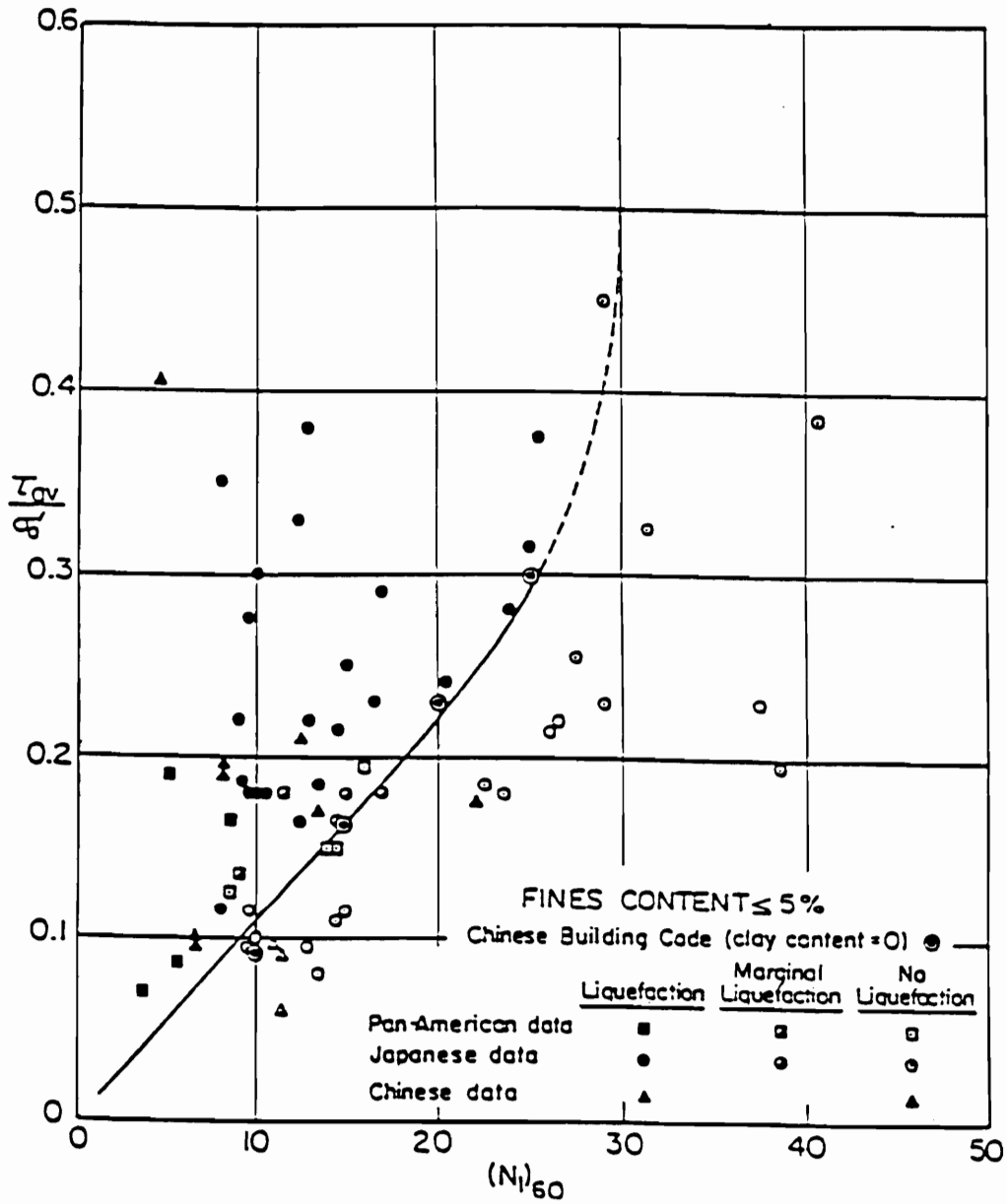


Figure 6.2 Relationship Between Stress Ratios Causing Liquefaction and $N_{1,60}$ Blowcounts for Clean Sands for Magnitude 7.5 Earthquake (Seed et al., 1984).

chart shown in Fig. 6.2 was developed for clean sands and for magnitude 7.5 earthquakes. Examining the chart, it can be seen that a boundary line separates the data points representing conditions favorable for liquefaction from those which are unfavorable. The position of the separating line is not definitive, but rather is based on judgement, as it can be seen in the figure that some points representing liquefaction lie below the line, while others corresponding to non-liquefaction lie above the line. The line is, however, positioned somewhat conservatively in that the majority of the "misclassified" points lie above the line and represent sites that did not liquefy although the points fall within the "liquefiable" portion of the chart. The implication to this study is that the accelerations back-calculated using the curve in Fig. 6.2, could, in some cases, be underestimated because of this inherent conservatism.

The cyclic stress ratio required to cause liquefaction is influenced by the magnitude and duration of seismic shaking, and the grain size of the material under consideration. The field performance data used to develop the chart in Fig. 6.2 inherently accounts for the duration of seismic ground motions. The majority of Seed's field data are for $M = 7.5$ earthquakes, although corrections can be made to account for the duration of earthquakes of different magnitude by the use of scaling factors which relate the

number of stress cycles required for liquefaction and the number of representative stress cycles for earthquakes of different magnitudes (Seed and Idriss, 1982). The liquefaction analyses in this chapter are performed for a magnitude 7.5 and a magnitude 6 event (magnitude here refers to surface wave magnitude, M_S). Both analyses are based on Seed's curve of Fig. 6.2 for magnitude 7.5 earthquakes, with the appropriate scaling factors used to construct a magnitude 6 curve (see Fig. 6.3). It should be noted that the 7.5 magnitude is slightly less than the upper bound 7.7 estimate for the 1886 event, but is reasonably close, and is favored because of the wealth of data collected for earthquakes of magnitude 7.5. The use of the $M = 7.5$ curve would, however, correspond to a slightly higher cyclic stress ratio and acceleration level needed to cause liquefaction than would be obtained with a curve constructed for a 7.7 event. The analyses for a magnitude 6 event are incorporated into the study for two main reasons: 1.) To demonstrate the effect of smaller magnitudes upon the back-calculated acceleration levels. This is especially important if it is to be considered that the 1886 event may have been smaller in magnitude than the estimated 7.7; and, 2.) The $M = 6$ analyses give an indication of the response of the Charleston sands to future events of this magnitude. This is important in that the seismological evidence indicates that a magnitude 6 event is prone to occur in the

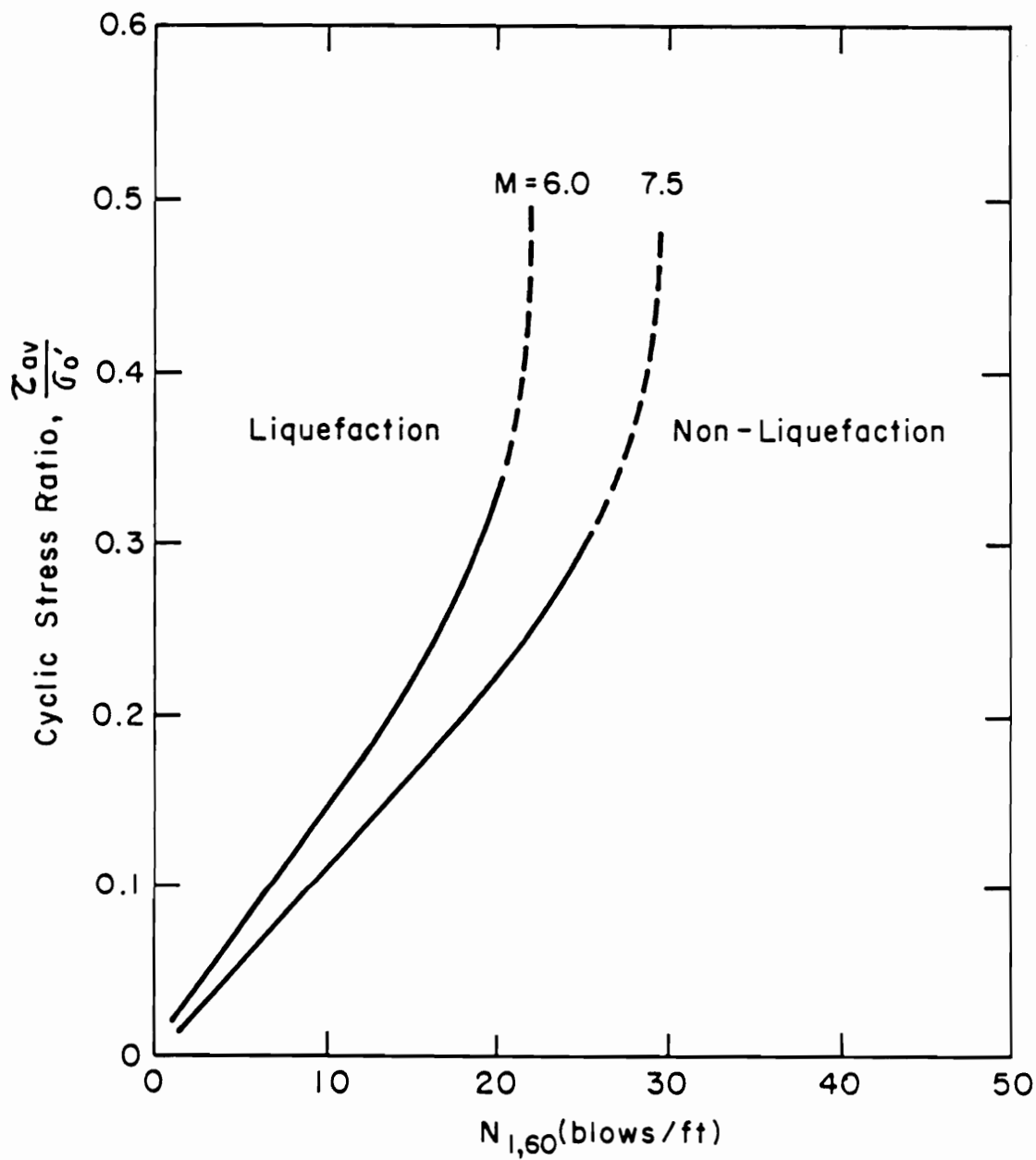


Figure 6.3 Correlation Between Field Liquefaction Behavior of Clean Sands and Modified Penetration Resistance for Magnitude 6 and 7.5 Earthquakes.

Charleston region in the not too distant future, with the recurrence interval estimated at roughly 225 years (Amick and Talwani, 1986; Johnson and Bazan-Zurita, 1986). Bollinger et al. (1989) estimate an $M = 6$ ($m_D = 5.8$) recurrence at 230-1700 years (95% confidence interval) for Charleston.

Loose silty sands and sandy silts tend to have a higher resistance to liquefaction than coarser sands at the same penetration resistance due to the decrease in penetration resistance associated with reduced mean grain size (Ishihara, 1985; Robertson and Campanella, 1985; Seed and De Alba, 1986). Corrections for mean grain size and fines content of the soil can be made by shifting the liquefaction boundary on the cyclic stress ratio versus penetration resistance chart as described by Seed and De Alba (1986). The plot in Fig. 6.4 shows the Seed chart for magnitude 7.5 earthquakes constructed for silty sands containing up to 35% non-plastic fines. It can be seen that the 15 and 35% curves trend in the same fashion as the curve for clean sands, but are shifted to the left along the abscissa. The analyses in this chapter account for the effect of fines by appropriately modifying the penetration resistances by the amounts reflected in Fig. 6.4, and then using the Seed curve for clean sands in Fig. 6.3 for the liquefaction analyses. An example would be that if a silty soil containing 15% fines is to be analyzed, the normalized blowcounts obtained from

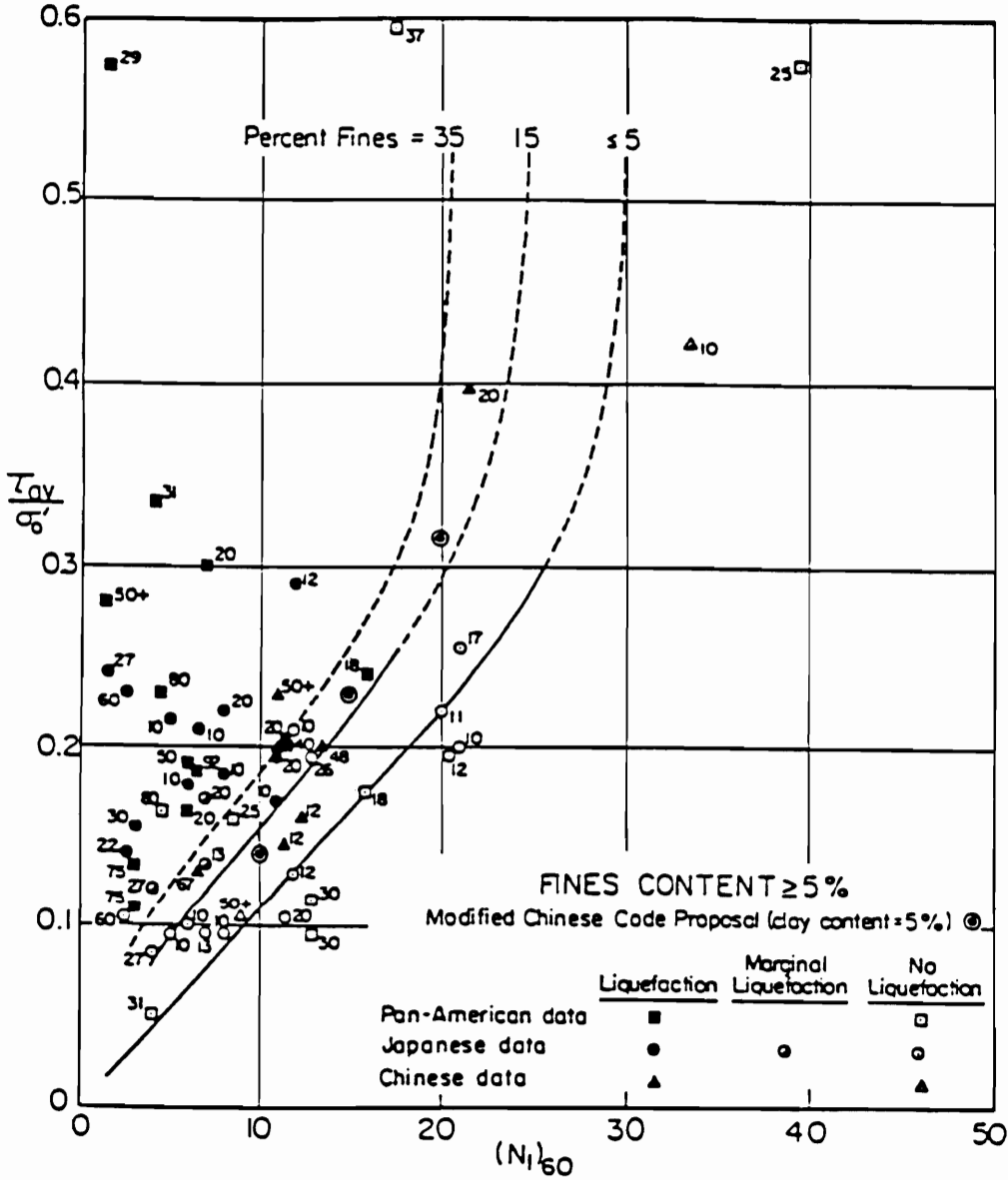


Figure 6.4 Relationships Between Stress Ratio Causing Liquefaction and $N_{1,60}$ Blowcounts for Silty Sands With 5 to 35% Fines for Magnitude 7.5 Earthquakes (Seed et al., 1984).

the stratum would first be increased by 5 blows/ft. to account for the horizontal offset between the clean sand curve and the curve for 15 % fines (see Fig. 6.4). The layer would then be analyzed with these modified blowcounts using the clean sand curve. This procedure would be effectively the same as using the 15% fines curve in Fig. 6.4 with the original normalized blowcounts, but is more convenient because only one curve is needed for all of the analyses.

The fines content of the soils at the Virginia Tech test sites as well as the sites investigated by commercial firms, were determined by either: 1.) laboratory classification tests on SPT or auger samples; 2.) friction ratio correlations from the CPT's; and/or, 3.) visual field descriptions from SPT boring logs. Less credibility is given to the visual field descriptions in that it became apparent in examining gradation curves and soil classification data from outside agencies, that some sandy soils had been misclassified on the SPT logs. The most frequently encountered example was the misclassifying of very fine sands (SP's) as silty sands (SM's). For this study however, wherever SM's were indicated on the basis of field descriptions, the appropriate corrections are made for the effect of the fines unless data are available to indicate otherwise. Because silty sands have an increased resistance to liquefaction, the implications are that the misclassifi-

cation would cause an overestimate of the accelerations required to liquefy the sands. This mainly pertains to those areas from which SPT borings (without laboratory classification data) were obtained from commercial firms.

The lack of a significant data base for CPT's in areas experiencing seismic liquefaction necessitates the conversion of the CPT resistance values to equivalent SPT blowcounts. For the analyses in this chapter, the CPT values are converted to equivalent SPT blowcounts using the Q_c/N factors recommended in the literature for soils with similar median grain sizes. The grain size of the Charleston sands corresponds to a Q_c/N factor of about 4.5 (Robertson and Campanella, 1983). However, it is notable that comparisons of the SPT and CPT resistance values at the sites where both were available from results of this investigation, suggested that the Q_c/N factor could be as high as 10. The reason for this difference in the field and published values is not obvious, although it was decided that the published values ($Q_c/N = 4.5$) would be used, but with the understanding that the equivalent SPT blowcounts may be too high, and that the acceleration levels predicted with these resistances may consequently be overestimated.

It is useful at this point to summarize the assumptions and analyses procedures presented thus far, and to review their implications to the analyses outcome. In most cases, the procedures tend to over-predict the acceleration levels

that would explain the liquefaction phenomena evidenced in 1886. The more important of these involve the following:

- 1) The densities of the sands in the source zones were assumed to be essentially unchanged from those which existed at the time of the earthquake. In reality, the sands in these zones, especially in areas where liquefaction was extensive, are most likely higher than those which existed prior to the 1886 quake. Because the present-day densities are used for back-calculation purposes, the acceleration levels predicted for the 1886 event could be overestimated.
- 2) The penetration resistances from the CPT's were converted to equivalent SPT blowcounts using the published Q_c/N factor of 4 to 5, although on-site calibration data suggested that this factor is probably too low for the sites investigated. The use of the published factor would increase the penetration resistances obtained in the CPT to SPT conversion process. For instance, if a Q_c/N factor of 4.5 is used, a sand stratum with an average Q_c value of 200 bars would be analyzed as having an equivalent blowcount of about 44 blows/ft. On the other hand, if a $Q_c/N = 10$ is used, an equivalent blowcount of 20 blows/ft. is obtained. The 44 blows/ft. value would be considered resistant to liquefaction, whereas the 20 blows/ft. resistance is indicative of liquefaction susceptibility even at

moderate levels of ground shaking. Thus, based on the Virginia Tech field tests, the use of published Q_c/N factor appears to be conservative for the Charleston sands, and in some cases, is expected to overestimate the back-calculated acceleration levels.

- 3) The Seed curve for a magnitude 7.5 earthquake was used in the analyses, although the magnitude of the 1886 event has been estimated to be as high as 7.7. The use of the 7.5 magnitude curve would result in back-calculated accelerations slightly higher than those that would be obtained if a curve constructed for a magnitude 7.7 event were used. However, ± 0.2 M units are within the standard error of estimate.
- 4) The Seed chart of Fig. 6.3 was used in the analyses although the conservative position of the separating line on the chart (i.e., several "non-liquefaction" points lie in the "liquefaction" zone) suggested that in some cases the cyclic stress ratios and accelerations needed for liquefaction could be underestimated.
- 5) The appropriate corrections for the effect of fines were made for all soils that were visually classified by commercial firms as having have more than 5% fines, although it was apparent that the fines content was sometimes overestimated. In cases where the fines content was overestimated and the correction for the effect of the fines was made, higher back-calculated

accelerations would result, as the sands would appear less liquefiable in the analyses procedures than would otherwise be indicated by their proper classification. Collectively, these factors seem to indicate that the analysis procedures are conservative in that they tend to favor higher back-calculated acceleration levels. The approach used in this study was to employ the analysis procedures discussed above with the understanding that some amount of conservatism was inherently introduced into the analyses, and that the back-calculated acceleration values for the 1886 event were probably upper bound values.

Finally, it should be considered that Seed's data are largely from the west coast and other very active, high strain-rate, generally interplate or plate-marginal seismic zones. Charleston, on the other hand, is a low activity, low strain-rate, intraplate zone. Thus, some differences in seismic source characteristics (mechanisms, focal depths, stress drops) can be expected. It is unclear, however, as to the effects of these differences upon the results obtained using Seed's methods for the Charleston environment.

Another difference is that the Charleston soils associated with past liquefaction activity are much older than the soils included in Seed's data base. However, other than obvious aging effects such as cementation or cold bonding of the sand grains (neither of which was observed),

there is no reason to believe that this would create any uniqueness that would fundamentally change the response of the Charleston soils to seismic loadings relative to soils from other regions.

6.3 SITES WITHIN THE 1886 MEIZOSEISMAL ZONE

ANALYSES OF VIRGINIA TECH SITES

6.3.1 Hollywood Site

The Hollywood Ditch site was the most thoroughly investigated site of the study -- the field and laboratory work done there was extensive, and our knowledge of soil conditions across the site and the amount of liquefaction that occurred there in 1886 are well established. Obermeier, in his investigations at the site, documented that the 1886 event produced only minor liquefaction along the 9000 ft. drainage ditch. Although most of the features he identified were close to the ground surface, it is important to note that his assessment is based on observations down to or close to the level of the source bed for the 1886 features (it may be remembered that the ditch is about 10 ft. deep and the source bed varies from 5 to 15 ft. in depth). This meant that any suppressed liquefaction features, which usually formed thin cracks in the upper soils, could be

identified, and that if that significant liquefaction developed at depth, the effects of the liquefaction was very likely to be detected. Also, the fact that the ditch cuts through the flank of an ancient beach ridge, which is an area most likely to have exhibited sand boils, is reasonably conclusive evidence that liquefaction features were not prominent anywhere else in the Hollywood area.

Obermeier identified 24 liquefaction features from the 1886 event, and most of these were small sand vents. The number and size of the features found close to (within 200 ft.) the Virginia Tech test holes are listed in Table 6.1. The data show that while small vents developed near four of the test holes, no features were exposed at five other test locations. These circumstances suggest that while the sands may have liquefied, they were not strongly disturbed during the earthquake, and that the present-day densities are probably close to those which existed at the time of the earthquake. It also suggests that the shaking at this site was not strong enough to cause disruption of the ground surface, given the relative thicknesses of the liquefiable and non-liquefiable layers.

In Fig. 6.5, a soil profile along the 9000 ft. ditch is shown with the layers that are potentially subject to liquefaction delineated by cross-hatching. The cross-hatched area corresponds to sand layers within the profile between 5 and 15 ft. (range of source bed identified by Gelinas,

Table 6.1 Geologic Evidence for Liquefaction Near the Hollywood CPT & SPT Test Locations.

Station	# of Features(1) Possibly Induced by 1886 Event	# of Features(1) Probably Induced by 1886 Event	Size of(1,2) Features	# of Features(1) Induced by Pre- 1886 Events	Size of(1,2) Features
W0900	0	0	--	3	L to H
W0100	0	0	--	3	M to L
0515	3	1	S	7	M to H
1475	3	3	S to M	8	M to H
2550	3	1	S to L	9	M to L
3490	1	2	S	7	M to L
4480	0	0	--	9	S to L
5550	0	0	--	3	M to L
6475	0	0	--	6	M to H

(1) Obermeier, (written communication, 1988).

(2) Sandblow Crater Diameter:

S 0 - 3 ft.

M 3 - 6 ft.

L 6 - 10 ft.

H > 10 ft.

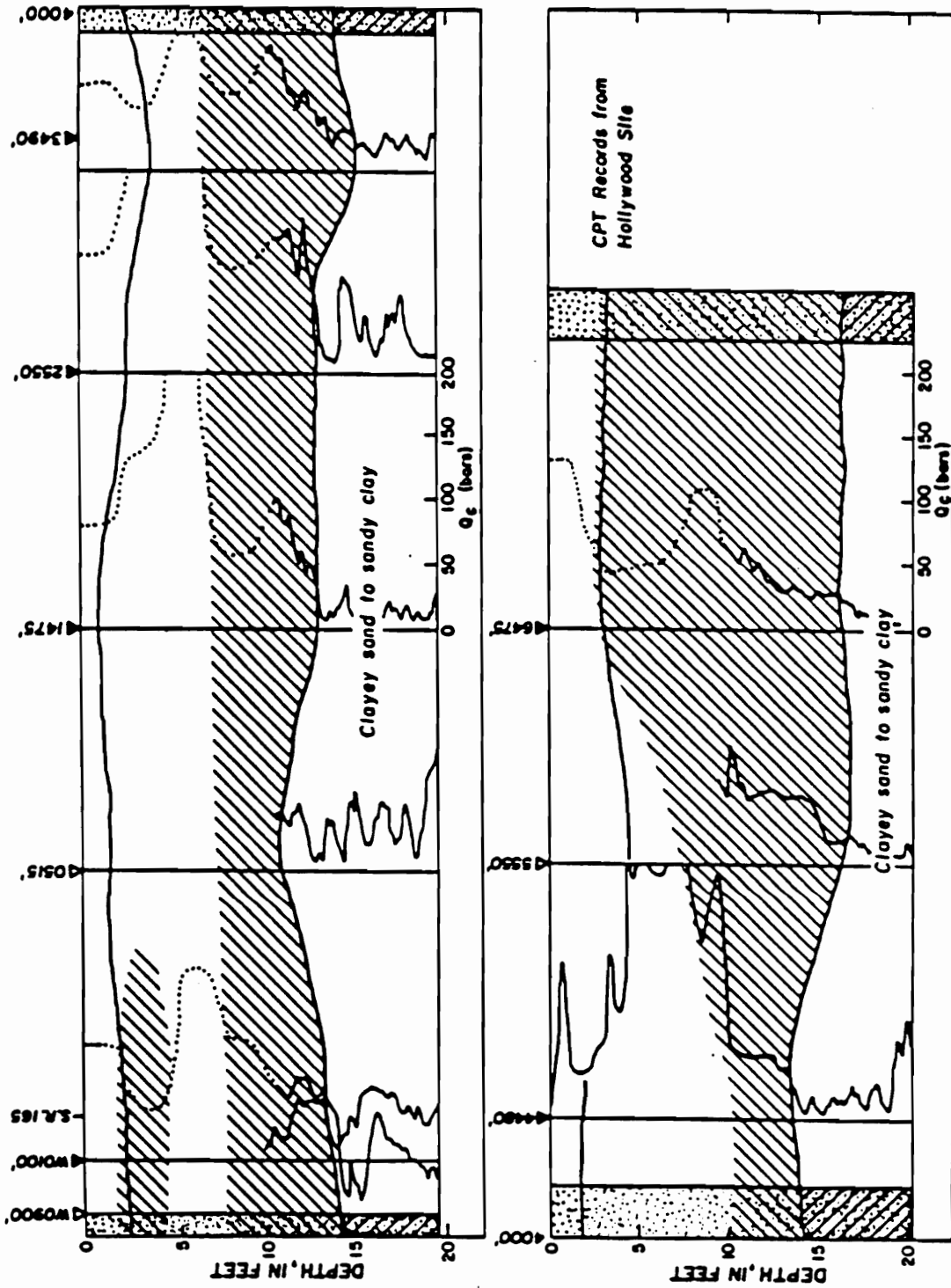


Figure 6.5 Location of Sand Layers Susceptible to Liquefaction Under Seismic Loadings Estimated for the 1886 Earthquake.

(1986)) with blow counts less than 30 blows/ft. It can be seen that the thickness of the liquefiable zone varies from about 5 to 12 ft., and that the non-liquefiable overburden ranges from 4 to 10 ft. in thickness. The relative thicknesses of these layers played a role in determining whether or not liquefaction is likely to be expressed at the ground surface, and this effect must be studied to properly assess the field performance documented at Hollywood. The layer effect was examined using Ishihara's (1985) chart (developed for magnitude 7.7 and 7.8 earthquakes) in Fig. 6.6. The chart was used by plotting the thicknesses of the liquefiable and non-liquefiable layers at each test hole for the Hollywood site. As indicated in the figure, all but two of the 11 test sites plot along the curve corresponded to an acceleration level of 0.3g or less to cause surface disruption by liquefaction. It should be noted that the acceleration curves in the figure do not indicate whether or not liquefaction should develop within the profile, rather they simply indicate what levels of accelerations are necessary to cause surficial ground disruption given the development of liquefaction. The position of the data points in Fig. 6.6 suggested that if the sands at the site are subject to liquefaction at a maximum acceleration level of 0.3g, significant ground disruption would be expected. It thus appears that given the lack of significant ground disruptions at Hollywood in 1886, that either: 1.) the sands

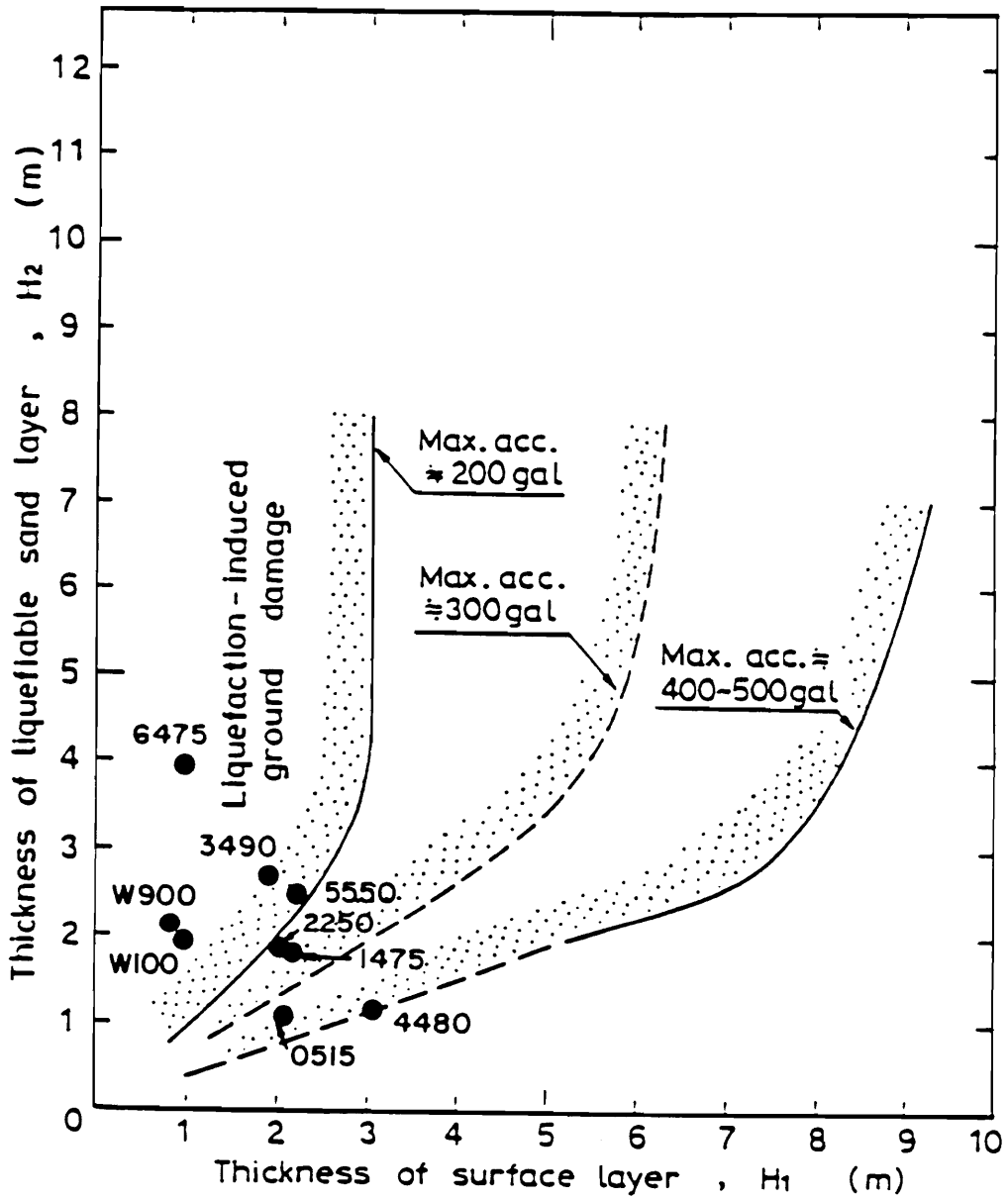


Figure 6.6 Estimated Layer Thickness Relations During the 1886 Earthquake for Each Hollywood Penetration Test Location.

at Hollywood were not liquefiable at 0.3g; 2.) the maximum acceleration level of the 1886 event was less than 0.3g; or, 3.) the 1886 earthquake was less than a magnitude 7.7. The susceptibility of the sands to liquefaction at different acceleration levels is considered next.

Analyses were made for each of the test holes at the site to check for the potential of liquefaction. An example of this is shown in Fig. 6.7, where for an earthquake of $M = 7.5$, cone tip resistances at which liquefaction would theoretically occur were calculated and plotted versus depth for acceleration values of 0.2, 0.3, 0.4, and 0.5g. These liquefaction-prone tip resistances were compared to the cone measurements obtained at STA 3490. In cases where the observed tip resistances were greater than the liquefaction-prone values, then presumably the soil is dense enough to resist liquefaction for the associated value of acceleration. On the other hand, if the measured tip resistances were less than those of the liquefaction prone values, then liquefaction would be expected to occur. The liquefiable layer at STA 3490 was judged to extend from a depth of 7 ft. to 17 ft., with the most susceptible layer at 12 to 16 ft. The overlying cap was assumed 7 ft. thick. In Fig. 6.7, it can be seen that there are zones of sand which were predicted to be liquefiable at acceleration values as low as 0.2g, although complete liquefaction of the liquefiable layer would require accelerations in the range of 0.3g.

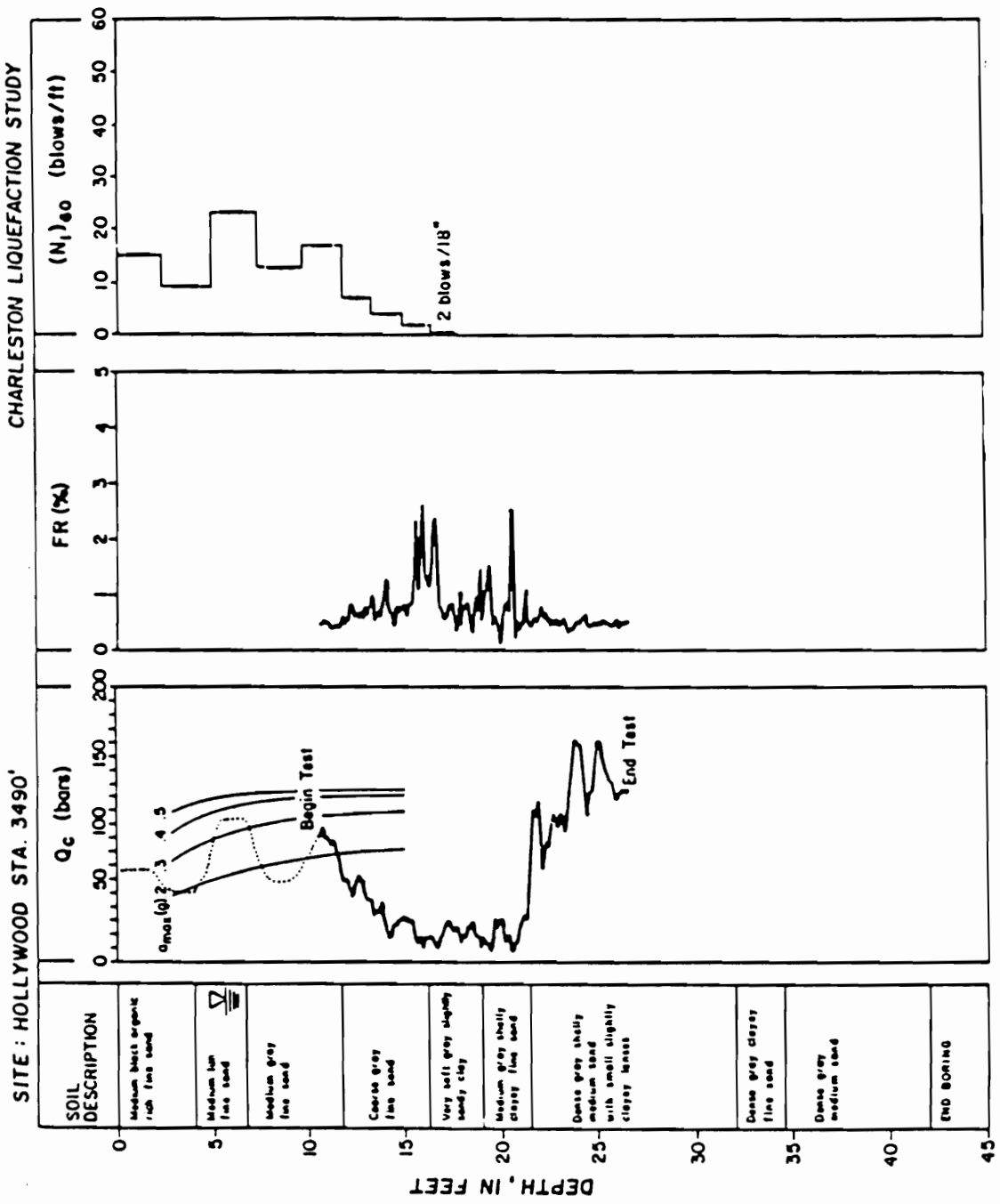


Figure 6.7 Comparison of Actual CPT Resistance at STA 3490' to Those at Which Liquefaction is Predicted For an Earthquake of Magnitude 7.5 and Various Peak Horizontal Accelerations.

Because it was determined earlier that 0.3g was also sufficient to produce significant ground disruptions at this location, it thus appears that if accelerations had been as large as those proposed by the geophysical evidence, 0.5-0.6g, major liquefaction and ground disruption should have occurred. However, Table 6.1 shows that only two small features from the 1886 earthquake were identified in this vicinity. Thus, a difference arises in the level of accelerations that were inferred for this site as obtained by different approaches.

Before considering possible reasons for the findings at STA 3490, it is useful to examine the findings at the other test stations in the same manner. The curves in Appendix A showed that 0.3g should cause liquefaction of the entire sand layer at almost all of the test locations at Hollywood. To illustrate this idea further, all of the CPT and SPT resistance values from the liquefiable portion of the Hollywood soil profile (cross-hatched region of Fig. 6.5) were combined and plotted in Fig. 6.8. The plot shows what percentage of the resistance values would be subject to liquefaction at different levels of ground acceleration for earthquakes of magnitude 6 and 7.5. In constructing the plot, the CPT tip resistances were converted to equivalent SPT values with a Q_c/N factor equal to 4.5 as recommended in the literature. The diagram shown in Fig. 6.3 was then used

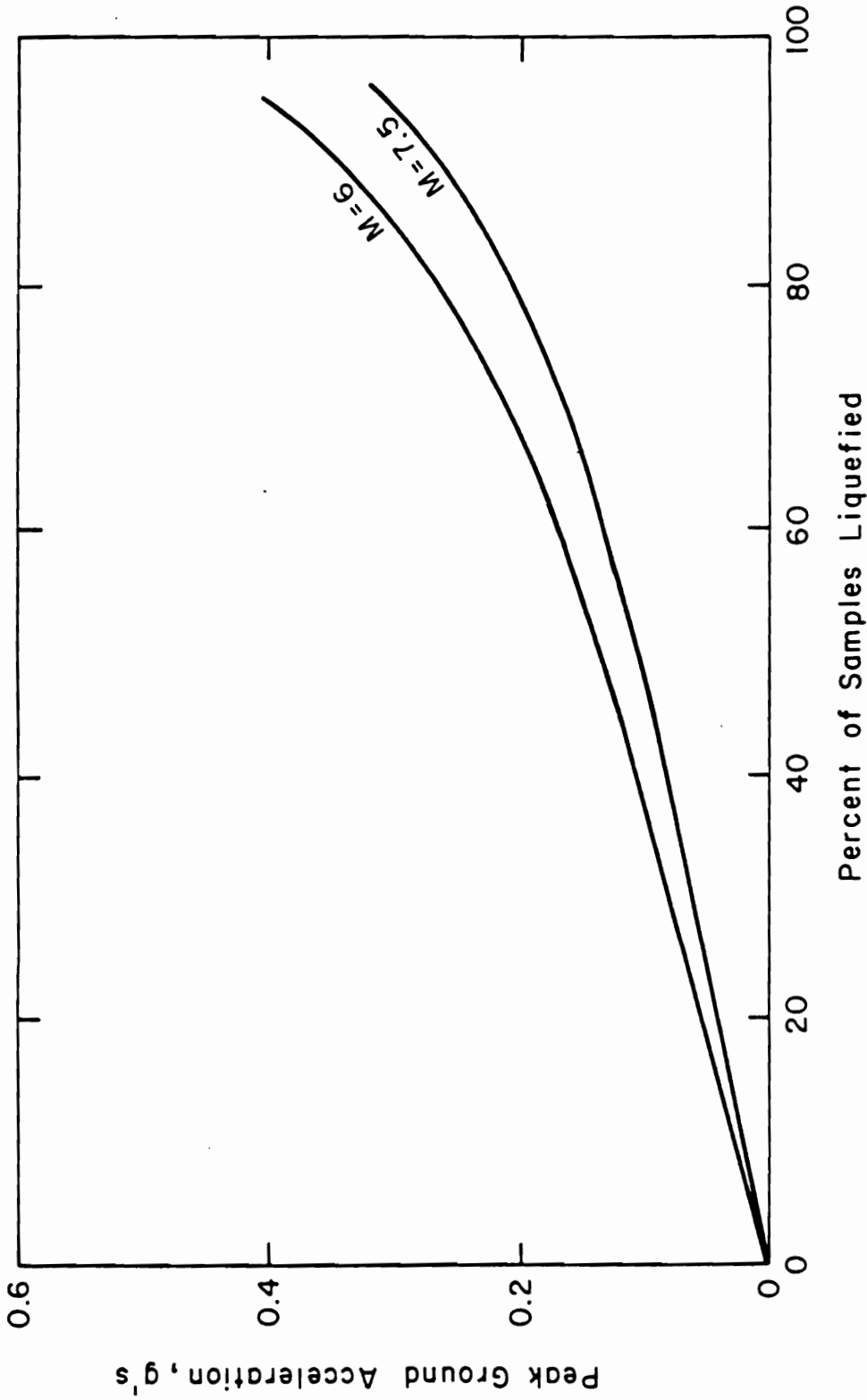


Figure 6.8 Plot Showing Percent of Liquefaction-Prone CPT and SPT Resistance Values From Liquefiable Layer at Hollywood Site for Various Acceleration Levels for $M = 6$ and 7.5 Earthquakes. Data Includes 43 CPT & SPT Samples With $QC/N = 4.5$.

to determine the acceleration values at which liquefaction would occur.

Examining Fig. 6.8, it can be seen that for an $M = 7.5$ earthquake, an acceleration level of about $0.3g$ is sufficient to liquefy 95% of the samples. Interestingly, with an $M = 6$ earthquake, the acceleration level required to cause the same amount of liquefaction is about $0.40g$. The fact that higher accelerations are needed to cause liquefaction for the $M = 6$ event compared to the 7.5 event is consistent with the fact that the duration of shaking increases with earthquake magnitude. Earthquakes of size $M = 6$ have been statistically shown to have about 5-6 significant cycles of shaking (Seed and Idriss, 1982), while an $M = 7.5$ event is estimated to have about 15 cycles of significant shaking. The number of significant cycles of shaking plays an important role in the cyclic behavior of soils. For an increased number of cycles, liquefaction would be expected to occur at lower cyclic stress ratios (and maximum acceleration levels). On the other hand, if the number of cycles is reduced, then it would be expected that a higher cyclic stress ratio (and maximum acceleration) would be required to cause liquefaction. Liquefaction of 95% of the samples implies that liquefaction would develop almost everywhere within the cross-hatched region in Fig. 6.5, and as indicated previously from Ishihara's charts, this amount of liquefaction would be expected to cause

significant ground damage. The exception to the rule is STA 4480 where it was shown in Appendix A that an acceleration higher than 0.5g would be required to induce ground damage. This is related to the fact that the liquefiable layer is only 4 ft. thick at this station. The paleoseismic evidence supports the fact that this location is less likely to display liquefaction features in that no 1886 liquefaction-related features were observed near this station, and in previous earthquakes, this area yielded smaller features than the other stations.

Based on the physical evidence at Hollywood and the data presented thus far, it would appear then that for a M = 7.5 event with about 15 cycles of significant shaking, accelerations no higher than 0.3g would suffice to explain the minor ground disruptions at the site. For a magnitude 6 event with a reduced number of cycles (5-6), a maximum acceleration level in the range of 0.4g would be consistent with the 1886 field performance. It should be pointed out, however, that these estimates were based on the accelerations sufficient to cause liquefaction in 95% of the samples within the cross-hatched region. Intuitively, these estimates are thought conservative in that it is not expected that 95% of the cross-hatched zone would have to liquefy to explain the minor ground disruptions observed by Obermeier. The actual acceleration level more logically would lie between the acceleration level required for

liquefaction of a significant percentage of the samples (0.2g for $M = 7.5$) and that required for significant ground disruption (0.3g for $M = 7.5$). It is difficult however, to determine this acceleration, especially considering that Ishihara's relationships make no provision for estimating accelerations for cases leading up to the situation where the entire liquefiable layer is liquefied and significant ground disruption results. There is also the question as to the meaning of a certain percentage of the penetration samples from within a particular layer liquefying (i.e., abscissa of plot in Fig. 6.8). That is, what percentage of the samples within the liquefiable layer would have to liquefy in order to produce ground disruption at the surface? An approach that can be used to address this issue is outlined in Appendix D. This involves finding the acceleration at which the Ishihara chart calls for surface disruption and comparing that with the percentage of the site test locations that are predicted to have liquefied at that acceleration. In this work, if the percentage of samples predicted to liquefy exceeds at least 50%, then the surface disruption acceleration was accepted as likely to have controlled behavior. If less than 50% of the samples were predicted to liquefy, then it was assumed that the acceleration at the site cannot be back-calculated with confidence, although upper and lower limits can be established in some cases. Reasons for these choices are

described in Appendix D. The process was felt to conservatively define the likely acceleration level. Using this procedure, it was shown in Fig. 6.9 for the Hollywood site, that minor ground disruption would be expected at an acceleration level of about 0.24g for an M = 7.5 earthquake, and at about 0.28g for an M = 6 event; these estimates are slightly less than the previous 0.3g and 0.4g estimates. These accelerations would correspond to the situation where about 85% of the samples from the cross-hatched layer were liquefied. The fact that ground disruption is expected to occur at comparable acceleration levels for both the M = 6 (5-6 significant cycles) and M = 7.5 (~15 significant cycles) events is reflective of the fact that the soils are liquefiable at a low number of seismic loading cycles. In cases where the soils are more resistant to liquefaction, the difference between the results from the M = 6 and M = 7.5 analyses would be greater.

The analysis procedures presented for the Hollywood site were also adopted for the investigation of the remaining sites, although less complete field data is available for most of these areas. The analyses of these sites, described subsequently, are summarized in Table 6.2.

6.3.2 Warren Site

Documentation of the conditions at the Warren site was not as complete as that at the Hollywood site, neither in

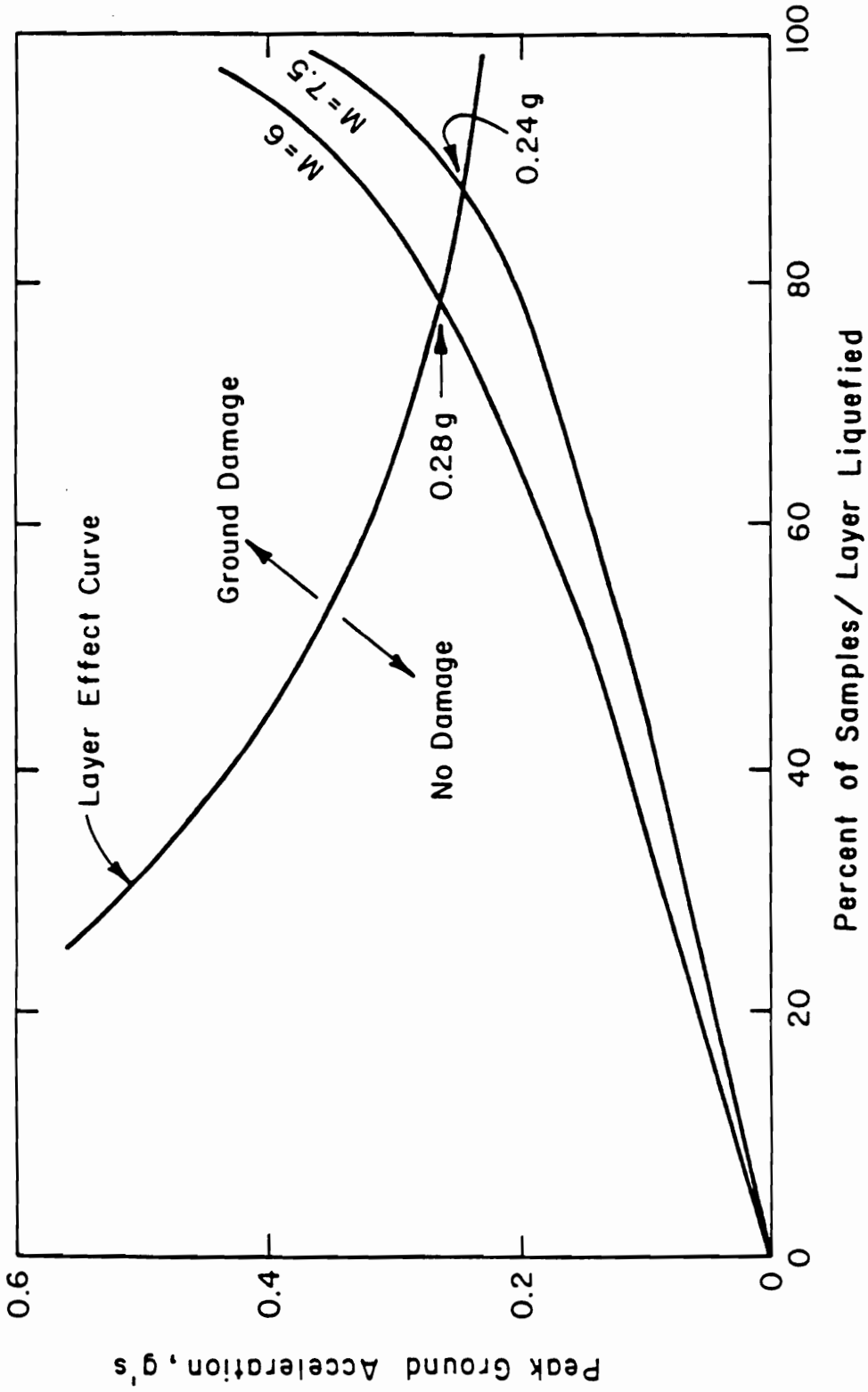


Figure 6.9 Refined Estimates of Threshold Accelerations Required to Cause Ground Disruption at Hollywood Site Based on Percent of Samples Liquefied and Relative Thicknesses of Liquefiable and Non-Liquefiable Layers (43 CPT & SPT Samples).

Table 6.2 Summary of Liquefaction Analyses of SPT & CPT Data From Virginia Tech Test Sites at Various Distances From 1886 Zone of Energy Release

Site	Distance From Energy Release (mi.)	Extent of Liquefaction	# of SPT/CPT Samples (3)	Threshold (4) Accelerations (g's) m=6	I of Samples Liquefied	Threshold (4) Accelerations (g's) m=7.5	I of Samples Liquefied	Probable (5) 1886 Accels if m=6	Probable (5) 1886 Accels if m=7.5
Sod Farm	4	Minor (1)	7	0.40	100%	0.30	100%	0.40	0.30 (6)
Hollywood	5	Moderate (1)	43	0.28	80%	0.24	85%	0.28	0.24 (6)
Ten Mile	5	Major (2)	7	0.23	40%	0.19	56%	> 0.23	> 0.19
Eleven Mile	5	Moderate (2)	6	0.25	100%	0.20	100%	0.25	0.20
Airport/AFB	5	Major (2)	89	0.29	35%	0.24	46%	> 0.30	> 0.25 (6)
Montague	6	Minor (1)	4	0.34	80%	0.28	100%	0.34	0.28 (6)
Warren	6	Minor (1)	11	0.31	70%	0.27	78%	0.31	0.27
Five Mile	10	Minor (2)	18	0.29	48%	0.24	56%	0.29	0.24
St. Michael's	13	Minor (2)	14	0.22	50%	0.18	60%	0.22	0.18 (6)
Manigault	13	Minor (2)	4	0.30	100%	0.25	100%	0.30	0.25
Mt. Pla. Pits	15	None (1)	42	0.20	76%	0.18	84%	< 0.20	< 0.18 (6)
Mt. Pla. Hall	15	Minor (1)	20	0.21	40%	0.18	50%	0.21	0.18 (6)
H. Clark Br.	18	None (2)	8	0.25	42%	0.20	50%	< 0.25	< 0.20
Oakland Plan. (entire site)	19	V. Minor (1)	16	0.26	80%	0.23	90%	< 0.26	< 0.23 (6)
Oakland Plan. (at sand vent)		Minor (1)	6	0.18	50%	0.15	60%	0.18	0.15 (6)
Edisto Isl.	19	Minor (2)	63	0.18	30%	0.14	35%	< 0.25	< 0.20
S. Tibwin/	34	None (1)	45	0.28	45%	0.22	50%	< 0.28	< 0.22 (6)
St. James Sch.									
Georgetown	56	None? (2)	9	0.32	45%	0.27	50%	< 0.32	< 0.27
Pawley's Isl/ Parkersville	65	None? (2)	26	0.33	46%	0.28	50%	< 0.33	< 0.28

(1) From Paleoseismic Investigation; (2) From Historical Observations; (3) From Within Liquefiable Layer, $Q_c/N = 4$ to 5; (4) Peak Horizontal Accelerations Required for Surficial Ground Disruption; (5) Based on 1886 Observations and Criteria Discussed in Appendix D. Values Plotted in Figs. 6.22 & 6.23; (6) Based on Excellent data.

terms of the knowledge of the past liquefaction at the site nor in terms of the amount of exploration performed there. Known liquefaction features at this site consisted only of one significant sand blow. The soil conditions in the upper 15 ft. as determined from the three CPT's and five auger holes at the site were similar to those at Hollywood. From the available evidence it appeared that the density of the sands in close proximity to the vent should have been changed significantly by the 1886 event, while sands located away from the isolated vent might be approximately the same as that which existed in 1886.

The soil conditions at the Warren site in terms of liquefaction resistance were more complex than at the Hollywood site. Typically, the liquefiable layers were thinner than at the Hollywood site, and more of the overburden is non-liquefiable. As indicated in the logs in Appendix A for the Warren site, the densities of some of the layers were low enough to theoretically liquefy at accelerations as low as 0.2g. However, as shown in Fig. 6.10, accelerations would have to approach 0.3g to cause surface disruption because of the relative thicknesses of the liquefiable and non-liquefiable layers. The implications of the field performance at this site upon the accelerations of the 1886 event were somewhat moot since several interpretations are possible as to the meaning of the physical evidence.

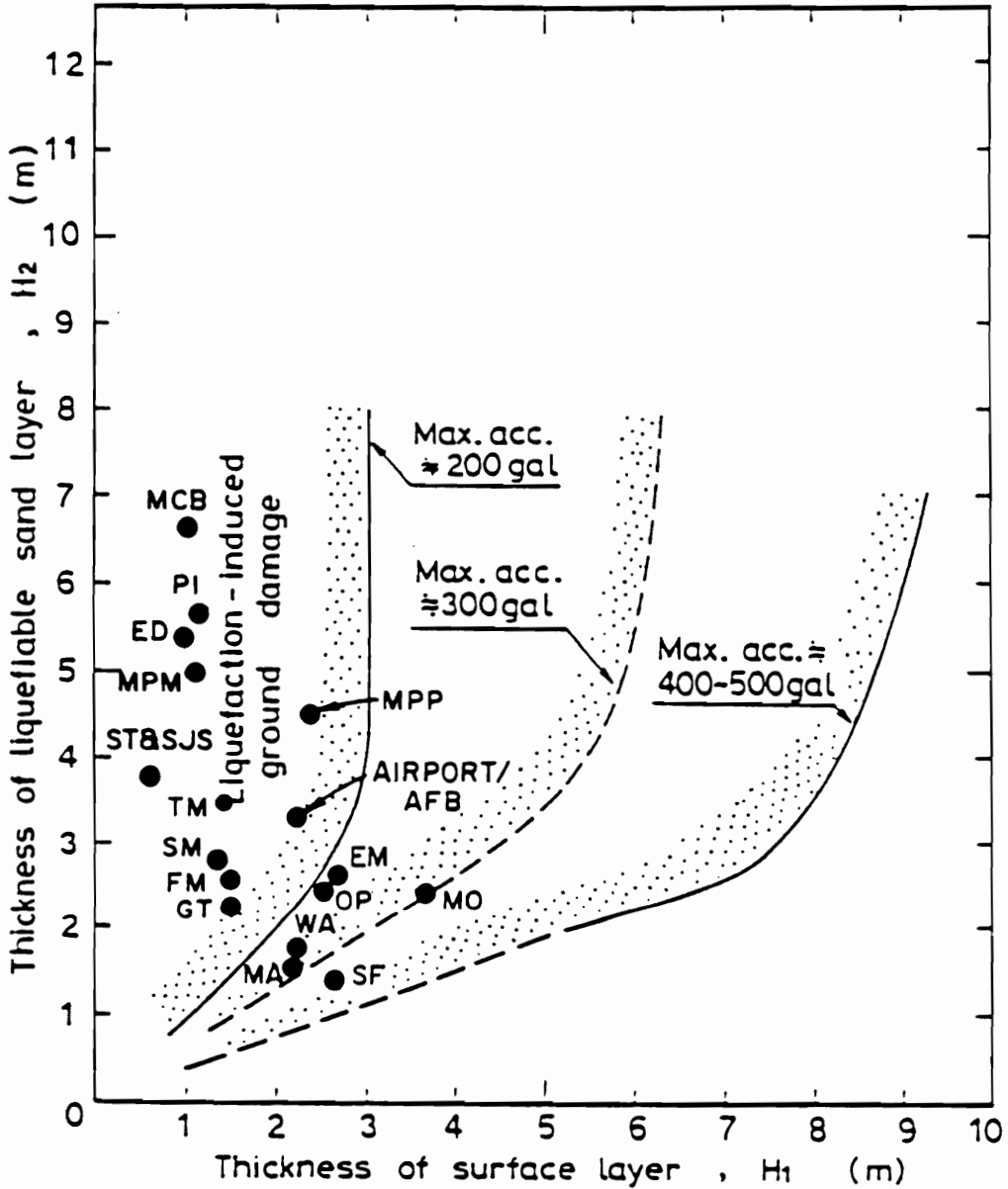


Figure 6.10 Estimated Layer Thickness Relations During the 1886 Earthquake for Each VA Tech Test Site Except Hollywood.

It may be remembered from Chapter 5 that the paleoseismic evidence on the mineralogy of the sandblow material implies that ejected material come from depths of 14 to 17 ft. (Cox, 1984; Gelinas, 1986). This is supported by the fact that angular clay fragments from the clay layer at 10 ft. were found in the sandblow ejecta. The CPT logs demonstrate that if the silty sands at depths below 13 ft. liquefied during the 1886 event, then extensive densification of this layer occurred. This is considered possible in light of the proximity of Warren 1 and 2 penetration tests to the location of the large liquefaction vent. It is instructive to note, however, that the Warren 3 CPT, located approximately 200 ft. away from the vent, also indicated that the lower sands were dense. This would appear to indicate that soil conditions at other areas across the site were not greatly susceptible to liquefaction. Given the fact that only one liquefaction vent was found at the site, it was postulated that the location of the vent feature was probably an area of isolated loose material which liquefied during the 1886 event, and subsequently densified. Further, it would be expected that if accelerations in 1886 had been as high as the 0.4-0.5g necessary to liquefy the lower sands and cause significant ground disruption today, that liquefaction and ground disruption would have occurred at other locations at the site.

6.3.3 Sod Farm and Montague Sites

As mentioned previously, the Sod Farm and Montague sites were tested with one or two CPT's in order to obtain a preliminary assessment of the soil conditions at the sites. The limited test coverage allows for a tentative evaluation of the sites, but does not allow for the extent of the loose sand layers at the sites to be determined. In this case, it was possible for localized zones of very loose material in the vicinity of the relic liquefaction features to go undetected.

Sod Farm Site

The soil profile at Sod Farm is similar to that at the Hollywood site. The sand layers show a decreasing density with depth until a soft clay layer is reached. As at the Hollywood site, the CPT data at the Sod Farm suggested that some of the sands were readily liquefiable, and would liquefy at accelerations as low as 0.2g. The most susceptible layer was about five ft. thick, and was overlain by 10 ft. of non-liquefiable overburden. Superimposing these layer thicknesses onto the Ishihara chart in Fig. 6.10 showed that accelerations would have to reach values of 0.3 to 0.4g to induce ground disruption. The lack of evidence for extensive liquefaction features at this site supported the idea that the accelerations of the 1886 earthquake may

not have been as high as those proposed by geophysical evidence (0.5-0.6g).

Montague Site

The one CPT performed at the Montague site showed that the soil at the location where the penetration test was made was denser than that found at the Hollywood and Sod Farm sites, and the accelerations required to cause liquefaction at this site were likely higher than those seen at the other sites. Liquefaction at this site was predicted as marginal at accelerations in the range of 0.3g, and that accelerations of 0.3g would be required to initiate ground disruption (Fig. 6.10). Thus, this acceleration level would be consistent with the development of the minimal liquefaction features found at the site by Obermeier. The results could not be used however, to shed light on the possibility of low levels of accelerations at the site since it was not in a marginal liquefaction environment.

6.3.4 Ten Mile Hill, Eleven Mile Post, and Airport Sites

The Ten Mile Hill area was different from the other sites, in that extensive liquefaction was known to have occurred there during the 1886 event, and, although field reconnaissances have been performed there by Obermeier, fewer relic liquefaction features were identified in this

area as compared to Hollywood. This is due mainly to the fact that ground exposures like those found at Hollywood were generally not available in this area. Thus, the majority of what is known about the extent of the liquefaction at Ten Mile Hill was based on the historical observations there.

Ten Mile Hill and Eleven Mile Post Sites

The soil profiles at the Ten Mile Hill and Eleven Mile Post sites were unique in that there was little in the way of a non-liquefiable layer in the upper part of the profile. The near-surface soils at these sites appeared susceptible to liquefaction at acceleration levels of 0.2 to 0.3g, and, as plotted on the Ishihara chart in Fig. 6.10, both of these sites should suffer significant ground damage at these accelerations. One possibility was that these upper soils may have been disrupted by the active sand boiling that occurred at the site, and their present-day densities could be lower than those which existed at the time of the 1886 earthquake.

It is instructive to note that following the earthquake, eyewitnesses in the Ten Mile Hill area reported seeing mica fragments in the ejected sand boil materials there. Also, several of the sand boils were plumbed, and it was found that the soft material was as deep as 12 ft. In

the auger borings conducted at Ten Mile Hill for the present study, mica was found abundant at depths of 12 to 14 ft. Thus, it would seem that for the eyewitness accounts to be true, liquefaction would have occurred below 14 ft., although the sand layers below this point today are dense and resistant to liquefaction. This suggests that either the lower sand layers densified as a result of the earthquake, or the liquefaction occurred higher in the profile, but somehow was able to disrupt the soils below. It appears that the former situation is more likely than the latter.

Charleston International Airport/ Air Force Base

As described in Chapter 5, the soil conditions at the Airport are variable and consist of areas with continuous loose to medium sands and other areas where interbedded clay layers are found near the top of the profile. The 50 SPT borings that were obtained from the site are used to evaluate the liquefaction potential of the sandy soils there.

The majority of the borings revealed an upper 7 ft. of non-liquefiable overburden, underlain by 10 to 12 ft. of soils that were liquefiable. At several locations, however, the non-liquefiable overburden was less than 7 ft. thick, and the liquefiable layer was greater than 12 ft. in thickness. In using the Ishihara chart, it was assumed that

all locations had at least 7 ft. of non-liquefiable overburden and about 11 ft. of underlying liquefiable soil. Hence, by adopting the profile which is the most unfavorable for producing ground disruptions, the 1886 earthquake accelerations predicted for this site would be upper bound values for some locations at the site.

The plot in Fig. 6.11 shows that for an $M = 7.5$ event, ground disruption should develop at accelerations in the range of $0.25g$. For $M = 6$, the acceleration level increases to about $0.30g$. Given the extensive liquefaction that developed in this area during the 1886 event, it is estimated that the maximum acceleration levels during the quake were at or above these threshold values. However, because liquefaction was extensive at the site in 1886, some densification of the sands could have taken place, such that the extent of loose zones of material previously subject to liquefaction was reduced after the 1886 event. This idea would lead to a reduction in the predicted acceleration levels for this site.

6.3.5 Five Mile Site

The ten soil borings obtained at the Five Mile site indicated that although significant thicknesses of loose material were present in the upper 20 ft. of the profile, these materials appeared to have a higher fines content than the soils at the other meizoseismal zone sites. Some of the

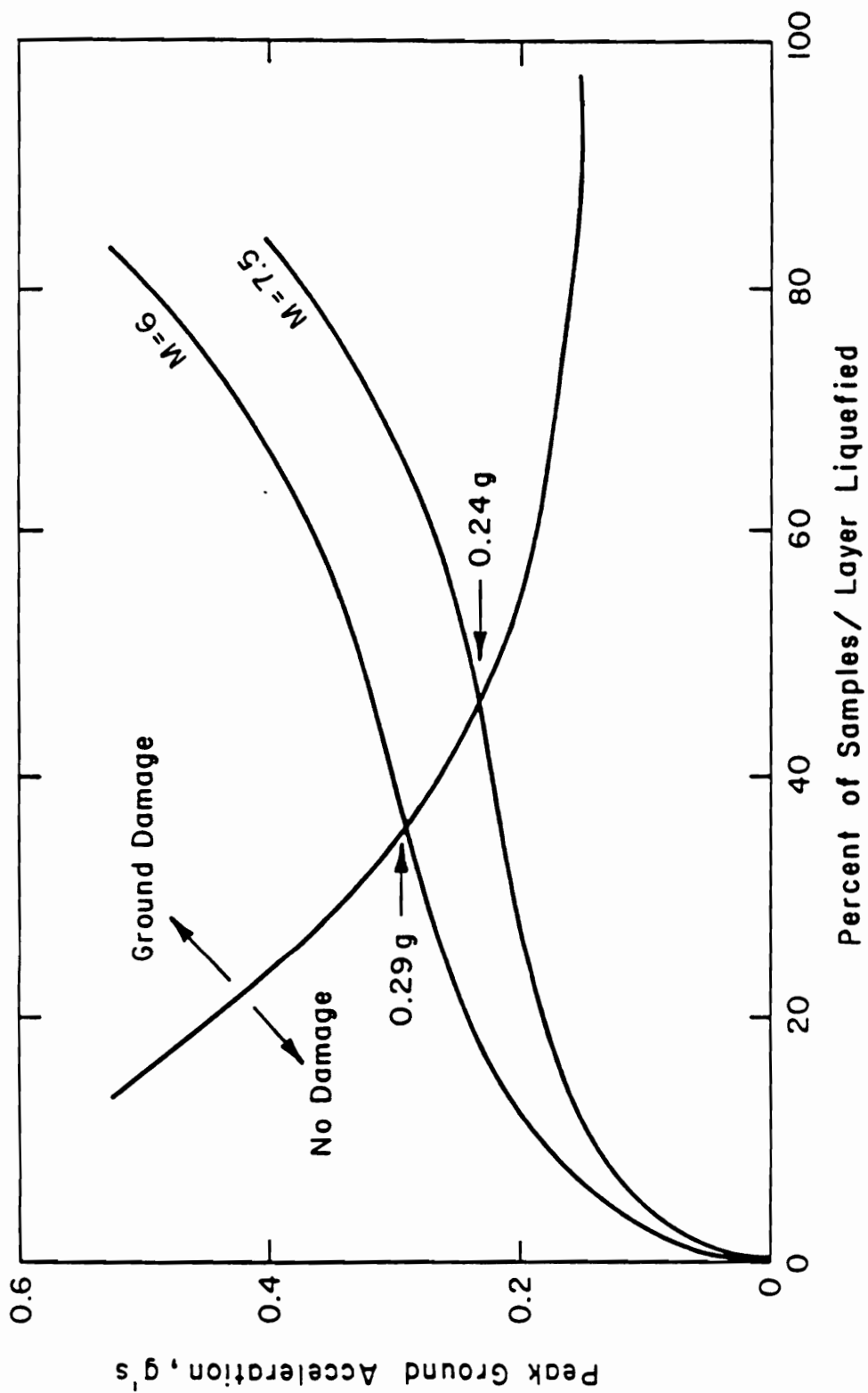


Figure 6.11 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Charleston Airport Site (89 Samples).

borings indicated fine silty sands with no clay present, while other borings revealed silty sands interbedded with clayey sands. This site was also unique in that it was located close to the 1886 meizoseismal zone boundary, and was situated farther away from the zone of energy release than the other sites investigated within the meizoseismal zone.

The liquefaction potential of this site was evaluated on the basis of the SPT blowcount data obtained there. The blowcounts analyzed were those obtained from the liquefiable layer at each test location. Generally, the liquefiable layer extended from about 5 ft. to a depth of 13 ft.

In examining the borings from this site, all sand layers that revealed clayey sands were considered non-liquefiable and were excluded from the data set. In cases where clayey sands were underlain by liquefiable material, the clayey sands were treated as non-liquefiable overburden. The average non-liquefiable cap thickness was about 5 ft., and the average liquefiable layer thickness was approximately 8 ft.

The SPT data from all of the test holes were combined and shown in Fig. 6.12 which indicates that for a magnitude 7.5 event, ground disruption was expected to occur for accelerations in the range of 0.25g. For a magnitude 6 event, the acceleration level required to produce the same amount of liquefaction is about 0.3g. Given the fact that

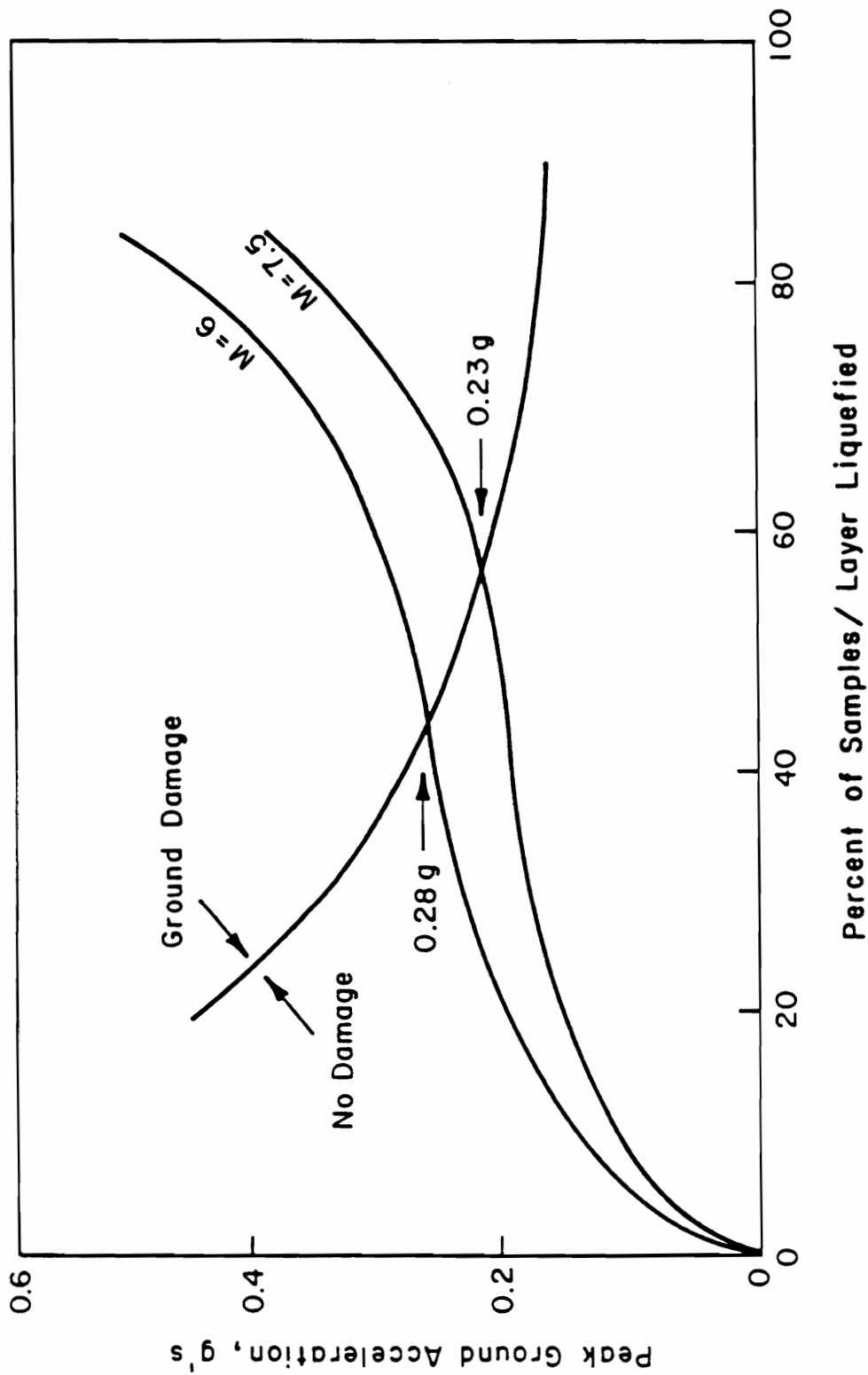


Figure 6.12 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Five Mile Site (18 Samples).

the historical accounts indicate only minor sand boiling at this location, it appears that the 1886 accelerations at this site did not exceed these threshold values.

6.3.6 Analysis Of Data Obtained From Outside Sources

The soil data collected from outside agencies and described in Chapter 5 were used to indicate what levels of ground accelerations would be necessary to produce significant liquefaction at various locations throughout the meizoseismal region. Table 6.3 lists the sites from which data were obtained and includes key site characteristics and soil information. Each site indicated in the table represents a composite of many SPT logs. This localized averaging of the soil conditions is considered appropriate in that the liquefaction behavior of the soils within the meizoseismal zone was best characterized by their average susceptibility. That is, because the meizoseismal zone was an environment of significant 1886 liquefaction activity, it would be improper to characterize the sites within this region on the basis of the lowest penetration resistances or thickest liquefiable layers. Such a characterization would lead to an underestimate of the accelerations required to produce the amount of liquefaction observed in 1886.

Table 6.3 also lists the acceleration levels at which ground disruptions would be expected to develop at each site. The accelerations listed are for the case where, as

Table 6.3 Summary of Liquefaction Analyses of SPT Data Obtained From Outside Agencies
For Sites Within the 1986 Haisoosaiamaal Zone

Site	Distance From Energy Release (mi.)	Extent of Liquefaction	φ of SPT/CPT Samples	Avg M _{1,60} (Blows/ft.)	Avg Thickness (3)	Liquefiable Layer	Avg Thickness (ft.)	USCS (4)	Threshold (5)		% of Samples Liquefied
									Accelerations (g's) m=6	Accelerations (g's) m=7.5	
MZ-2	7	None	6	13	17	4	SH	0.26	0.20	50%	
MZ-6	13	Minor	14	16	14	8	SH	0.23	0.20	50%	
MZ-7	12	None	7	19	9	8	SH	0.21	0.19	60%	
MZ-8	8	Minor	40	17	12	10	SH	0.36	0.30	58%	
MZ-10	1	Minor	20	16	12	7	SH	0.28	0.26	88%	
MZ-21	5	Minor	10	23	7	3	SH	0.35	0.28	50%	
MZ-23	7	None	4	11	6	5	SP-SH	0.30	0.26	73%	
MZ-47	5	Mnr. to Mod. V. Minor	8	8	> 4	10	SP-SH	0.32	0.28	64%	
MZ-B	7		13	17	9	4	SH	0.20	0.17	60%	
MZ-C	4	None	10	16	8	5	SP-SH	0.23	0.20	61%	
MZ-D	4	None	10	20	9	5	SH	0.32	0.26	52%	
MZ-I	5	None	13	17	3	6	SH	0.23	0.20	62%	
MZ-L	6	None	25	16	5	5	SH	0.26	0.22	68%	
MZ-O	2	Minor	13	25	11	6	SP	0.22	0.18	70%	
MZ-P	2	Minor	4	18	11	7	SH	0.26	0.22	58%	
MZ-Q	> 1	Minor	10	22	7	12	SP-SC	0.42	0.36	78%	
MZ-R	7	V. Minor	16	18	7	12	SP	0.18	0.16	58%	
MZ-S	5	Minor	35	16	> 17	3	SP	0.38	0.32	52%	
MZ-T	11	None	2	17	5	7	SH	0.40	0.35	100%	
MZ-V	4	Minor	18	21	7	5	SH	0.22	0.18	52%	

(1) Based on Historical Observations in General Vicinity of Site.; (2) From Within Liquefiable Layer; (3) Includes Correction for Fines Content; (4) Based on Visual Classification; (5) Estimated Peak Accelerations Required for Surficial Disruption (see Appendix D). Values Plotted in Figs. 6.22 & 6.23.

explained for the Hollywood site, it was assumed that the liquefaction was initiated immediately below the non-liquefiable overburden. Examining the data, it can be seen that most of the sites are shown to liquefy at acceleration levels in the 0.2-0.3g range for $M = 7.5$ events, and 0.3-0.4g for $M = 6$ events. When the locations of these sites are compared to the map of the historical observations given in Fig. 2.3, the data collectively support the findings of the field work at the other sites. That is, the documented observations of liquefaction activity in 1886 can be explained by acceleration levels in the 0.3g range for most of the meizoseismal region.

6.4 SITES OUTSIDE THE 1886 MEIZOSEISMAL ZONE

ANALYSIS OF VIRGINIA TECH SITES

6.4.1 Oakland Plantation

In terms of significance to this study, the Oakland Plantation site is second only to Hollywood in that it represents an area away outside of the epicentral region where the presence of thick zones of readily liquefiable material are present, and where thorough paleoseismic investigations have been performed. The borings and penetration tests at the site revealed that the upper 30 ft.

of the soil profile was dominated by clean fine sands in a very loose to medium condition. In terms of conventional liquefaction analyses for an earthquake the size of the 1886 event, these sands were obviously susceptible to liquefaction at very low levels of ground acceleration. Fig. 6.13 shows a soil profile from the site developed from several of the test holes performed there. The most obviously susceptible layer was located from about 8 to 16 ft., as blowcounts in this layer were consistently below 10 blows/ft. Also, it was noted by Obermeier that the sands in the liquefaction feature were free of shells which are present at depths from 15 to 20 ft. This would suggest that the source sands were above 15 ft., and consistent with the position of the layer at 8 to 16 ft.

Fig. 6.14 indicates that essentially all of the samples within the layer at 8 to 16 ft. would liquefy at accelerations in the 0.20g range for a magnitude 7.5 event, and at about 0.25 g for an $M = 6$ event. According to Ishihara's chart in Fig. 6.10, this liquefaction should produce significant ground disruption. The fact that only one very small 1886 feature was found at the site suggested that the ground motions at this site did not exceed the threshold values. It should be mentioned that the penetration tests performed next to the feature revealed the loosest sands at the site. This would be consistent with the fact that Oakland Plantation is in a marginal liquefaction environ-

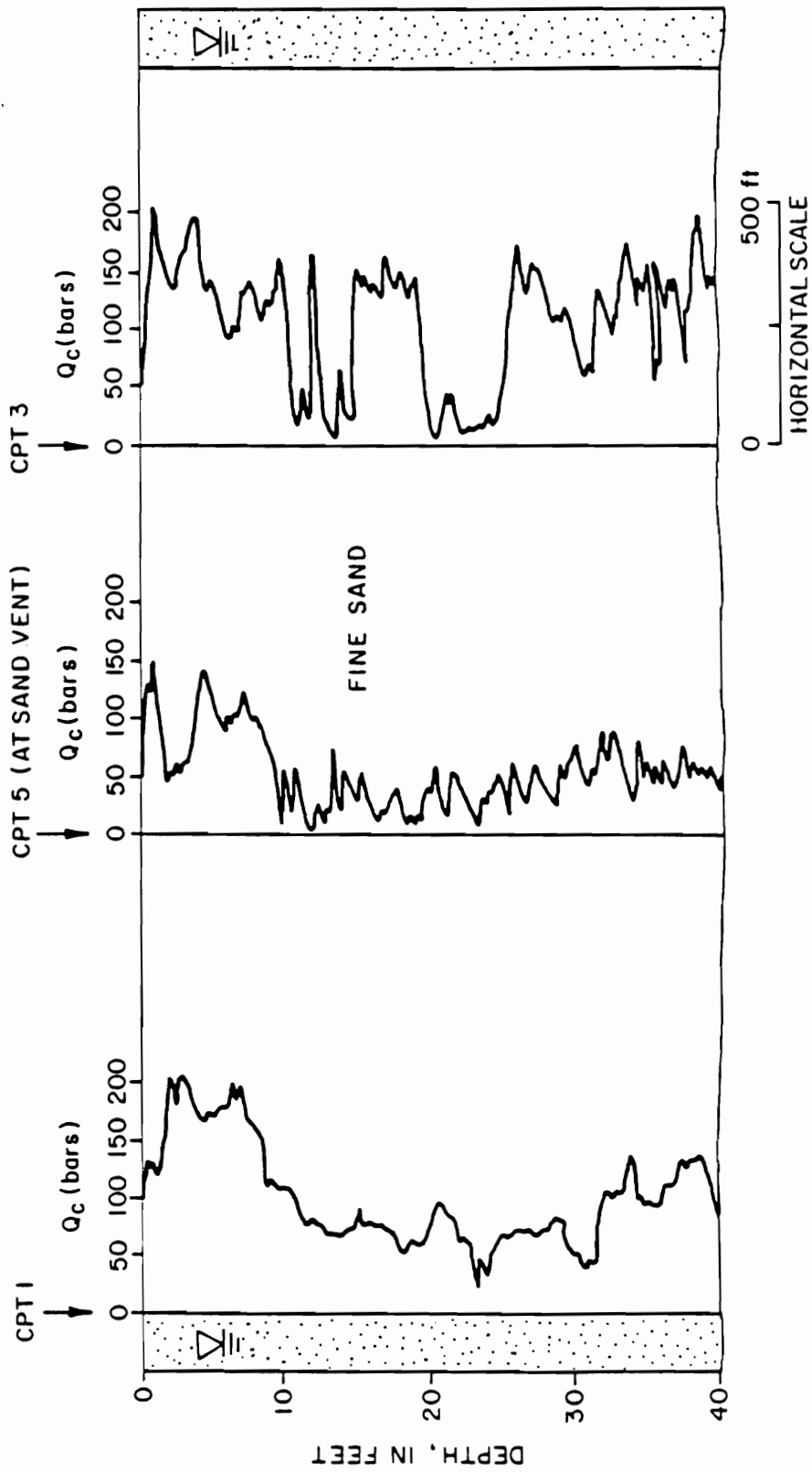


Figure 6.13 Soil Profile Showing CPT Resistances Along the Edge of Dewatered Sand Pit at Oakland Plantation. Note That Position of Water Table Shown is For Dewatered Condition.

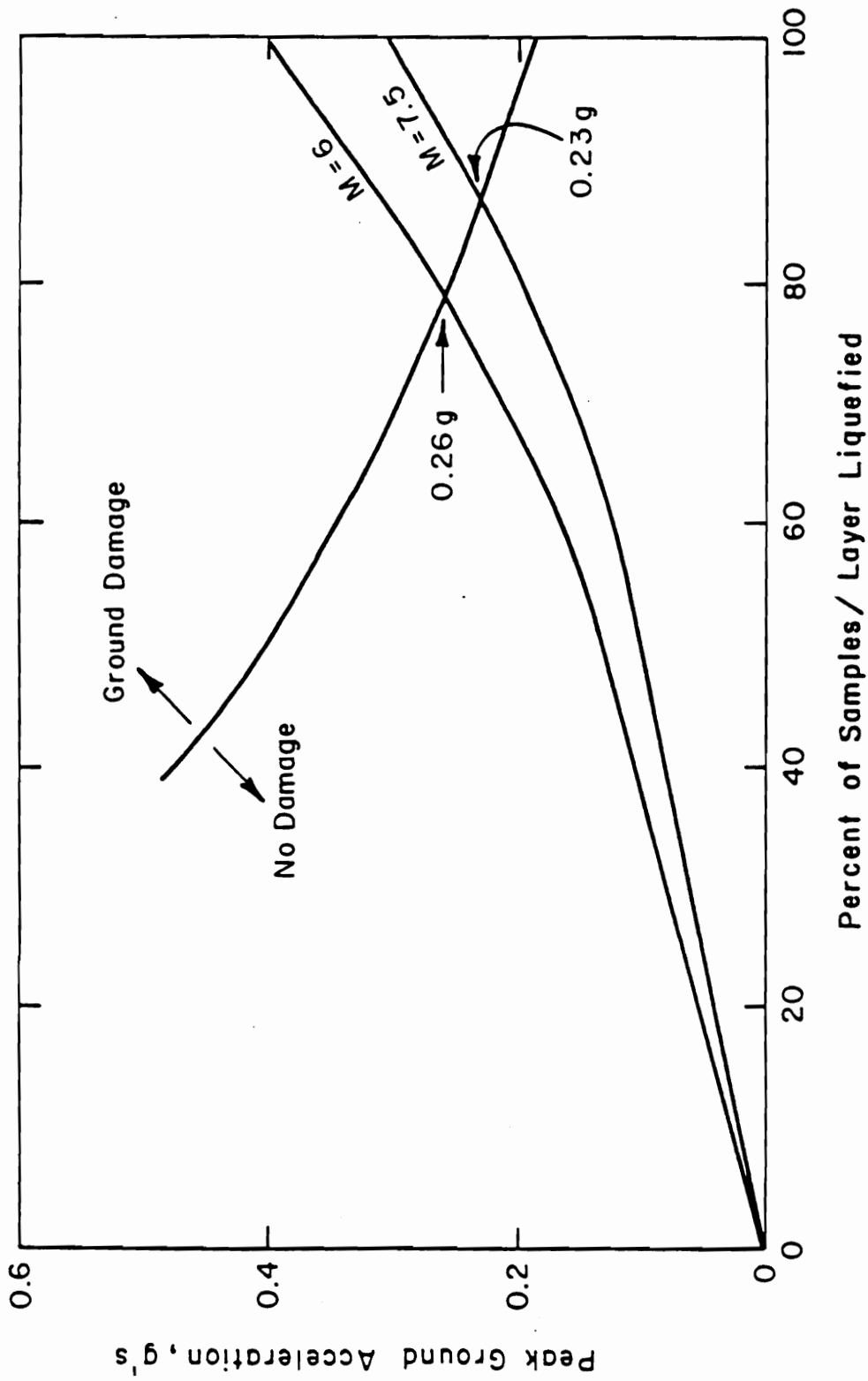


Figure 6.14 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Oakland Plantation Site (16 Samples).

ment. Also, it should be noted that the assessment of 1886 liquefaction activity this site was based on Obermeier's examination of the soil profile from the ground surface to depths of 10 to 12 ft. This discounted the possibility that significant liquefaction took place at depth, but was masked from the near-surface soils. If the acceleration levels at this site are back-calculated solely on the basis of the soil conditions at the location of the vent feature, as opposed to the average conditions across the site (Fig. 6.14 is based on average conditions) the acceleration estimates would be considerably lower; about 0.15g for $M = 7.5$ and 0.20g for $M = 6$.

6.4.2 The City of Charleston

ST. MICHAEL'S CHURCH

The soil profile at St. Michael's Church (Fig. 5.29) revealed the presence of loose to medium sands at the site. The liquefiable layer at the site was located generally between 5 and 15 ft. Normalized blowcounts in this layer were consistently below 15 blows/ft. The physical evidence indicated that marginal liquefaction took place under the church foundation, although eyewitness reports indicated that sand boils were not observed at any locations around the church grounds. The blowcounts from the SPT borings

were used in Fig. 6.15 to illustrate that for a magnitude 7.5 earthquake, liquefaction and ground disruption is expected to occur at acceleration levels as low as 0.2 g. For an M = 6 earthquake, ground disruption would be expected to occur at an acceleration level in the range of 0.25 g. With these estimates and our knowledge of the fact that marginal liquefaction occurred at the site with only minimal damage to the church, the ground accelerations at this site were expected to have not greatly exceeded the 0.2 g. range.

A recent study by Elton and Marciano (1990) which evaluated the structural performance of the church during the 1886 quake, estimated the peak acceleration at the site to be 0.33g. It should be pointed out however, that although this study was based on a fundamental approach, the findings are questionable because no consideration was given to the fact that a significant portion of the building damage suffered at St. Michael's was due to liquefaction of the foundation soils.

MANIGAULT RESIDENCE

The soil profile at the Manigault residence revealed the presence of extremely loose material with blow counts consistently below 3 blows/ft. down to about 12 ft. The upper sands at this site however, had a higher fines content than those typically encountered at the other test loca-

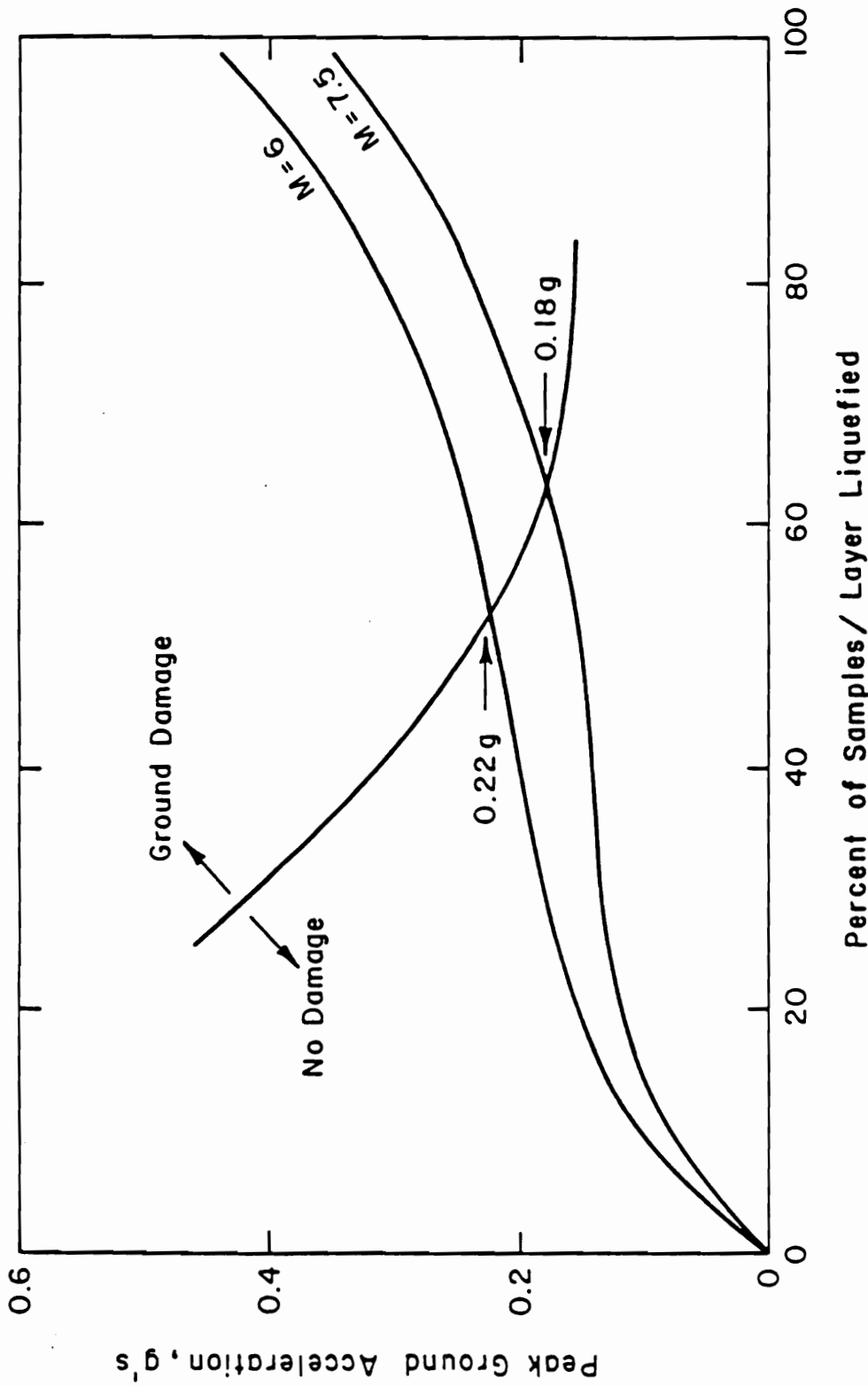


Figure 6.15 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at St. Michael's Church Site (14 Samples).

tions. The soils were classified as silty sands to clayey sands, and where necessary the appropriate corrections for the presence of non-plastic fines were made in the liquefaction analyses. The top 7 ft. of the profile, which consisted primarily of very loose SC material was considered non-liquefiable overburden. Fig. 6.16 shows the boring profile from a location close to the sand boil feature identified by Manigault. Outlined in the figure is the layer of silty sand from 7 to 12 ft. which was the most susceptible to liquefaction. Included in the figure is a table which lists the accelerations necessary to cause liquefaction of the sands as well as those necessary to induce ground disruptions. The table indicated that the silty sand material was predicted to liquefy at accelerations close to 0.1g for magnitude 6 and 7.5 events. Also, for an $M = 7.5$ event, ground disruption would be expected at accelerations of less than 0.25g. Manigault reported however, that no significant sand boils or any other obvious expressions of liquefaction occurred at the site. This would suggest that the 0.25g acceleration level is probably an upper bound for the level of ground shaking at this location. It is expected that accelerations for an $M = 6$ event however, would have to have been closer to 0.30g (from table in Fig. 6.16) to explain the observations at the site.

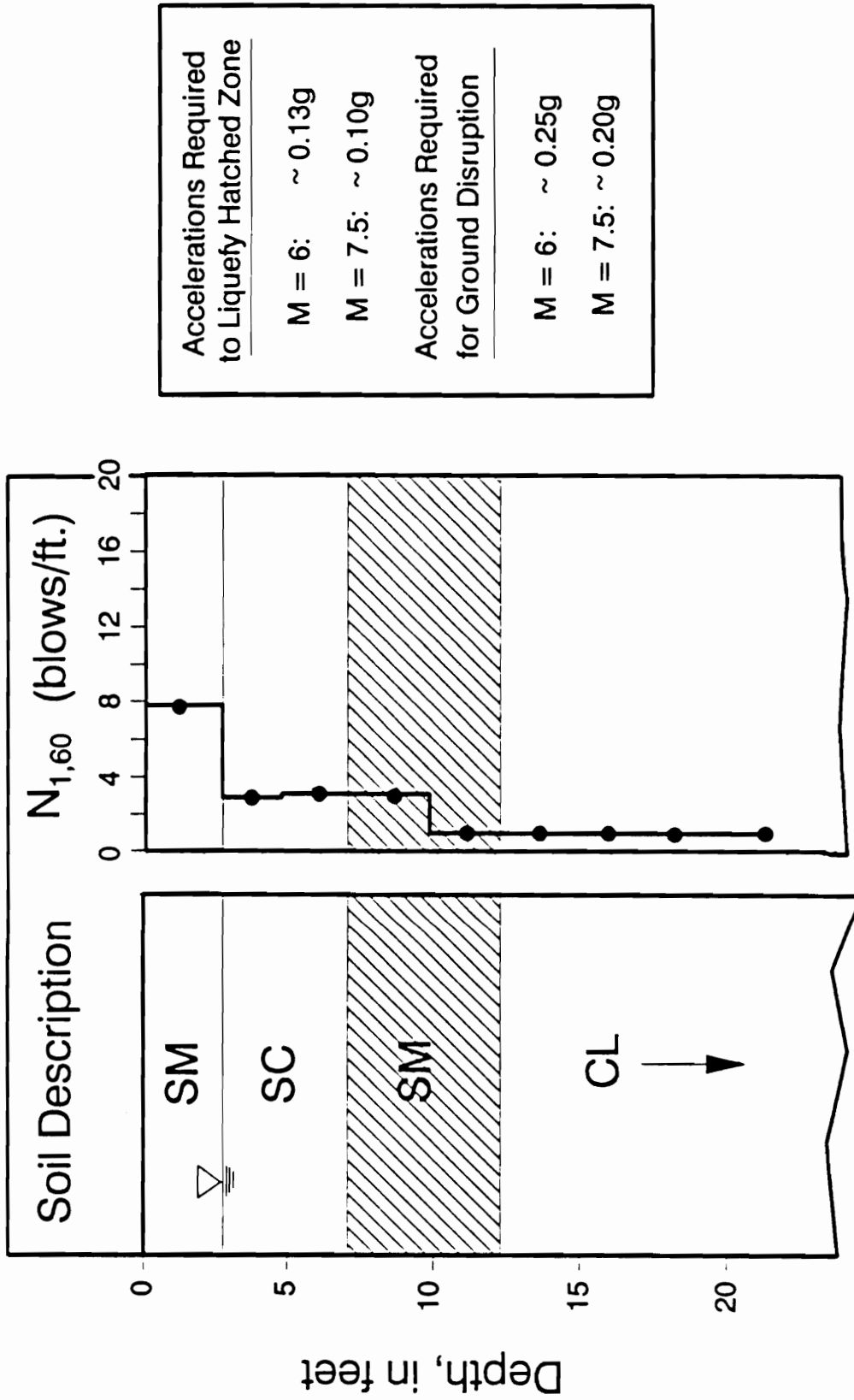


Figure 6.16 Soil Information for Former Manigault Residence in Downtown Charleston.

6.4.3 Mt. Pleasant Area Sites

MT. PLEASANT PITS

The soil profile at the Mt. Pleasant Pits site revealed that liquefiable sands are present in the upper reaches of the deposits. The profile from Fig. 5.32 in Chapter 5 showed that the sand layers that were most susceptible to liquefaction were generally between 8 and 22 ft. The water table was estimated to have been at about 4 ft. below the ground surface at the time of the earthquake, although the level was lowered by the dewatering operation at the time of the borings. In Fig. 6.17, a plot is shown to indicate the susceptibility of the sand layers to different levels of maximum ground acceleration. The samples shown in the figure are those CPT and SPT samples obtained from the liquefiable portion of the soil profile shown in Fig. 5.32. The CPT values were converted to equivalent SPT blowcounts using a Q_c/N factor of 5.0. It can be seen that for $M = 7.5$ and an acceleration of 0.2g, liquefaction of almost all of the samples would occur and ground disruption would be likely. The acceleration required to produce the same amount of ground disruption for a magnitude 6 event would also be in the 0.20g range. The historical observations in the vicinity of the site do not indicate any significant liquefaction activity from the 1886 quake. More important-

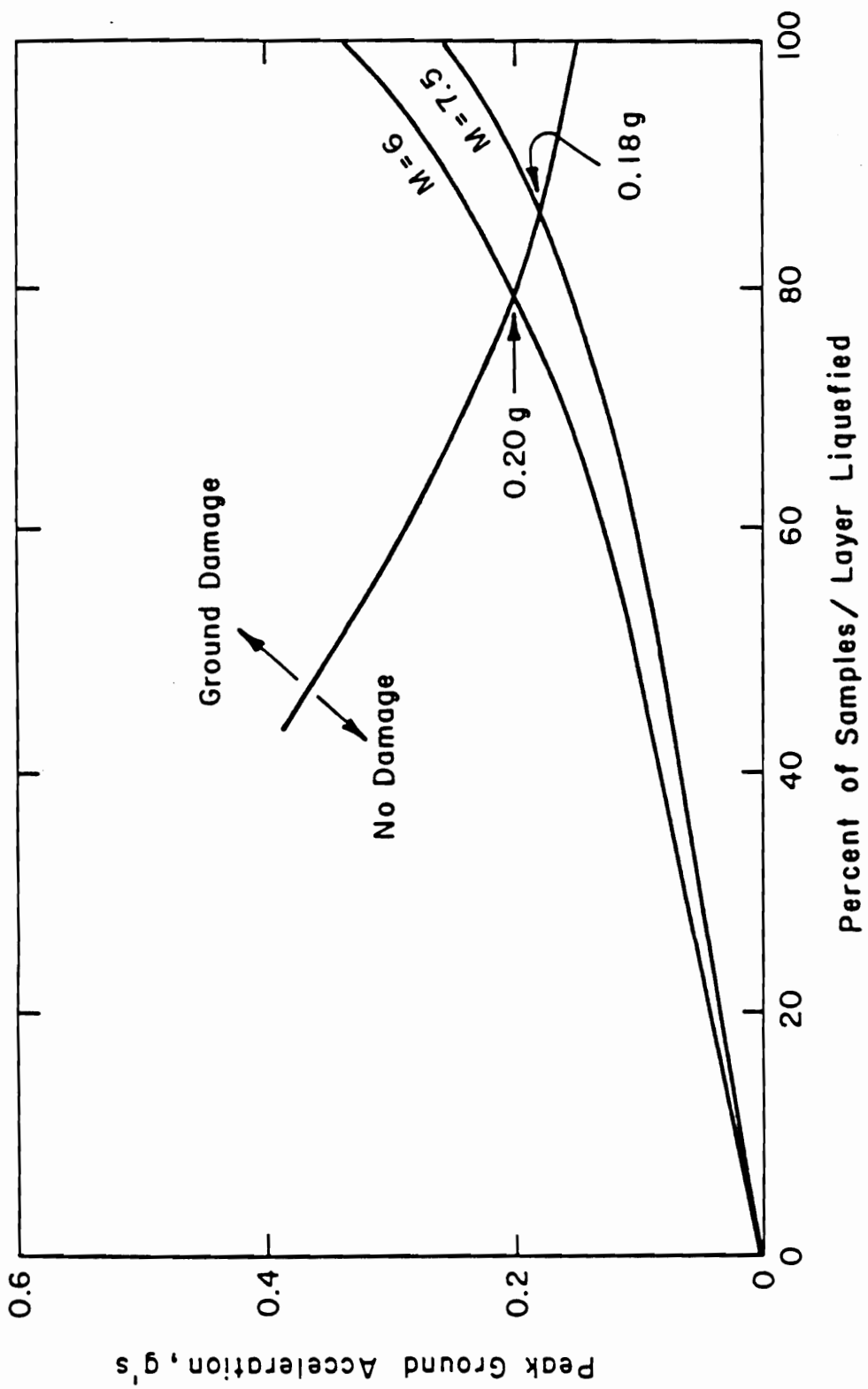


Figure 6.17 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Mt. Pleasant Pits Site (42 Samples).

ly, the observations at the site by Obermeier revealed no signs of liquefaction whatsoever, and his observations were performed down to depths of up to 15 ft. On the basis of this knowledge, it would be expected that ground motions of the 1886 event at this site were not sufficient to liquefy the sands there, and that maximum ground accelerations did not greatly exceed the threshold values.

MT. PLEASANT STRIP MALL

Obermeier's observations at the Mt. Pleasant Shopping Center site indicated that very minimal liquefaction occurred at the site in the form of a small liquefaction vent. The soil profile given in Fig. 5.33 of Chapter 5 indicated the presence of loose liquefiable material generally from depths of 4 ft. to at least 20 ft. It is expected that the loose sands extend beyond 20 ft., but the borings at the site were terminated at this depth. Liquefaction and major ground disruption was predicted to occur at the site for an acceleration in the range of 0.2g for a magnitude 7.5 event, and at less than 0.25g for a magnitude 6 event. Given the minimal evidence for liquefaction at the site, it is expected that these acceleration levels are upper bound estimates for ground shaking at the site in 1886.

MARK CLARK BRIDGE SITE

The work done at the Mark Clark Bridge site was done to supplement our knowledge of soil conditions in the Mt. Pleasant area. The SPT borings and CPT's obtained at the Mark Clark Bridge site revealed the presence of liquefiable silty to silty fine sands in the upper 20 ft. of the soil profile. The two SPT's at the site indicated that the sands were consistently loose to medium in density to about 22 ft. Because our knowledge of the amount of liquefaction that occurred at this site during 1886 was somewhat moot, less emphasis was placed on the liquefaction analyses here than elsewhere. However, it is known with good confidence that liquefaction across the general area was very minimal as indicated by historical references and paleoseismic investigations of U.S. Geological Survey personnel. The plot given in Fig. 6.18 was based on the two SPT's performed at the site and was constructed to show what levels of acceleration are necessary to cause liquefaction of the sand stratum from 3 ft. to 22 ft. and disruption of the ground surface. The figure indicates that for a magnitude 7.5 event, accelerations in the range of 0.2g would cause both ground disruption and liquefaction of most of the sand stratum. The acceleration level required to produce this amount of liquefaction activity for a $M = 6$ event would also be close to 0.2g.

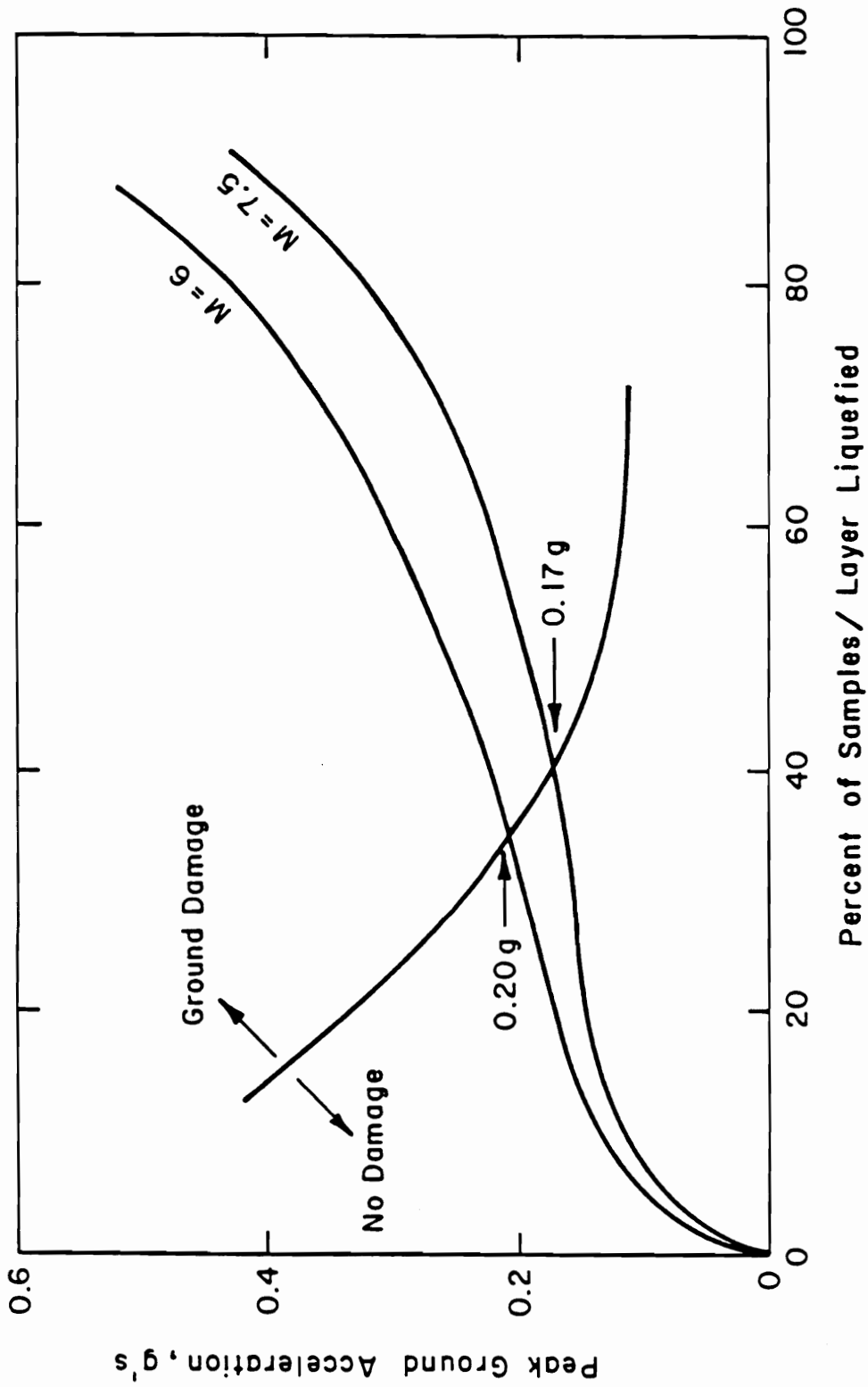


Figure 6.18 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Mark Clark Bridge/Isle of Palms Connector Site (8 Samples).

6.4.4 South Tibwin Sites

The soil profiles at the South Tibwin and Saint-James Santee School sites revealed uniform soil conditions and relatively equal liquefaction susceptibilities of the soils across the investigated area. The historical records indicated that little or no liquefaction was observed in this area following the 1886 quake. Obermeier's investigations at the South Tibwin site revealed the presence of undisturbed near-surface soils free of liquefaction features or signs of ground disruptions. No paleoseismic investigations were performed at the school site. However, given the close proximity of the two sites and uniformity of soil conditions between them, the implications of both sites were essentially the same, and for the analyses given here, the sites were combined. The penetration data obtained at the sites indicated the presence of liquefiable sands from below the water table at 2 ft., down to about 12 ft. where soft clay was encountered. Eleven SPT's from the school site and one from the South Tibwin site were used in Fig. 6.19 to show that ground disruption and liquefaction was predicted to occur at an acceleration level in the 0.2g range for both a magnitude 7.5 and magnitude 6 earthquake. The fact that no liquefaction or ground disruptions were evidenced at either of the sites was consistent with the idea that ground accelerations in this

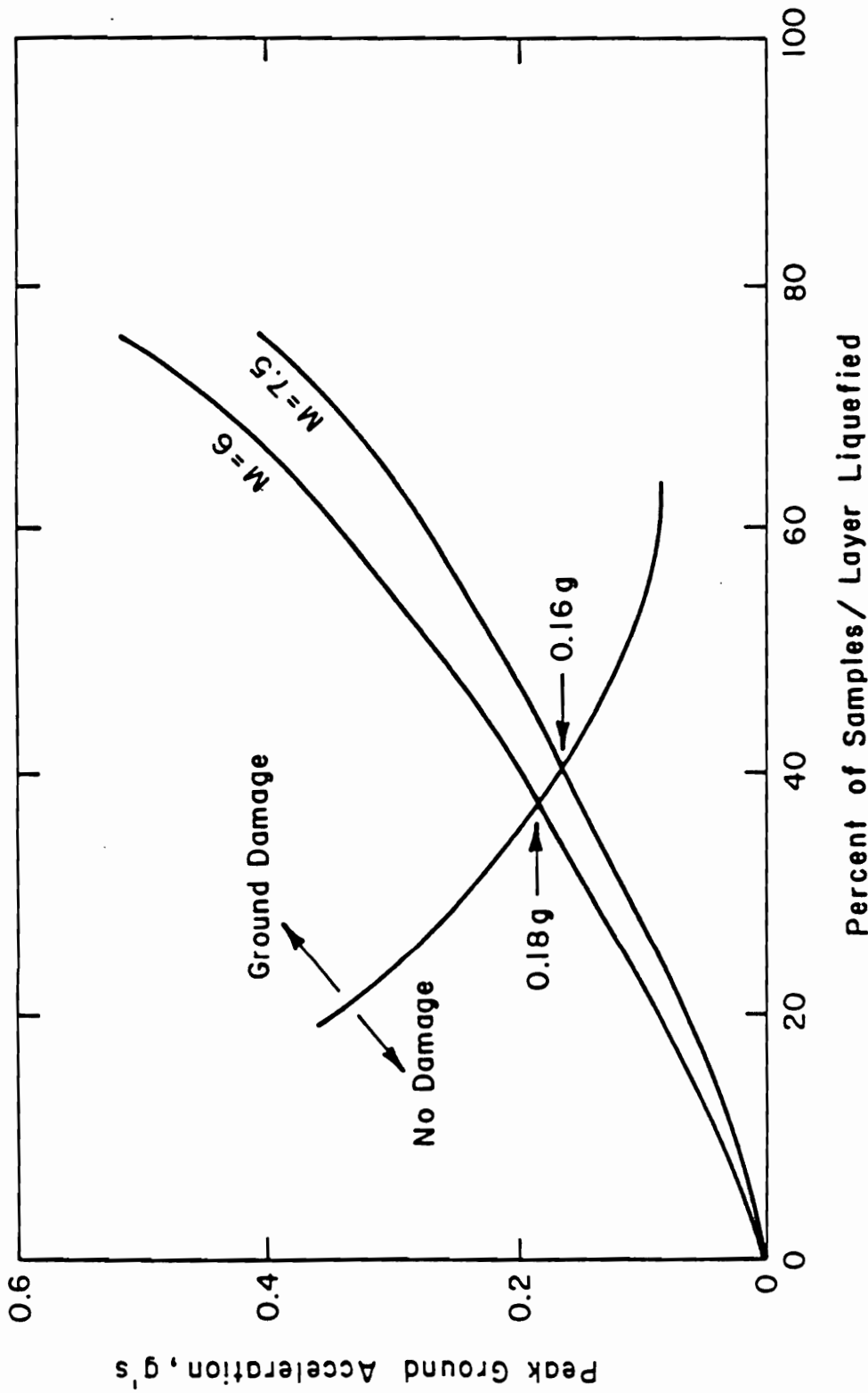


Figure 6.19 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at South Tibwin/St. James-Santee Schools Sites (45 Samples).

vicinity during the 1886 event were at or below the threshold values.

6.4.5 Georgetown Area Sites

KING AND PRINCE STREETS

The physical evidence at the King and Prince Streets site was somewhat moot in that it was uncertain as to whether the historical accounts accurately reflected the occurrence of liquefaction there in 1886. The preponderance of the evidence however suggested that little or no liquefaction developed there, and this position was maintained in the analysis of the site. The penetration data performed at the site consisted of 3 CPT's and 1 SPT all of which indicated the presence of liquefiable material from 5 ft. to an average depth of 12 ft. The CPT values were converted to equivalent SPT values and used with the one SPT in Fig. 6.20 to show the susceptibility of the sands in the upper 12 ft. to liquefaction. The figure shows that the accelerations required to cause liquefaction and ground disruption for an $M = 7.5$ earthquake were close to $0.3g$. It was predicted for an $M = 6$ earthquake that the accelerations necessary for the same amount of liquefaction were in the $0.35g$ range. The implications were that the accelerations at the site should not have greatly exceeded these values to

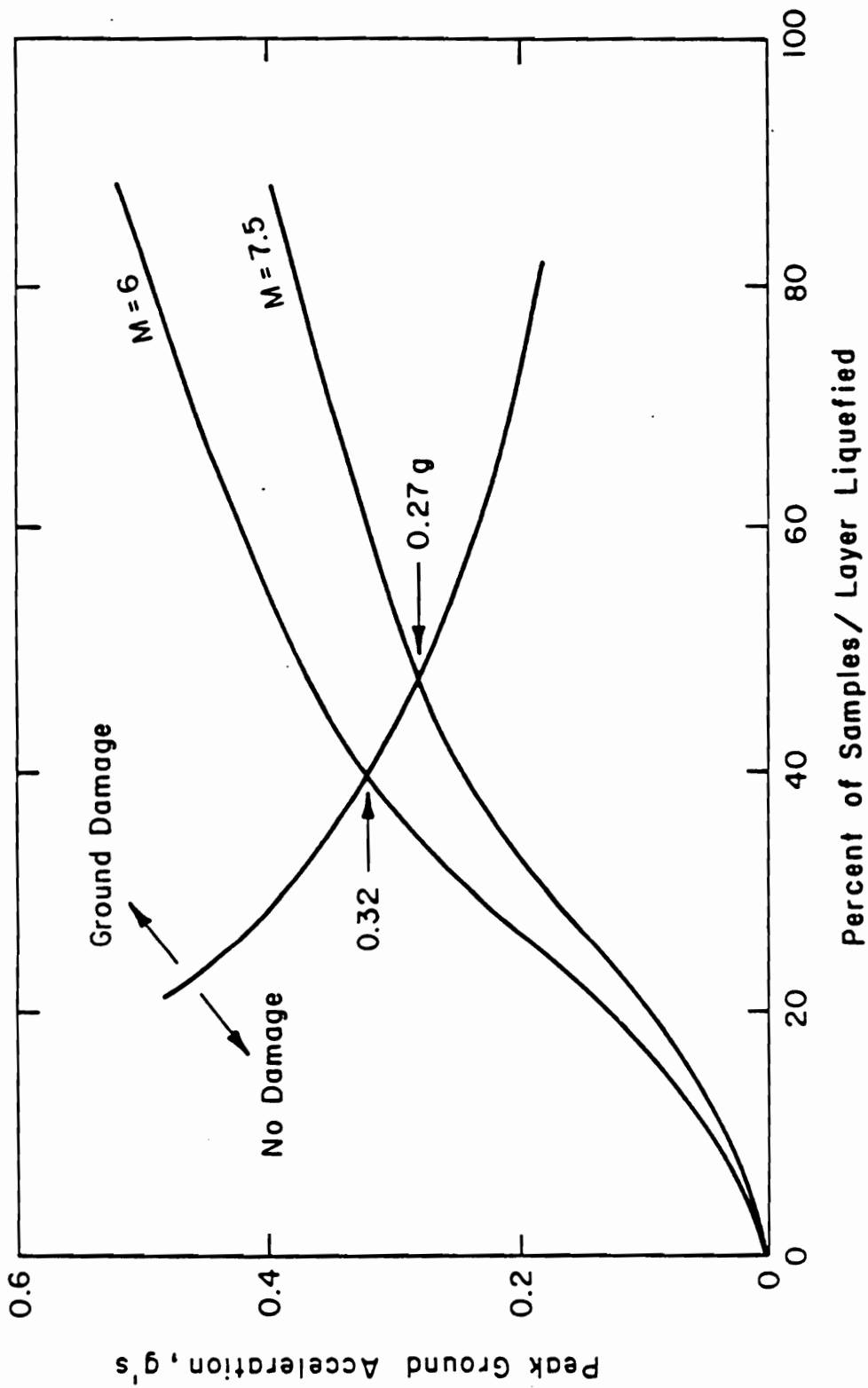


Figure 6.20 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Georgetown (King & Prince Sts.) Site (9 Samples).

remain consistent with the fact that little or no liquefaction occurred there.

PARKERSVILLE/PAWLEY'S ISLAND SITE

The soil conditions at the Parkersville site indicated the presence of liquefiable material, although the soils encountered there were denser than the sands at many of other sites. Only one of the four CPT's performed there indicated a susceptibility of the soils to liquefaction at low levels of ground acceleration. The other three tests indicated sands of moderate susceptibility. The historical accounts indicated that some type of ground disruption occurred at the site in 1886, although it is uncertain as to whether the damage was liquefaction-related. The CPT's revealed that a majority of the samples would be liquefiable at accelerations in the range of 0.25 to 0.30g. for a magnitude 7.5 event, and 0.35 to 0.40g for a magnitude 6 earthquake. These estimates must be tempered by the fact that this site is more than 65 miles from the zone of energy release and that liquefaction at the site is likely to be governed by the loosest sands at the site.

6.4.6 Edisto Island Site

The soil borings and penetration tests done at the Seaside Plantation on Edisto Island revealed the presence of

thick layers of liquefiable material. Our knowledge of the amount of liquefaction that occurred in the vicinity of the plantation is based on historical accounts which indicated that minor sand boiling was the most significant liquefaction expression observed in the area. The soil profile was dominated by loose fine to silty fine sands to depths greater than 30 ft. The most susceptible sand stratum extended from below the water table at 3 ft. down to a depth of about 22 ft. The 8 CPT's and one SPT performed there were used to indicate the susceptibility of the sands to liquefaction at different acceleration levels. The CPT values were converted to equivalent SPT blowcounts in the analyses. Fig. 6.21 shows the levels of acceleration that are predicted to cause ground disruption and liquefaction of a significant portion of the upper sand stratum. It can be seen that an acceleration of close to 0.2g is expected to cause significant ground disruption for both an $M = 7.5$ and $M = 6$ earthquake.

6.4.7 Analysis Of Data From Outside Sources

The SPT data obtained from local firms for sites outside of the meizoseismal region were used to estimate acceleration levels that would be required to initiate liquefaction (and surficial ground disruption) at various locations throughout the study region. The analyses were focussed upon sites which had characteristics appropriate

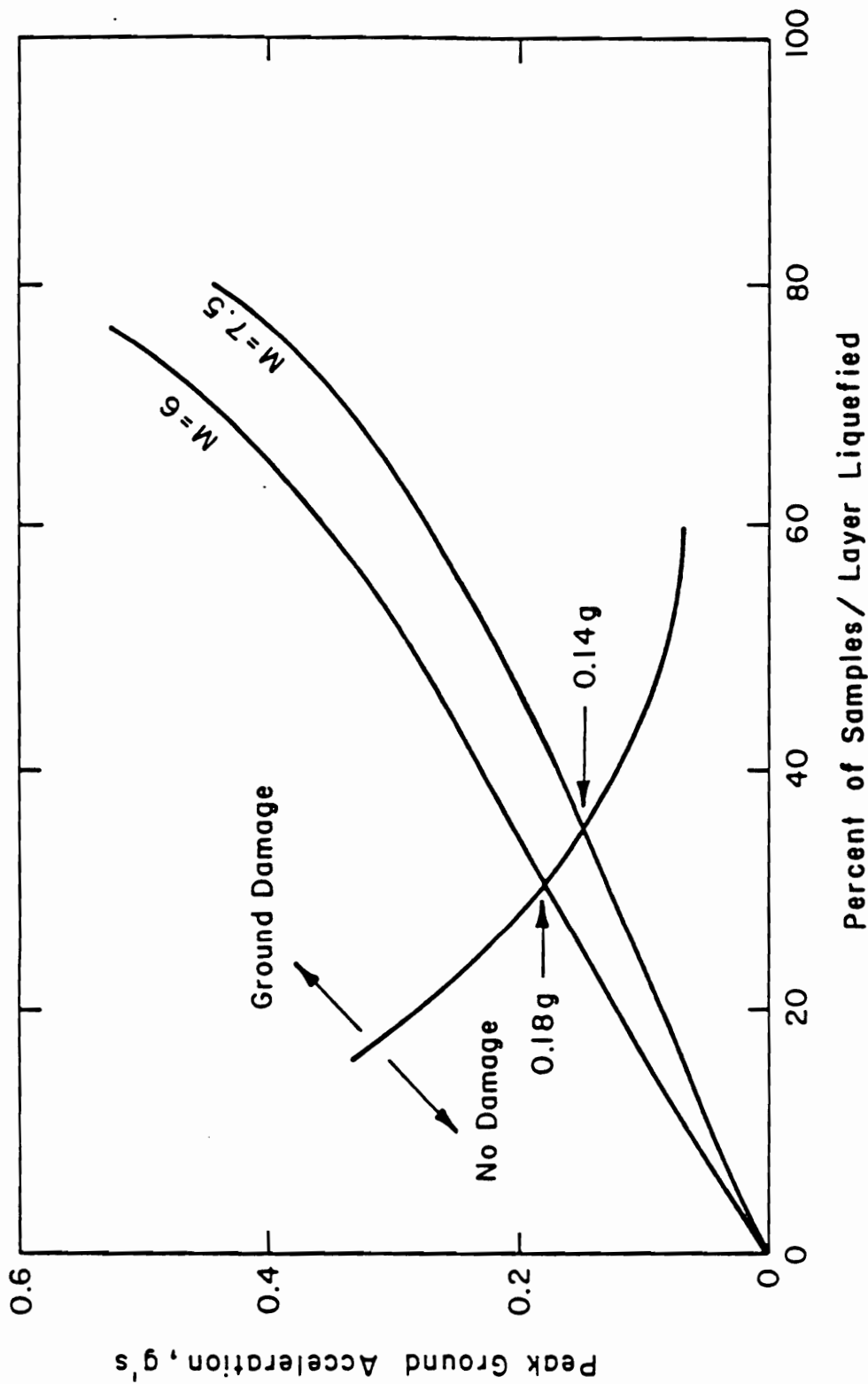


Figure 6.21 Refined Estimate of Accelerations Required to Cause Surficial Ground Disruption at Edisto Island Site (63 Samples).

for liquefaction, but were not covered by the Virginia Tech research group. This corresponds to several locations within the 85,000 year-old and present beach deposits along the reach from Hilton Head Island to the south of Charleston to near Pawley's Island to the north. In examining the data base of SPT logs for this region, it became apparent that soil conditions along these deposits were remarkably similar, and the implications of many of these data upon the acceleration levels of the 1886 event, are the same. For this reason, only a portion of the data was analyzed, and the scope of these data were limited to those areas judged to have the most significance.

It is important to consider that liquefaction activity outside of the meizoseismal region was minimal and localized during 1886. In this marginal liquefaction environment, it would be expected that liquefaction and ground disruption occurred at localized areas with the highest relative liquefaction susceptibility. Thus, it was judged that sites outside of the meizoseismal region were best characterized by the SPT boring with the lowest resistances and with the most favorable combination of non-liquefiable overburden and liquefiable soil layer thickness. For the analyses here, the lower bound average of the SPT blowcounts at each site was used to determine threshold acceleration values. Typically about 25 borings were used in this averaging process. The lower bound average was estimated by

subtracting one standard deviation from the mean of the resistance values (this approximately corresponds to the lower 20th percentile). The blowcounts obtained using this procedure were found to be close to the minimum values recorded at the sites. Additionally, this procedure accounted for variability of soil conditions and allowed for broad areas to be characterized more reasonably. The latter is important in that outlying areas far from the epicentral region, such as Hilton Head Island, were analyzed as individual sites, although they encompass broad areas.

The sites selected for analysis are listed in Table 6.4 which gives key information for each site including the threshold acceleration values. The accelerations were estimated using the procedures described in the previous analyses. The information contained in the table indicates that:

- 1) Significant thicknesses of liquefiable material are present at most locations, and the non-liquefiable overburden is generally less than 5 ft.
- 2) Soil conditions with respect to stratigraphy and density are relatively consistent from site to site.
- 3) Liquefaction and ground disruption at most sites would be expected to occur at acceleration levels of less than 0.2g for magnitude 6 and 7.5 events.

Also, based on the pattern of 1886 liquefaction activity shown in Fig. 2.3, the acceleration values listed

Table 6.4 Summary of Liquefaction Analyses of SPT Data Obtained From Outside Sources For Sites Outside the 1000
Metrosesimal Zone

Site	Distance From Energy Release (mi.)	Extent of Liquefaction (1) 1000	Number of SPT Samples (2)	Avg $M_{1,60}$ (3)	Standard Deviation (Blows/ft.) (4)	Lower Bound $M_{1,60}$ (Blow/ft.) (5)	Avg Thickness Liquefiable Layer (ft.) (6)	Avg Thickness Capping Layer (ft.) (7)	Threshold Accelerations (g's) M=6 (8)	Threshold Accelerations (g's) M=7.5 (9)	Percent Samples Liquefied (10)	Percent Samples Liquefied (11)
John's Isl.	10	Minor	80	14	4	10	14	5	0.17	0.14	60%	68%
James Isl.	13	Minor	40	8	3	6	16	3	0.09	0.07	50%	70%
Kiawah & Seabrook Isl.	14	None	354	12	6	6	21	3	0.13	0.10	50%	50%
Edisto Isl.	18	V. Minor	174	18	9	9	19	5	0.16	0.12	50%	58%
Isle of Palms	20	None	33	26	3	23	13	5	0.24	0.20	50%	50%
Cape Romain	26	None	32	19	6	13	10	4	0.16	0.15	58%	70%
St. Helena Isl	38	None	80	24	5	19	17	3	0.19	0.15	50%	50%
Beaufort	40	None	155	17	5	12	15	4	0.16	0.14	60%	70%
Hilton Head	53	None	294	27	9	18	21	3	0.17	0.13	50%	50%
Bluffton	55	Minor (6)	70	42	10	23	17	3	0.31	0.25	50%	50%
Dabidue	61	None	80	24	6	16	17	4	0.17	0.14	50%	50%

(1). Based on Historical Observations in General Vicinity of Site.

(2). From Within Liquefiable Layer

(3). Includes Correction for Fines Content.

(4). Mean $M_{1,60}$ Value Minus One Standard Deviation (~20th percentile). This value used to compute Threshold Accelerations.

(5). Estimated Peak Accelerations Required for Surficial Ground Disruption (see Appendix D). Values Plotted in Figs. 6.22 & 6.23.

(6). Anick et al., 1990 in MURLG/CR-5613, 1990.

in Table 6.4 can be used to gain some insight into the attenuation characteristics of the 1886 earthquake. At sites where no 1886 liquefaction was observed, it was considered that accelerations were less than the threshold values listed in Table 6.4. On the other hand, at sites where liquefaction activity was documented, the level of ground accelerations was expected to have been at or above the threshold values, depending upon the extent of the liquefaction. Because liquefaction outside of the meizoseismal zone was marginal during the 1886 event however, its occurrence at most of the locations can be explained by the values in the table.

Examining Fig. 2.3, the sites located just outside of the meizoseismal zone, namely Johns Island and James Island, are shown to have experienced minor liquefaction during the 1886 event. The threshold values for these areas range from 0.1 to almost 0.2g for a magnitude 6 event, and from 0.1g to about 0.15g for a magnitude 7.5 event. Thus it would be expected that accelerations in the range of 0.2g for an $M = 6$ event, and slightly less than 0.2g for an $M = 7.5$ event, would be upper bound estimates of 1886 ground shaking at these areas. It can also be seen in Fig. 2.3 that there is a general absence of liquefaction activity at areas beyond Edisto Island to the southwest of Charleston, and beyond Mt. Pleasant to the northeast. Both locations were situated about 20 miles from the zone of energy release.

This would imply that maximum ground accelerations at locations beyond about 20 miles from the epicentral region were less than the threshold values listed in Table 6.4; that is, less than 0.2g for $M = 6$ and less than about 0.15g for $M = 7.5$.

Thus, if it is assumed that the 1886 earthquake was an $M = 7.5$ event, then it would appear that the observed patterns of liquefaction activity outside of the meizo-seismal zone can be explained by: 1.) accelerations in the range of 0.2g for areas just outside of the meizoseismal zone; and, 2.) accelerations less than about 0.15g for areas about 20 miles outside of the zone of energy release. It should be noted however, that knowledge of soil conditions and past liquefaction activity at the sites in Table 6.4 are less definitive than at the Virginia Tech sites, and thus the attenuation patterns suggested by these data can only be thought of in a general sense.

6.5 ESTIMATED ATTENUATION OF SEISMIC ENERGY FROM THE 1886 EVENT

A more complete picture of the attenuation pattern is illustrated in Fig. 6.22 where test data from the Virginia Tech sites and that from the commercial firms are combined and plotted. In the figure, the distance of each site from the zone of energy release was plotted against the peak

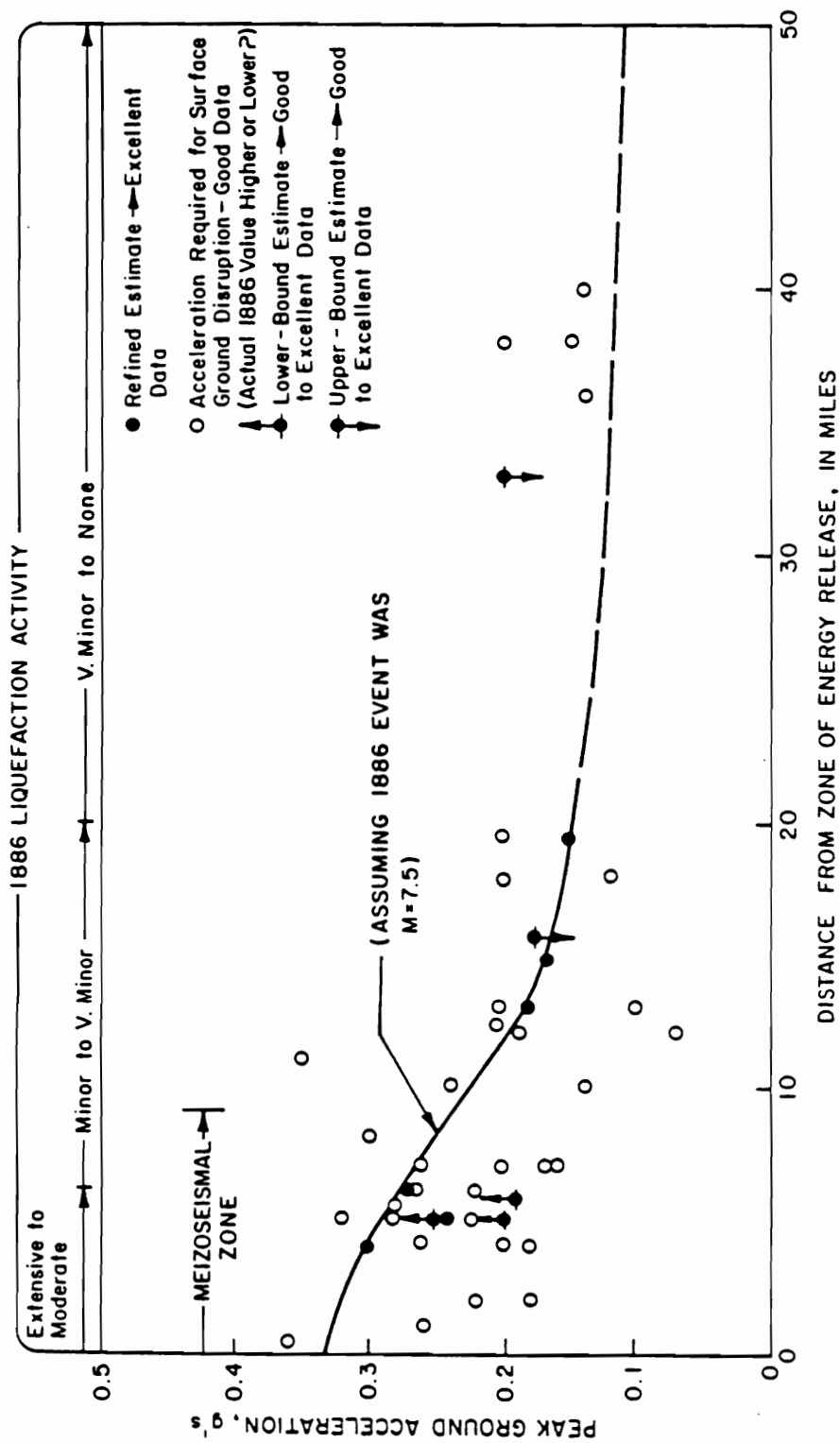


Figure 6.22 Estimated Pattern of 1886 Maximum Ground Accelerations Based on 1886 Field Performance and Penetration Testing. Curve Shown Was Developed Assuming 1886 Event Was $M = 7.5$. Data Taken From Tables 6.1, 6.2, and 6.3.

acceleration values required to produce surficial ground disruption (threshold accelerations).

In examining the plot it is important to keep in mind that the threshold accelerations are not necessarily the acceleration levels that are predicted to have occurred at the sites in 1886. In estimating the actual acceleration levels, it is necessary to consider the amount of liquefaction that occurred at the particular site relative to the threshold accelerations. In cases where marginal ground disruption occurred, the actual 1886 accelerations are expected to be close to the threshold values. On the other hand, at sites where the liquefaction was either extensive or non-existent, the likely 1886 acceleration levels can only be bounded; these sites are denoted in Fig. 6.22 by the data points with arrows attached. The arrows point either up or down to indicate whether the 1886 accelerations were likely to have been greater than or less than the threshold values. The solid data points (with no arrows attached) represent sites for which excellent field data were available and where refined estimates of the 1886 acceleration level could be made. The open circles represent sites where the extent of 1886 liquefaction activity was known only in a general sense. Thus, the significance of the estimated threshold accelerations at these sites is somewhat moot in that the actual 1886 acceleration level could have been higher or lower than these values, and cannot be

closely estimated without better knowledge of 1886 field performance. Collectively, these factors were considered in constructing the attenuation curve shown in Fig. 6.22, which is judged to represent the pattern of peak ground accelerations that would be most consistent with the findings of this investigation.

The attenuation curve for the $M = 7.5$ event shows the accelerations ranging from about 0.35g near the zone of energy release to 0.2g at 10 miles. Beyond 10 miles, a slow decrease in the acceleration from 0.2g to close to 0.1g at about 45 miles is observed. An attenuation curve developed for an $M = 6$ event is also shown in Fig. 6.23. It can be seen that the $M = 6$ curve indicates higher accelerations near the source zone ($\sim 0.4g$).

6.6 SHAKE ANALYSES

One objective of this study is to investigate the possibility that soil amplification or other dynamic site response effects could cause the development of liquefaction to be different from site to site. In assessing this possibility, the Hollywood and Oakland Plantation sites were selected for analysis with the computer code SHAKE (Schnabel et al., 1972). The Hollywood site was chosen to be representative of sites within the meizoseismal zone, while

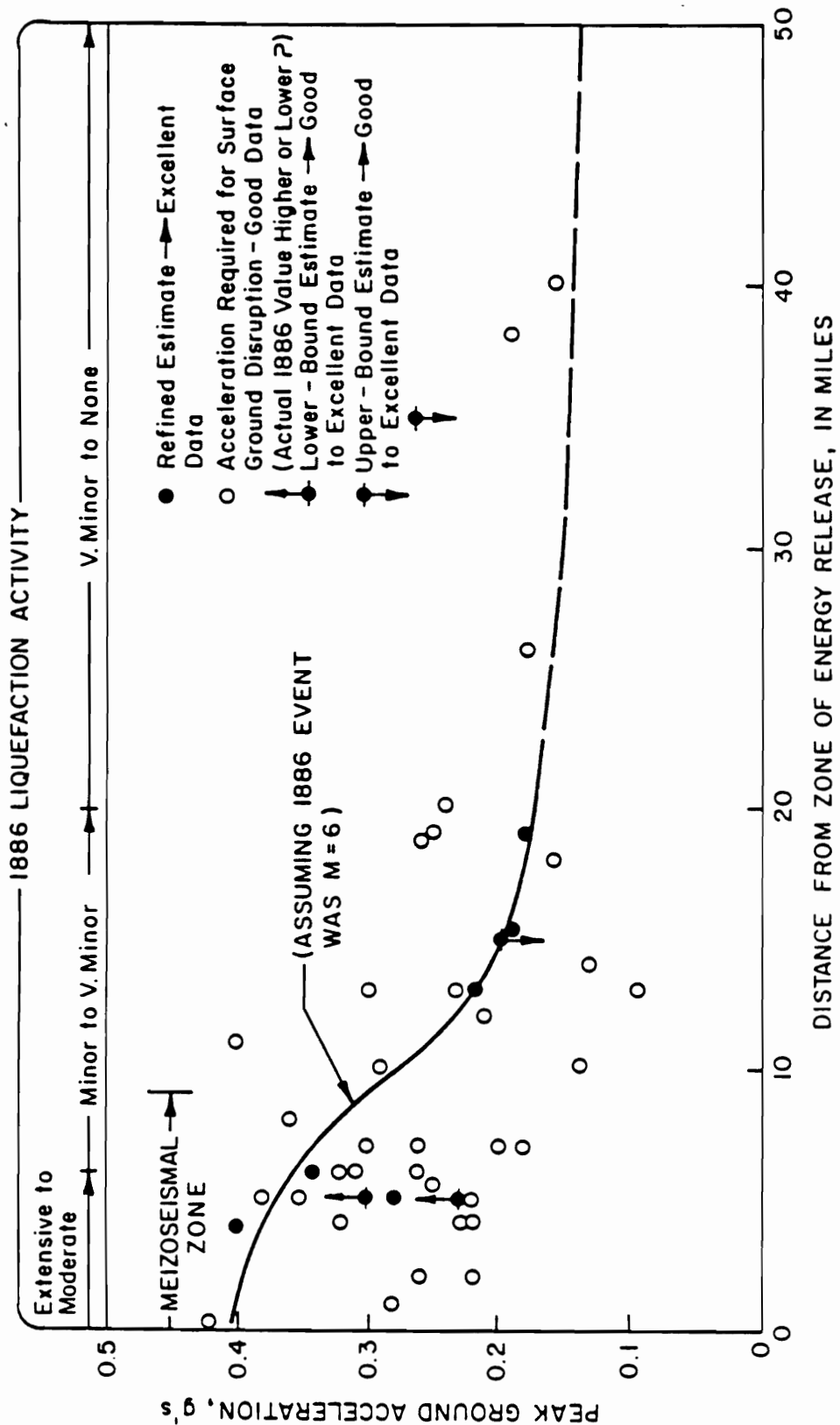


Figure 6.23 Estimated Pattern of 1886 Maximum Ground Accelerations Based on 1886 Field Performance and Penetration Testing. Curve Shown Was Developed Assuming 1886 Event Was M = 6. Data Taken From Tables 6.1, 6.2, and 6.3.

Oakland Plantation was is considered representative of sites outside of this region.

SHAKE is a one-dimensional program that computes site response in a horizontally layered soil system subjected to shear waves propagating upward from an elastic halfspace. Equivalent linear soil properties are used with an iterative procedure to obtain soil properties that are compatible with the strains developed in each layer. The code requires input of dynamic soil properties including shear moduli and damping factors, as well as curves for the degradation of these parameters with shear strain. Dynamic soil properties for the sand, clay, and marl materials at the Hollywood and Oakland sites were estimated on the basis of: 1.) indirect correlations to in-situ penetration resistances and laboratory strength tests; 2.) dynamic laboratory testing done by commercial firms; and, 3.) cross-hole and down-hole shear wave velocity tests performed by commercial firms. Shear wave velocity tests in the Cooper Marl revealed velocities of about 2000 fps near the top of the formation, and greater velocities with depth; hence, Cooper Marl was considered "bedrock" for the base input.

In that no motions for strong ground shaking have been recorded for the Charleston area, two synthetic motions developed for the Charleston area by Chapman et al. (1989) were used as bedrock input motions for the analyses. The motions are designated as the LO-Q and HI-Q motions, with

the LO and HI indicative of the assumptions regarding the "quality factor" or stiffness characteristics of the rock and stiff sediments underlying the Charleston region. The LO-Q motion was developed assuming a low quality factor (soft sediments) and contains a greater percentage of low frequency energy. The HI-Q earthquake assumes a stiffer sediment profile, and contains more high frequency content. Both motions were used in the analyses to bound the possible effects of each assumption.

Because it is understood that the synthetic motions may not be representative of those of the 1886 event, they are primarily used from the perspective of assessing relative site responses. It is also understood that the predominant period of earthquake motions influence site response; a variation in the natural period of the input motion would result in a different response of the site. A parametric study of both the Oakland and Hollywood sites was to be made by varying the natural period of the input motions at the base and assessing the site response for each period. However, the motions developed by Chapman contained several periods with high energy contents, and a predominant period could not be determined. This is consistent with the fact that his motions were developed for sites close to the epicenter which would be expected to contain a higher percentage of high energy frequencies. It was thus decided to use the generic motions developed by Chapman without a

parametric variation of the predominant period of the earthquake.

The main objective of the SHAKE analyses was determining the effect of the soil column upon the input motion propagating from the bedrock upward through the soil profile. The parameter of interest was the maximum acceleration, as this would give an indication of the induced shear stresses that would tend to cause liquefaction. (It may be remembered that acceleration can be directly related to shear stress, which in turn, is related to the development of liquefaction).

For the analyses, the earthquake motions were scaled to accelerations of 0.25 and 0.55 for both the LO-Q and HI-Q records, and input at the base of the profile; a total of four runs were carried out for each site. The accelerations obtained in the near-surface soil layers were compared to those input at the base. An increase in the acceleration, or amplification, would correspond to an increase in the shear stress that would tend to cause liquefaction. In the opposite situation where the motions are de-amplified, the tendency for liquefaction would be less. These effects were then compared from site to site to assess the relative tendency for the development of liquefaction given the same input motions.

The results and other important points of the analyses are summarized below:

- 1) No significant differences resulted between the analyses performed with the LO-Q motion and HI-Q motion. Also, there were no differences in the relative responses of the sites using the two motions.
- 2) At high acceleration levels (0.55g) both sites showed similar responses as they both tended to de-amplify the ground motions that propagated toward the ground surface. The maximum acceleration at the ground surface was about 25% less than that of the maximum acceleration for bedrock at both sites for both the LO-Q and HI-Q records.
- 3) At low acceleration levels (0.25g) both sites tended to amplify the ground motions. The maximum acceleration level at the ground surface was about 30% higher than that at bedrock level for both sites for both the LO-Q and HI-Q motions.
- 4) The relative difference in the site response of the Hollywood site to that of the Oakland Plantation appears to be minimal. This is consistent with the fact that both sites are similar in terms of the factors that most influence site response. The sites both consist of soft sediments with about the same relative stiffnesses and about equal thicknesses of the sediments over stiff material.

In performing SHAKE analyses for other sites within the study region it appeared that the single-most important

parameter was the thickness of the soil profile over firm material (i.e., Cooper Marl). Among the sites across the study region, the depth to firm material is fairly consistent, especially within individual geologic units. On this basis, along with the analyses given here, it is suggested that there are no anomalous trends or localized site response effects that would tend to explain the selective pattern of liquefaction activity evidenced during the 1886 event.

CHAPTER 7

SUMMARY AND CONCLUSIONS

First-hand accounts of sand boils and other liquefaction-related phenomena associated with the 1886 Charleston earthquake provide clear evidence that liquefaction was common in this event. Recent paleoseismic investigations in the Charleston area have led to the discovery of important sites which exhibit relic liquefaction features from the 1886 event as well as other seismic events as old as 7200 years. This information has provided significant insight into the seismic history of the Charleston region, and has led to an improved understanding of the nature and source of the seismic shaking there. Still, little hard data exists in terms of the 1886 ground motion characteristics and the maximum level of induced ground accelerations. This investigation was undertaken to study the liquefaction findings in the Charleston area from the perspective of geotechnical

engineering. This involved obtaining engineering parameters at the sites, and drawing conclusions from this data regarding the liquefaction phenomena themselves, and the levels of causative seismic shaking.

The field work performed for this study involved general site reconnaissance and in-situ testing at locations where the extent of liquefaction caused by the 1886 event is well known, either from historical documentation or recent paleoseismic work. Testing was performed at locations within and outside of the areas where liquefaction was prominent, and at locations close to and far from the epicentral area. Soil conditions at these sites were defined on the basis of the in-situ tests and supplemental laboratory data. Based on the present soil conditions, the likely shaking levels of the 1886 earthquake were then estimated as the accelerations that would serve to explain the documented phenomena at each site.

At important sites where 1886 performance was well documented, field tests were performed by Virginia Tech personnel. The primary field test method was the Cone Penetration Test (CPT) and to a lesser extent, the Standard Penetration Test (SPT). Field testing was accomplished in two phases over a two-year period. The first year of the work primarily involved testing within the meizoseismal zone of the 1886 earthquake. During this time, a specially adapted light-weight drill rig was used to perform tests

with a small-scale electric cone designed at Virginia Tech. With each CPT, an auger boring was made to obtain soil samples for visual and sieve classifications of the soil. This phase of the testing also involved the hiring of a commercial firm to perform SPT's at one particularly well-documented site. The second year of the field work focussed on sites further away from the meizoseismal region. The field testing at these sites included performing SPT's and standard-sized CPT's using a full-sized drill rig. The total field effort from both phases of the work included 57 CPT's, 6 SPT's, and 35 auger borings. Additionally, more than 2000 SPT logs were obtained from local consulting firms to provide data in areas not covered by the research group. Laboratory testing done in conjunction with the field study included soil classification tests and static and dynamic triaxial tests.

Based on the results of the Virginia Tech investigations, the conclusions fall into two categories:

- 1) Those concerning the geotechnical setting of the Charleston vicinity; and
- 2) Those which explain the observations of liquefaction intensity associated with the 1886 earthquake.

The principal conclusions concerning the geotechnical setting include:

- 1) Liquefaction was primarily associated with beach deposits that roughly parallel the present coastline.

- 2) The areas most associated with historical and paleo-seismic evidence for liquefaction are ones with a high ground water table and fine to silty sand layers in the upper 20 ft.
- 3) 85,000-year-old and younger beach deposits located nearest to the Atlantic Ocean are more susceptible to liquefaction than older beach deposits (up to 230,000 years old). However, many of the soil layers within the older deposits continue to be prone to liquefaction and have liquefied multiple times in the past.
- 4) There is evidence for progressive densification in the older beach ridge deposits. The older beach deposits typically consist of denser material near the top of the soil profile underlain by looser material at depth. Younger beach ridge deposits are more uniform in density with depth than the older ones.
- 5) It is unique to the Charleston environment that the soils which have liquefied multiple times in the past 10,000 years are up to 230,000 years old. This is different from more active seismic environments where soils over 10,000 years old are not expected to be subject to further liquefaction.
- 6) The relatively low rate of significant seismic activity in the Charleston region can be used to explain why very old soil deposits remain susceptible to liquefac-

tion.(In more active regions, more densification occurs due to more frequent episodes of seismic shaking).

- 7) Soil conditions and liquefaction susceptibility of sediments within individual beach deposits are relatively consistent from location to location.
- 8) At sites where large liquefaction craters were formed such as Ten Mile Hill, the soil profiles consist of loose sands overlain by medium to very dense and often weakly-cemented sands with low permeability. This brittle-but-weak capping layer can be used to explain the formation of the large "explosion" craters.
- 9) Behavior of the Charleston sands in cyclic triaxial tests proved to be typical of other fine-grained sands.
- 10) Many sites in the city of Charleston and the surrounding region remain readily susceptible to liquefaction at relatively low levels of ground shaking (0.2g to 0.3g).

The conclusions related to the issue of the seismic loading required to produce the observed 1886 phenomena include:

- 1) Based on one-dimensional site response analyses, no evidence for anomalous site response or soil amplification effects was found at any of the test sites. The observed attenuation of liquefaction activity with distance from the 1886 source zone is thought to be due to normal attenuation of earthquake energy.

- 2) If the magnitude of the 1886 event was as large as has been conventionally estimated ($M = 7.7$), a difference arises in the acceleration levels suggested by the seismological evidence and those estimated from this investigation.
- 3) Assuming that the 1886 event was of magnitude 7.5 leads to the conclusion that accelerations no higher than 0.3 to 0.4g can explain the observed liquefaction phenomena within the meizoseismal zone. Similarly, liquefaction activity at areas immediately outside of the meizoseismal zone can be explained by maximum ground accelerations in the range of 0.2g. All of these acceleration estimates are lower than those proposed by the seismological evidence (0.5 to 0.6g in the meizoseismal zone).
- 4) If it is assumed that the magnitude of the 1886 event was less than 7.5, then the accelerations proposed by the seismological evidence and those from this study are in closer agreement. For instance, if it is assumed that the 1886 earthquake were as small as magnitude 6, then a maximum acceleration level of about 0.4g within the meizoseismal zone would explain the observed liquefaction phenomena.
- 5) Although the acceleration levels estimated by the seismological evidence and those suggested by this study are closer if it is assumed that the magnitude of

the 1886 quake was less than 7.7, it seems unlikely that the magnitude of the 1886 event could have been as low as 6 because of the widespread liquefaction and ground disruptions generated by the earthquake.

- 6) The evidence of the field studies and liquefaction analyses would suggest then that the level of seismic loading was less than that generated by a magnitude 7.5 earthquake but not as low as that which would be associated with a magnitude 6 event.

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APPENDIX A

CPT AND SPT RECORDS:

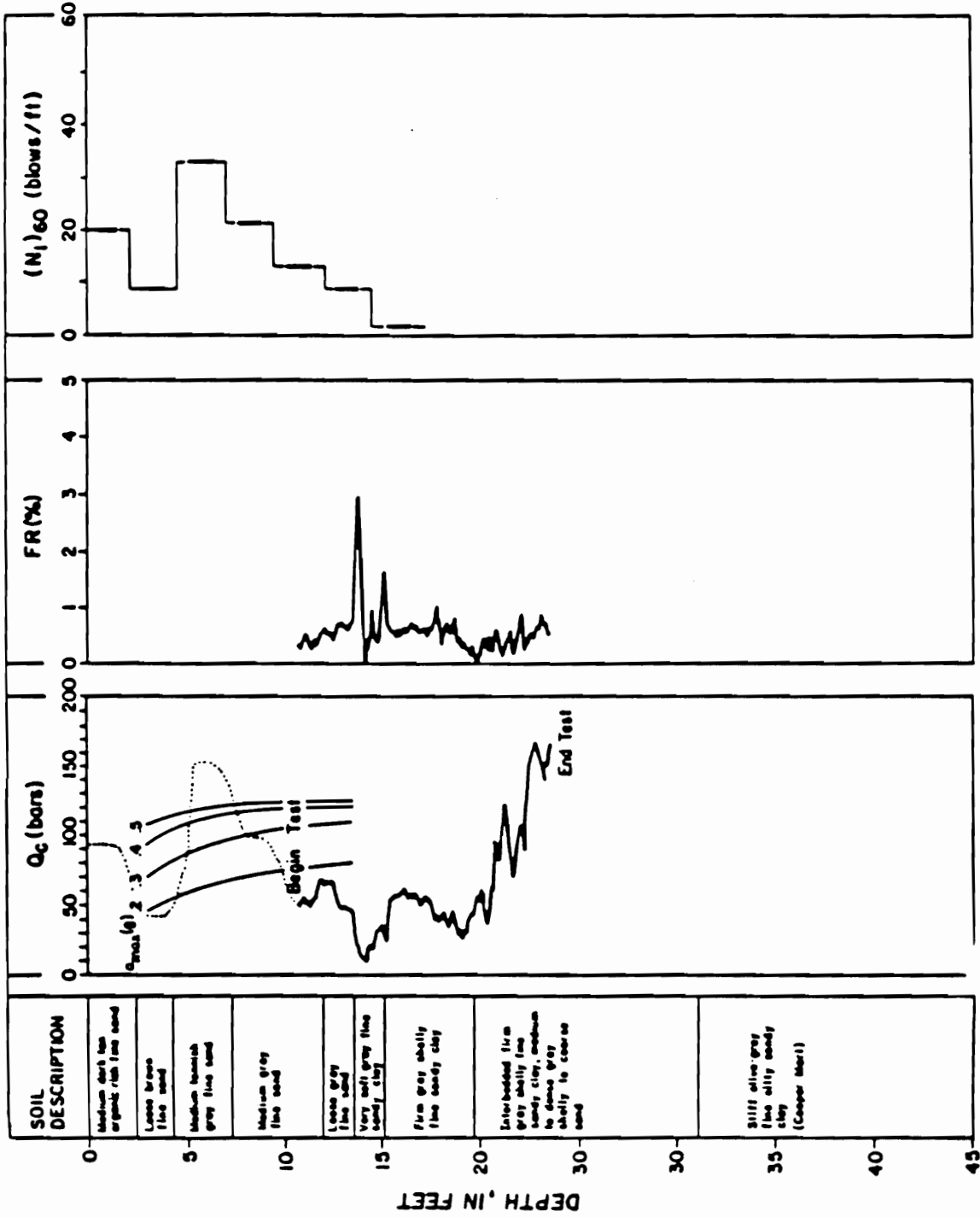
HOLLYWOOD, WARREN, SOD FARM, MONTAGUE, ELEVEN MILE POST, AND
TEN MILE HILL SITES

HOLLYWOOD SITE

CPT AND SPT RECORDS

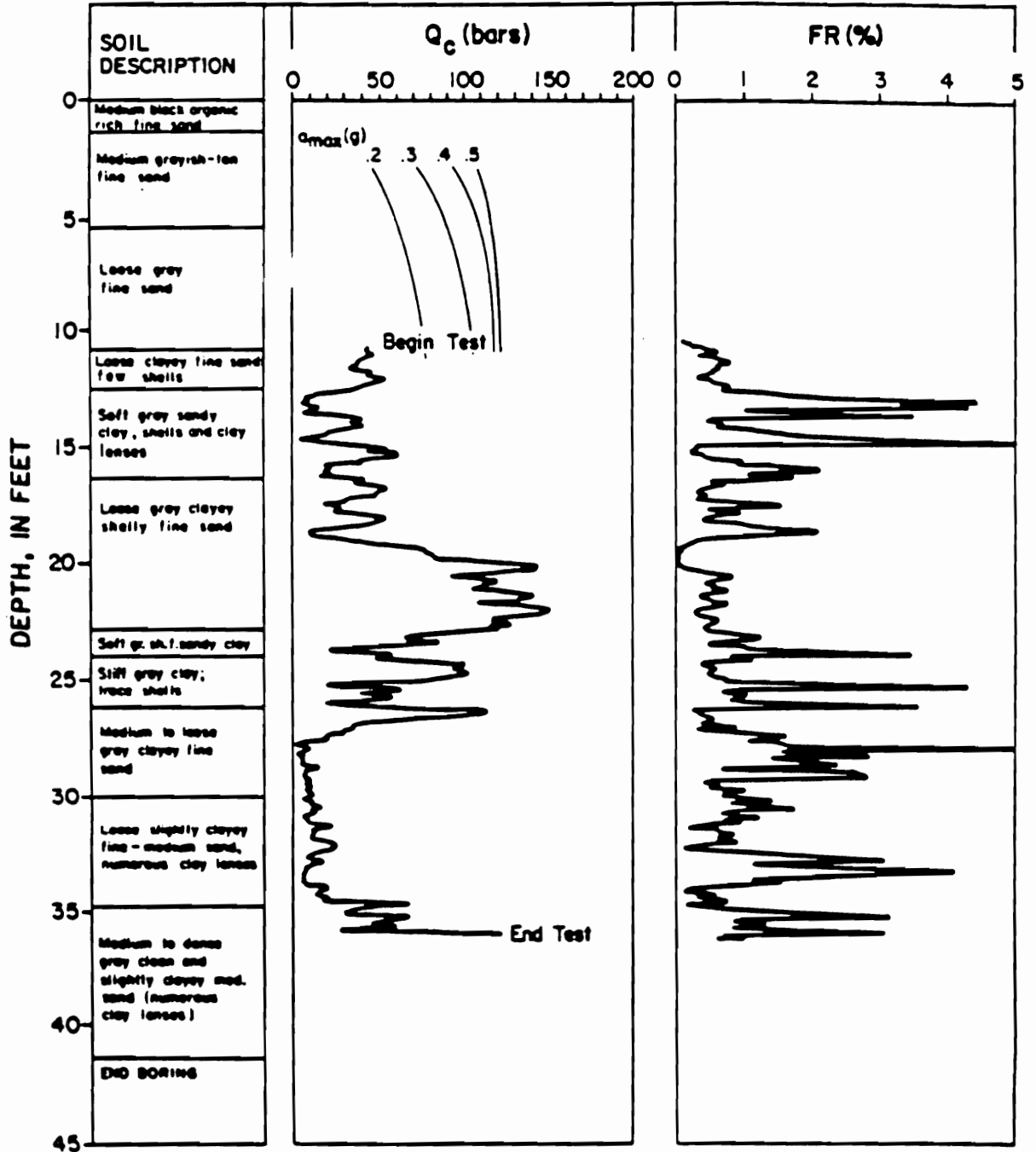
SITE : HOLLYWOOD STA. W0100'

CHARLESTON LIQUEFACTION STUDY



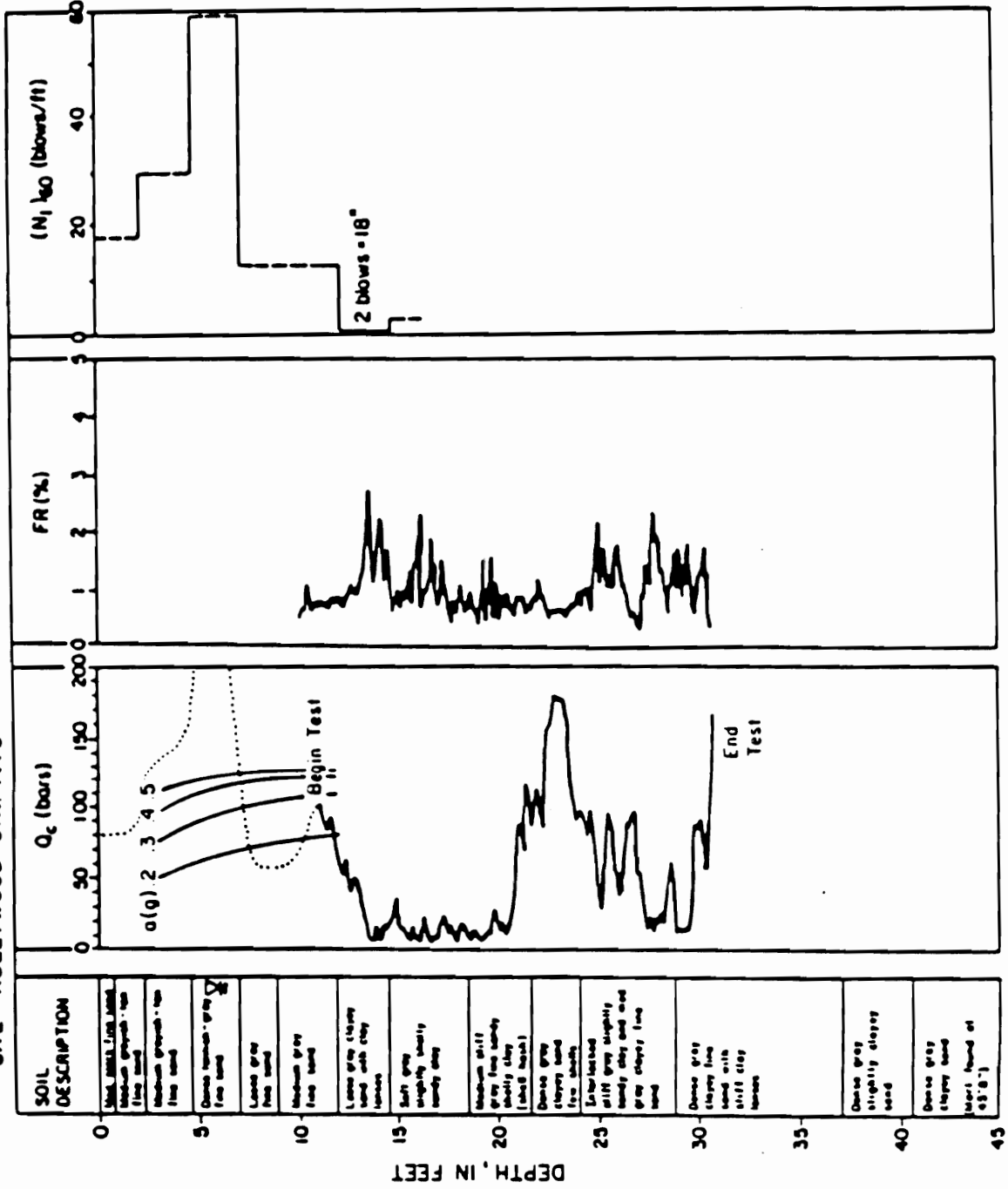
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CHARLESTON LIQUEFACTION STUDY



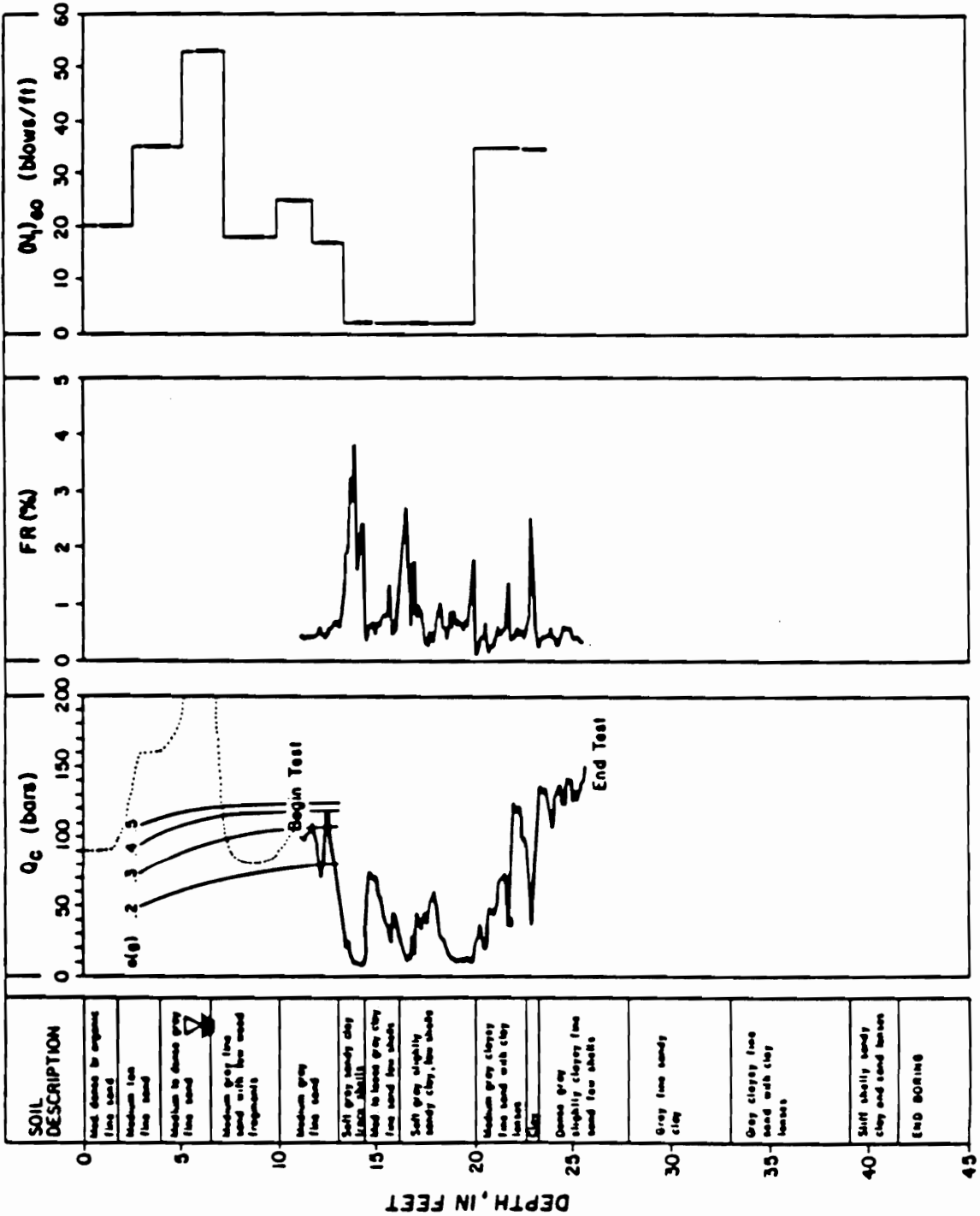
CHARLESTON LIQUEFACTION STUDY

SITE : HOLLYWOOD STA. 1475'



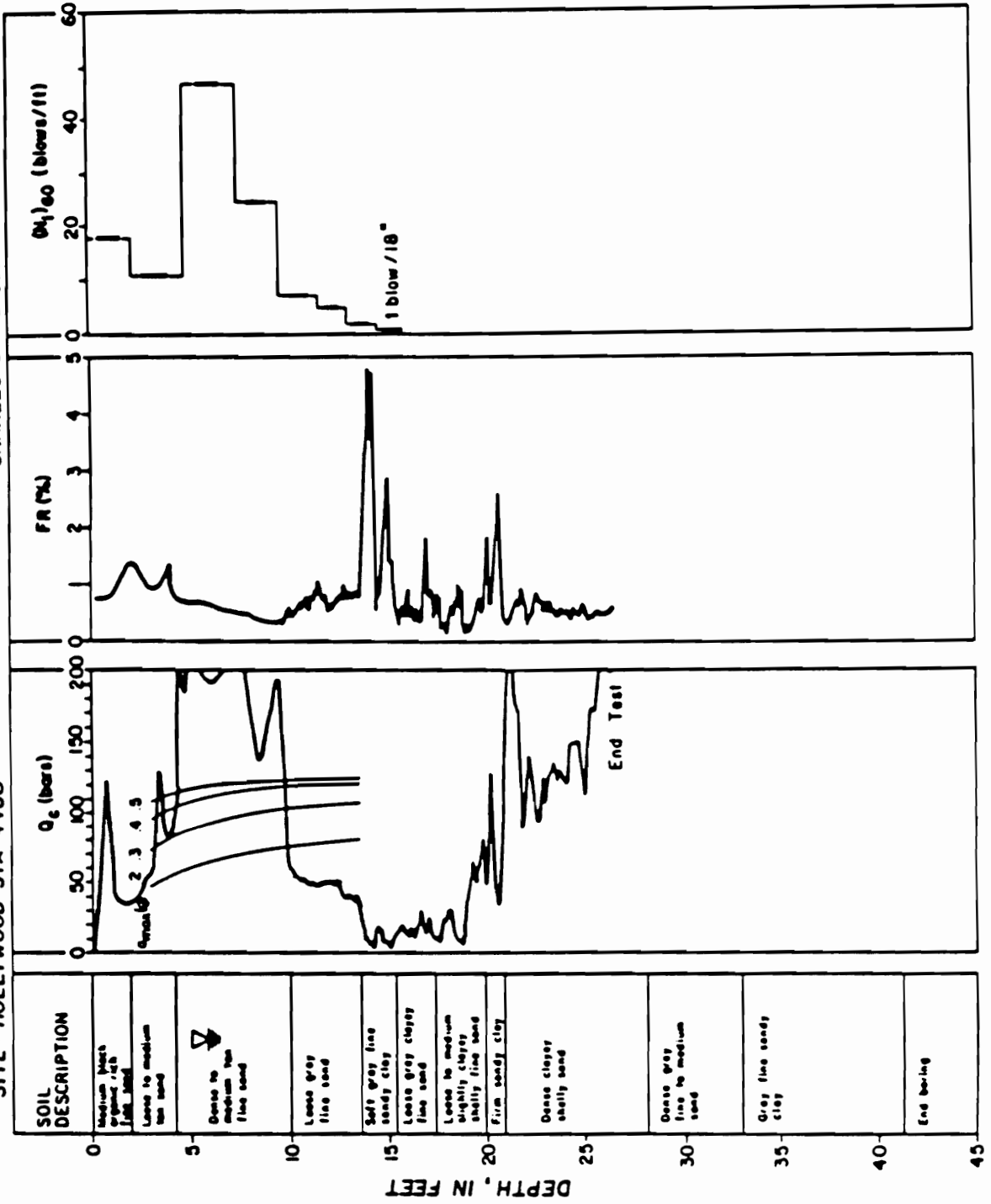
CHARLESTON LIQUEFACTION STUDY

SITE - HOLLYWOOD STA. 2550'

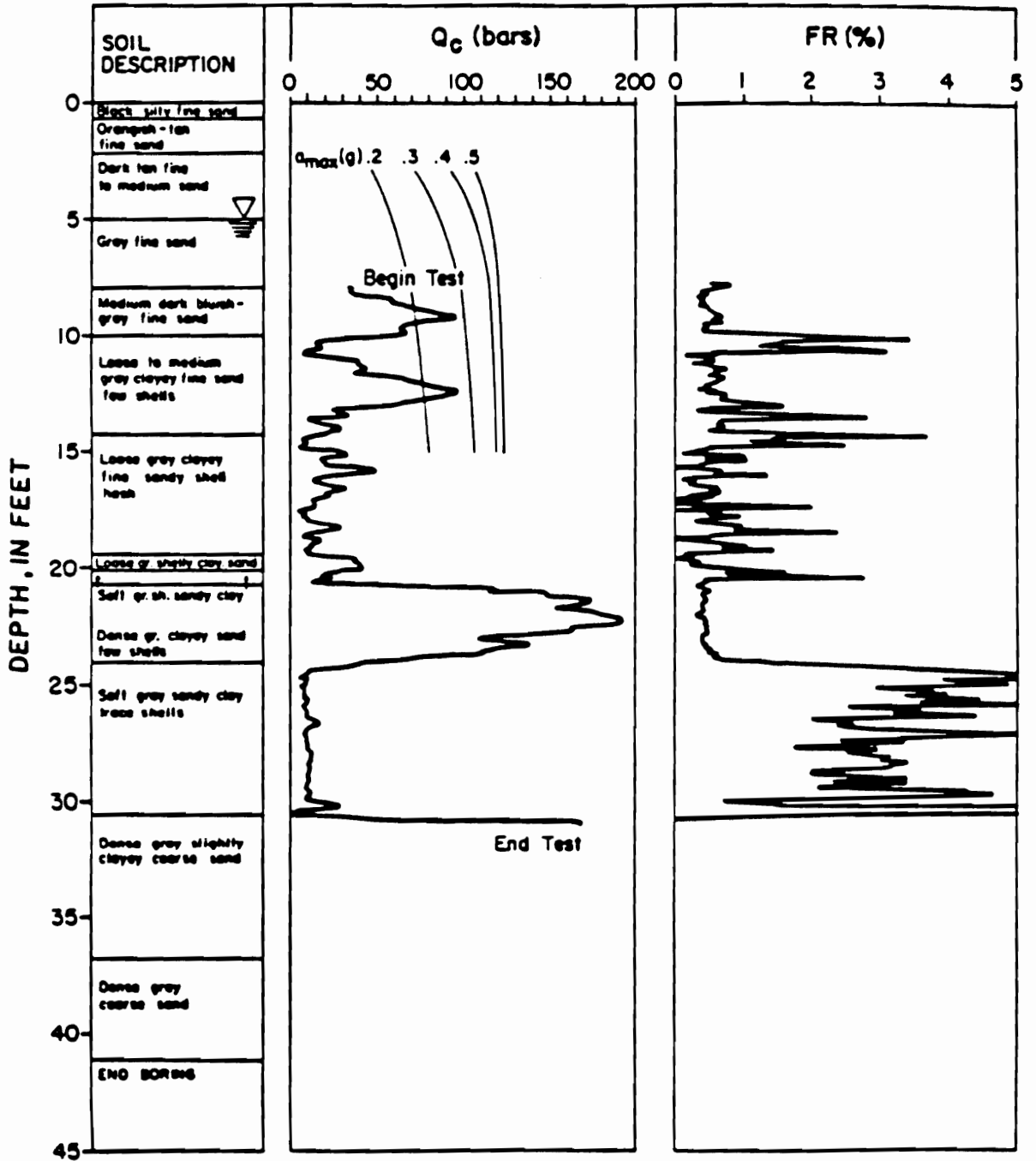


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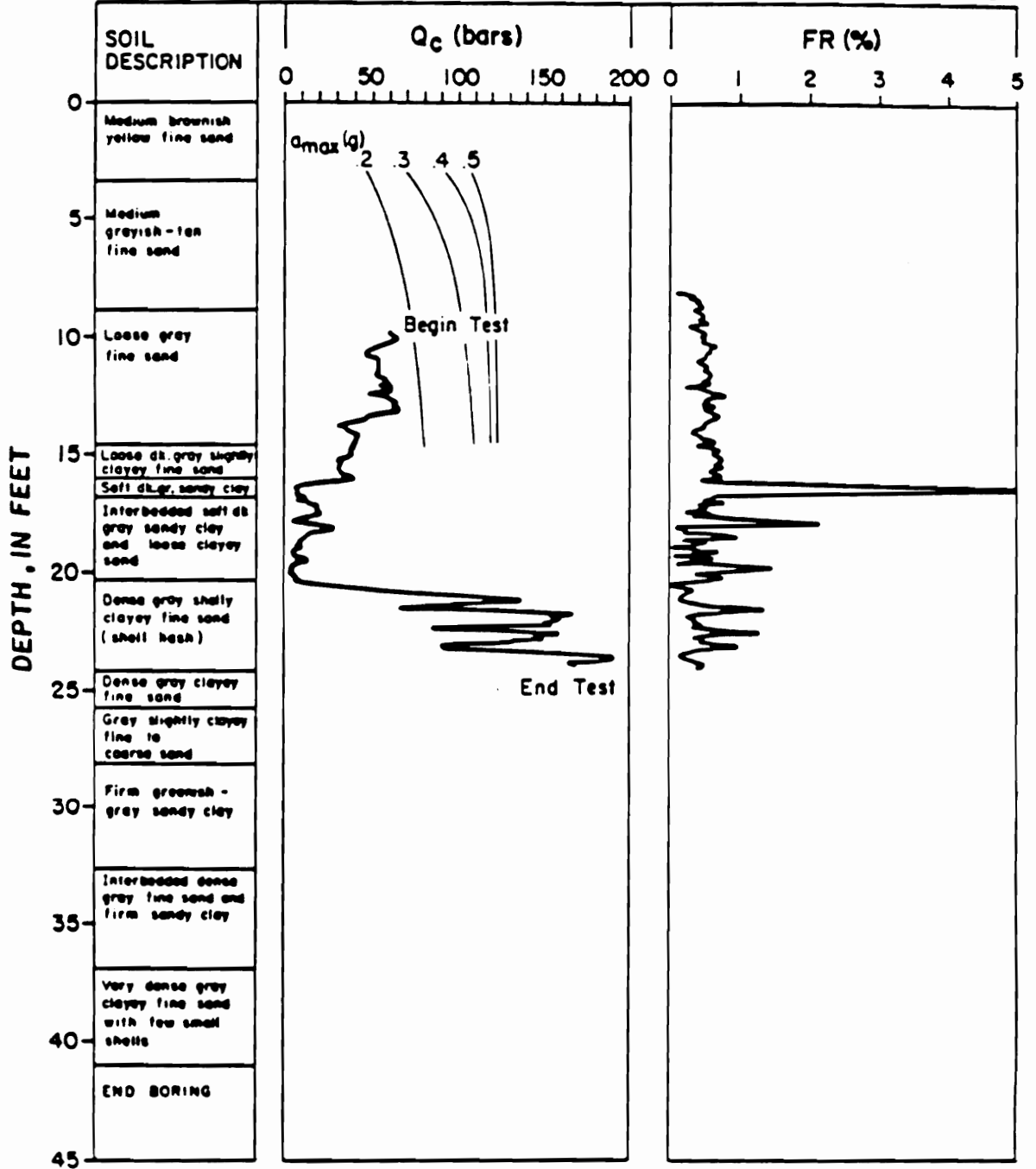
SITE : HOLLYWOOD STA 4480'



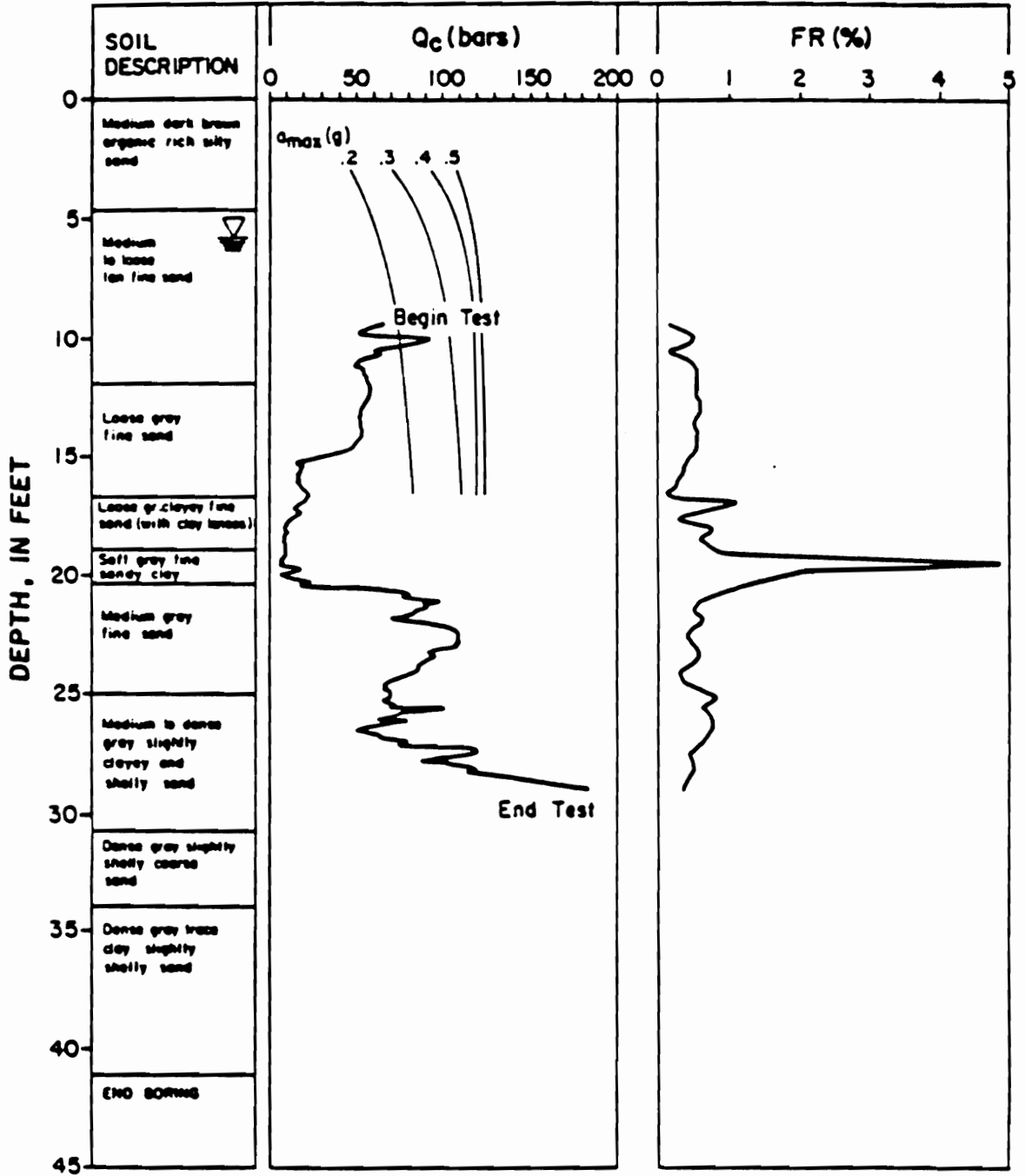
SITE: HOLLYWOOD STA. S0725' CHARLESTON LIQUEFACTION STUDY



SITE : HOLLYWOOD STA. N0560' CHARLESTON LIQUEFACTION STUDY

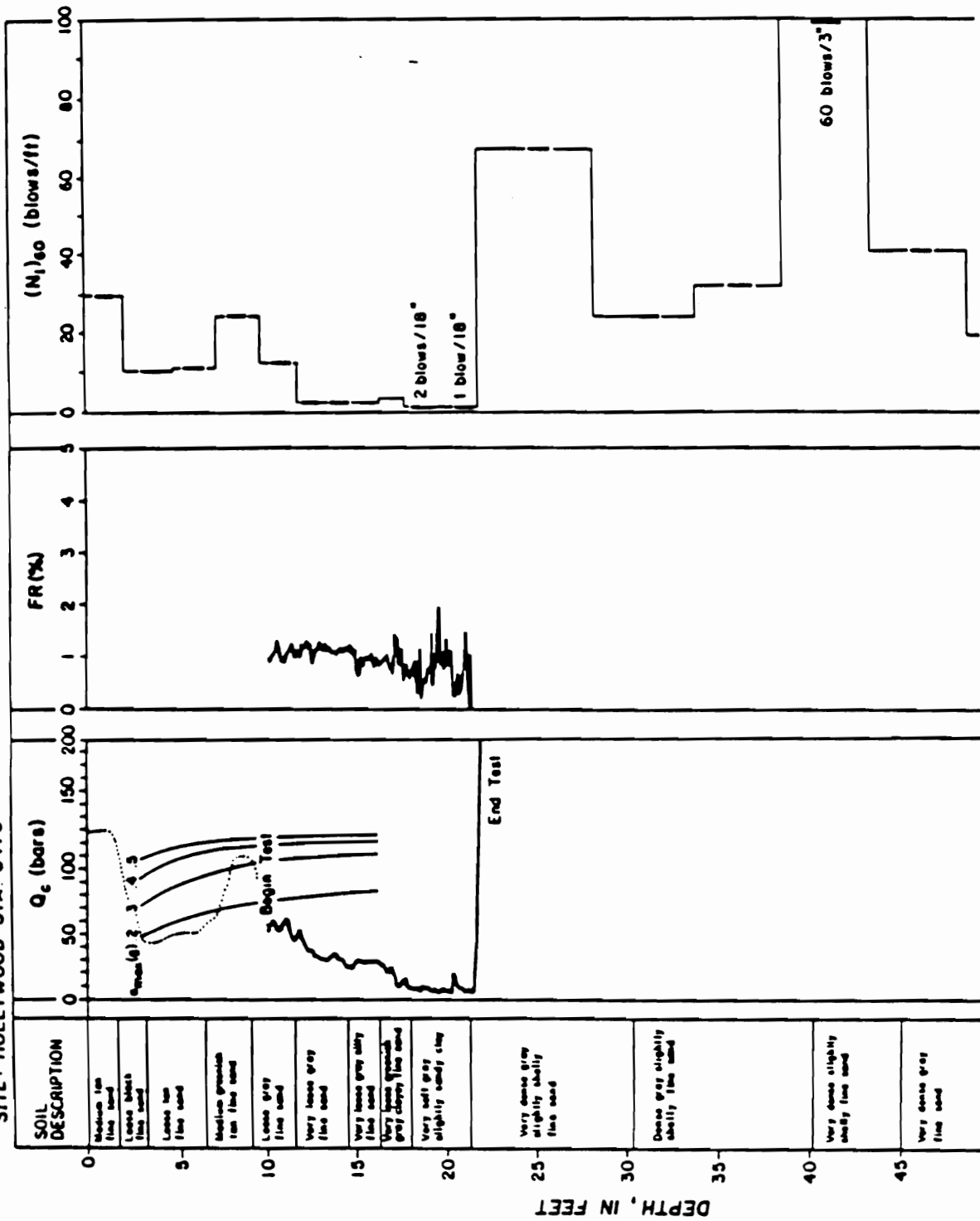


SITE : HOLLYWOOD STA. 5550' CHARLESTON LIQUEFACTION STUDY



CHARLESTON LIQUEFACTION STUDY

SITE: HOLLYWOOD STA. 6475



SOIL CONSULTANTS, INC. CHARLESTON, S.C.

LOG of BORING

Project: Liquefaction Study For VPI, Hollywood, SC
 Boring No. N-100 S. C. I. Project No. 87179 Date 11-25-87
 Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. _____
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	From	To	From	To		
1	0'0"	1'6"	0'0"	2'0"	Medium brown fine sand.	3-4-8
2	2'6"	4'0"	2'0"	4'6"	Loose brown fine sand.	2-3-3
3	5'0"	6'6"	4'6"		Medium tan fine sand.	7-10-14
4	7'6"	9'0"			Medium gray fine sand.	4-7-10
5	10'0"	11'6"			Medium gray fine sand.	5-5-6
6	12'6"	14'0"			Loose gray fine sand.	4-5-3
7	15'0"	16'6"			Very soft gray inorganic clay with slight sand content.	1-1-1
8	16'6"	18'6"			Shelby Tube	
					Pushed 24"	
					Recovered 18"	
9	18'6"	20'6"			Shelby Tube	
					Pushed 24"	
					Recovered 21"	
10	38'0"	39'6"			Shelby Tube	
					Pushed 18"	
					Recovered 17"	

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

Remarks: Location, depth & sampling as directed by Client.

SOIL CONSULTANTS, INC.
CHARLESTON, S.C.

MYRTLE BEACH, S.C.

SAVANNAH, GA.

LOG of BORING

Project: Liquefaction Study For VPI, Hollywood, SC

Boring No. 2500 S. C. I. Project No. 87179 Date 11-24-87

Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. _____
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 Inches
	From	To	From	To		
1	0'0"	1'6"	0'0"		Medium brown fine sand.	3-4-8
2	2'6"	4'0"		4'6"	Medium brown fine sand.	4-11-13
3	5'0"	6'6"	4'6"	7'0"	Dense brown fine sand.	11-20-19
4	7'6"	9'0"	7'0"		Medium gray fine sand.	8-7-7
5	10'0"	11'6"			Medium gray fine sand.	5-9-12
6	11'6"	13'0"		13'0"	Medium gray fine sand.	6-7-8
7	13'0"	14'6"	13'0"		Very soft gray inorganic clay with slight sand content.	1-1-1
8	16'0"	17'6"		22'0"	Very soft gray inorganic clay with slight sand and shell content.	1-1-1
9	22'0"	23'6"	22'0"		Dense gray fine sand with very slight inorganic clay content.	15-17-20

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

Remarks: Location, depth & sampling as directed by Client.

SOIL CONSULTANTS, INC. CHARLESTON, S.C.

MYRTLE BEACH, S.C.

SAVANNAH, GA

LOG of BORING

Project: Liquefaction Study For VPI, Hollywood, SC
 Boring No. 4500 S. C. I. Project No. 87179 Date 11-24-87
 Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. _____
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	From	To	From	To		
1	0'0"	1'6"	0'0"	2'0"	Medium black and tan fine sand.	2-5-6
2	2'6"	4'0"	2'0"	4'6"	Loose black and tan fine sand.	2-3-4
3	5'0"	6'6"	4'6"	7'0"	Dense black and tan fine sand.	4-14-20
4	7'6"	9'0"	7'0"	9'6"	Medium gray fine sand.	9-9-10
5	10'0"	11'6"	9'6"	11'6"	Loose gray fine sand.	3-3-3
6	11'6"	13'0"	11'6"	13'0"	Very loose gray fine sand.	2-2-2
7	13'0"	14'6"	13'0"		Very soft gray inorganic clay with slight sand content.	3-1-1
8	14'6"	16'0"			Very soft gray inorganic clay with slight sand content.	1/18"

Testing and Sampling in accordance with ASTM D. 1586-87 (1974)

Remarks: Location, depth & sampling as directed by Client.

SOIL CONSULTANTS, INC.
CHARLESTON, S.C.

MYRTLE BEACH, S.C.

SAVANNAH, GA

LOG of BORING

Project: Liquefaction Study For VPI, Hollywood, SC

Boring No. 6500 S. C. I. Project No. 87179 Date 11-23-87

Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. _____
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per & Inches
	From	To	From	To		
1	0'0"	1'6"	0'0"		Medium tan fine sand with roots down	2-5-14
				2'0"	to 10"	
2	2'6"	4'0"	2'0"		Loose black fine sand.	3-3-4
3	5'0"	6'6"		7'0"	Loose tan fine sand.	2-4-4
4	7'6"	9'0"	7'0"	9'6"	Medium tannish gray fine sand.	5-9-10
5	10'0"	11'6"	9'6"	11'6"	Loose gray fine sand.	4-5-5
6	11'6"	13'0"	11'6"		Very loose gray fine sand.	3/16" - 1/12"
7	13'0"	14'6"			Very loose gray fine sand.	1/12"-1/6"
8	14'6"	16'0"			Very loose gray fine silty sand.	1-1-1
9	16'0"	17'6"		17'6"	Very loose gray fine silty sand.	2-2-1
10	17'6"	19'0"	17'6"		Very soft gray inorganic clay with slight sand content.	2/18"
11	20'0"	21'6"		22'6"	Very soft gray inorganic clay with slight sand content.	1/18"
12	25'0"	26'0"	22'6"	29'6"	Very dense gray fine sand with slight shell content.	30-45
13	30'0"	31'6"	29'6"		Dense gray fine sand with slight shell content.	9-18-15
14	35'0"	36'6"		39'6"	Dense gray fine sand with slight shell content.	16-16-30
15	40'0"	41'9"	39'6"		Very dense gray fine sand with slight shell content.	40-60/3"
16	45'0"	46'6"		49'6"	Very dense gray fine sand.	20-27-37
17	50'0"	51'6"	49'6"		Dense gray fine sand with slight shell and phosphate content.	9-15-20
					Continued	

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

Remarks: ** Locally called Marl.

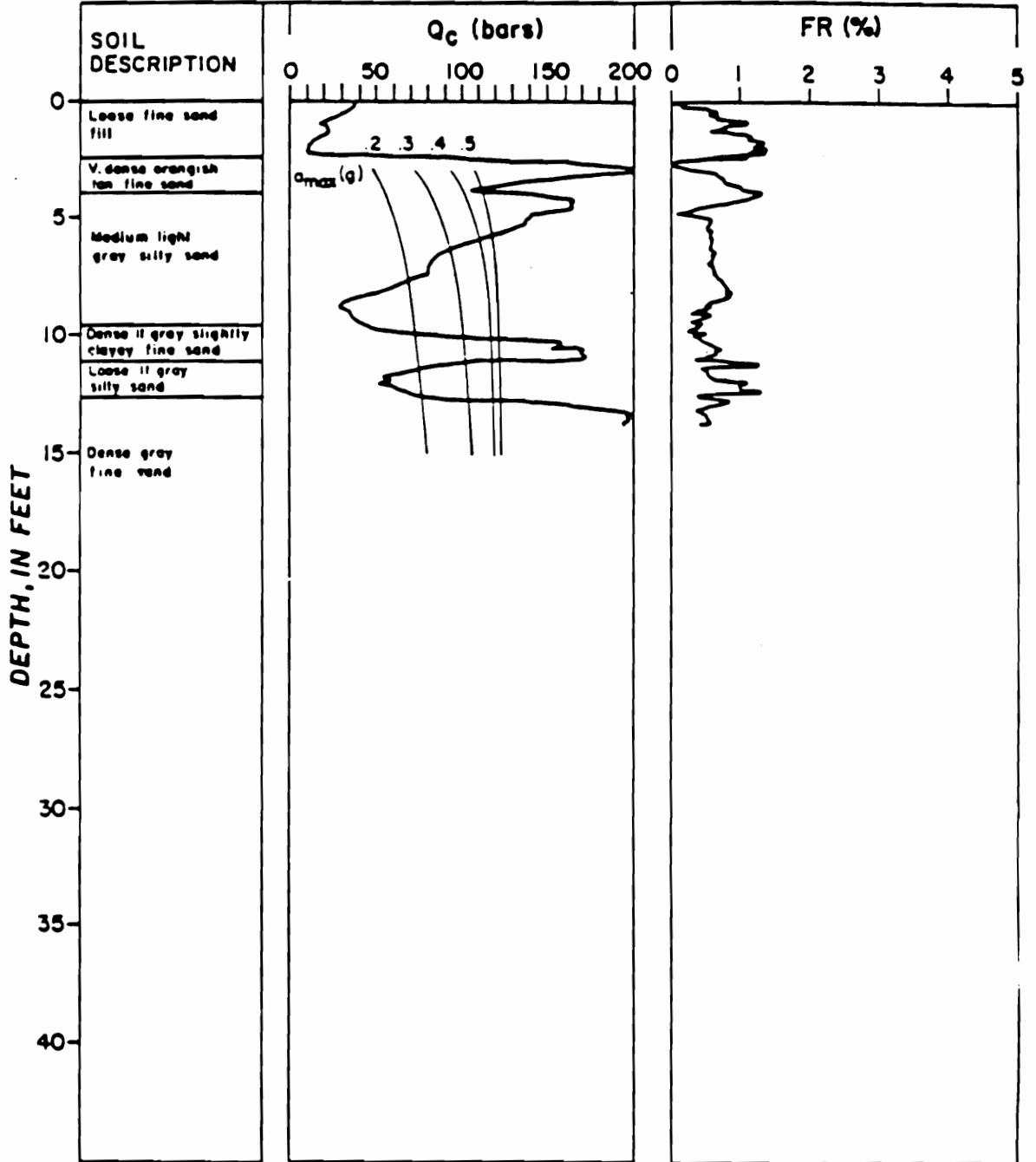
Location, depth & sampling as directed by Client.

WARREN SITE

CPT RECORDS

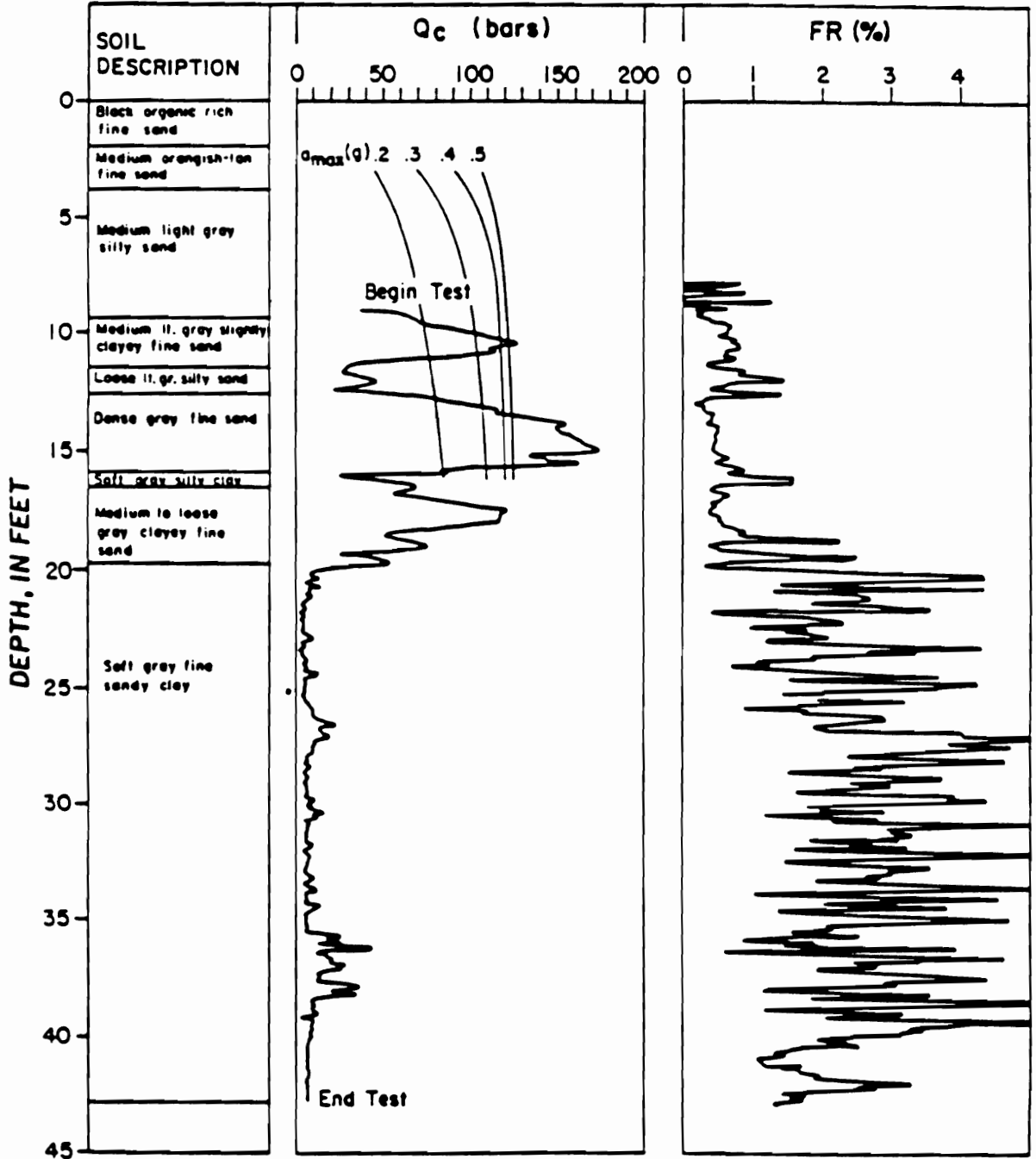
SITE: WARREN STA. W1

CHARLESTON LIQUEFACTION STUDY



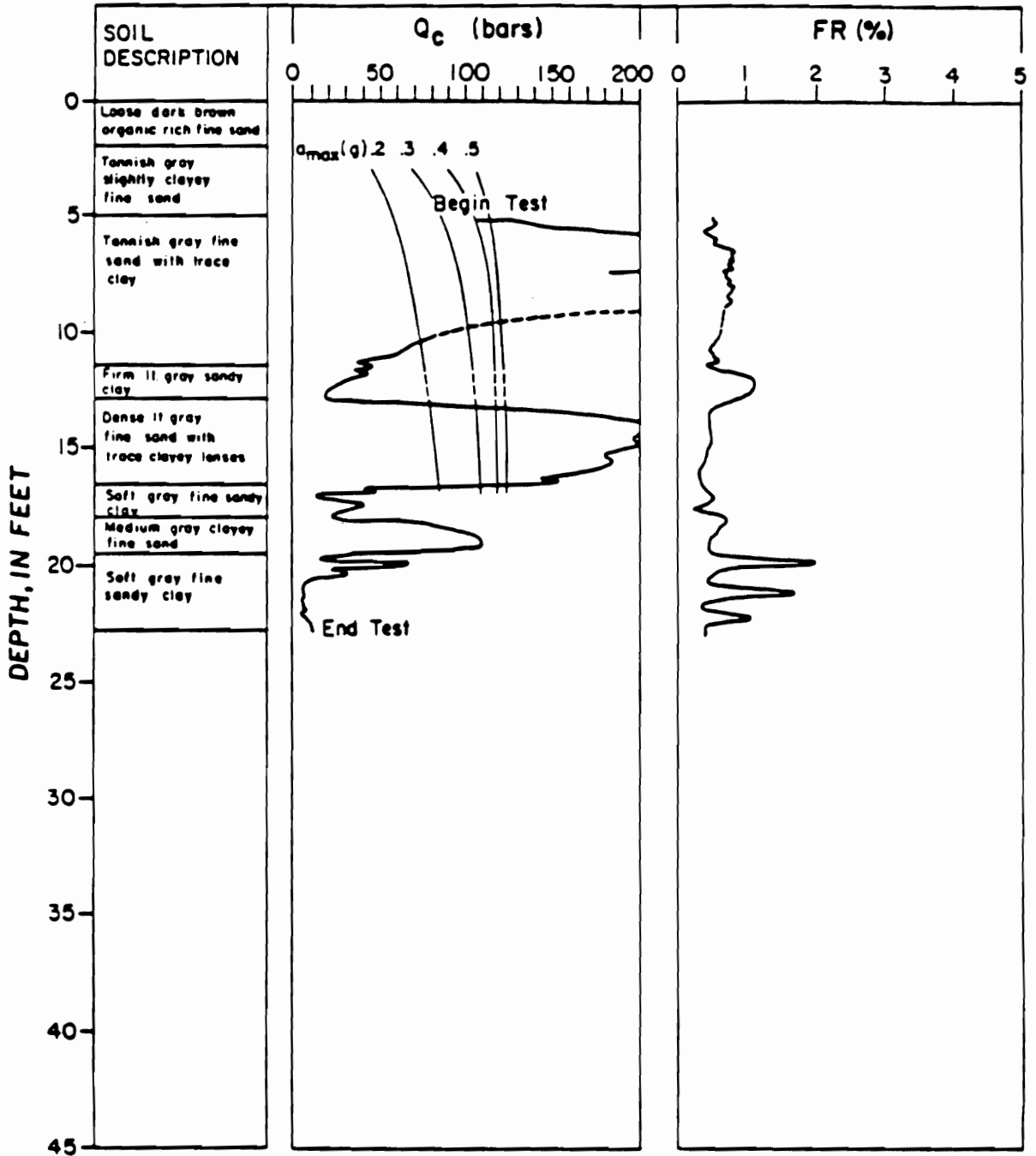
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CHARLESTON LIQUEFACTION STUDY



SITE: WARREN STA. W3

CHARLESTON LIQUEFACTION STUDY

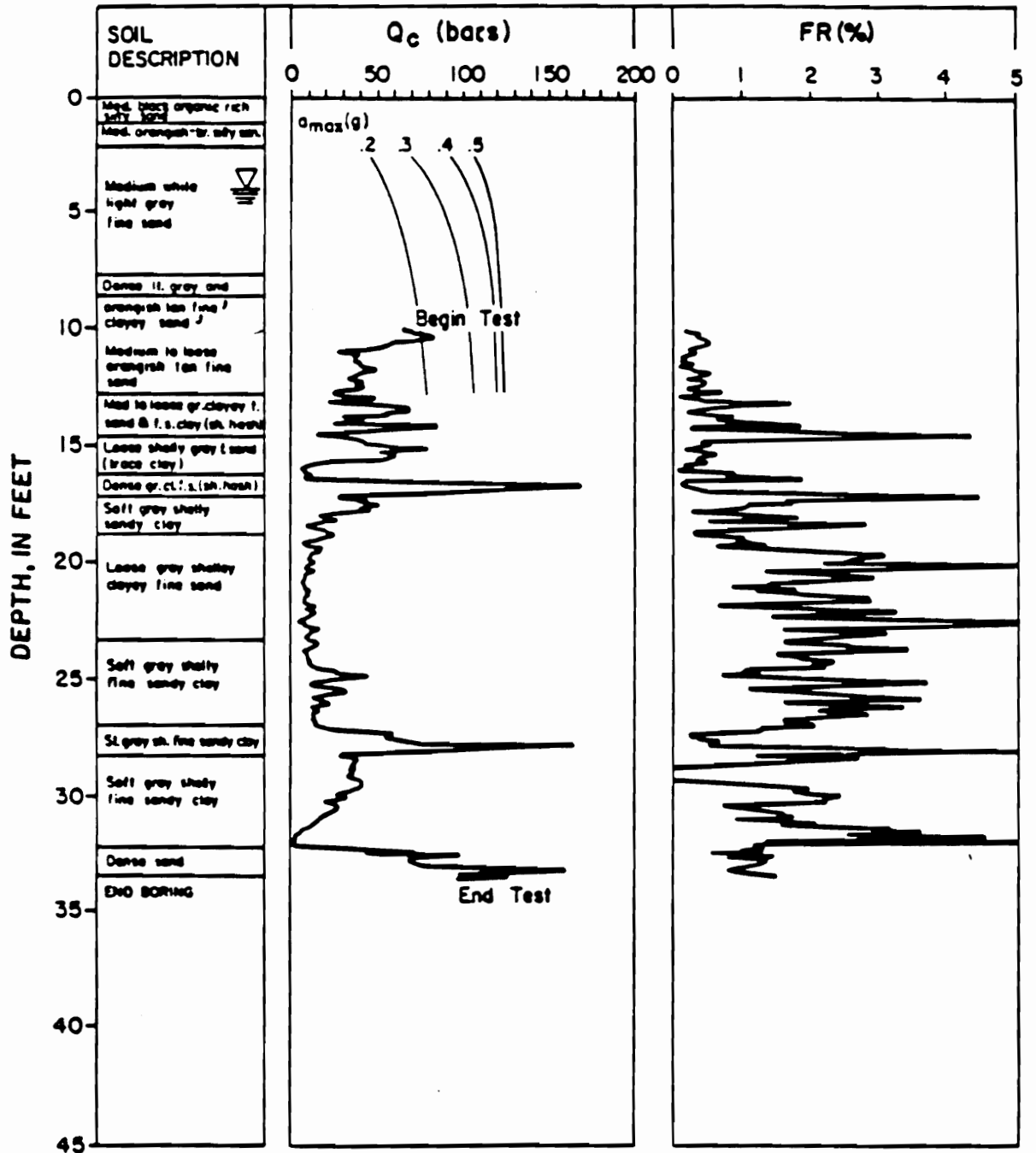


SOD FARM SITE

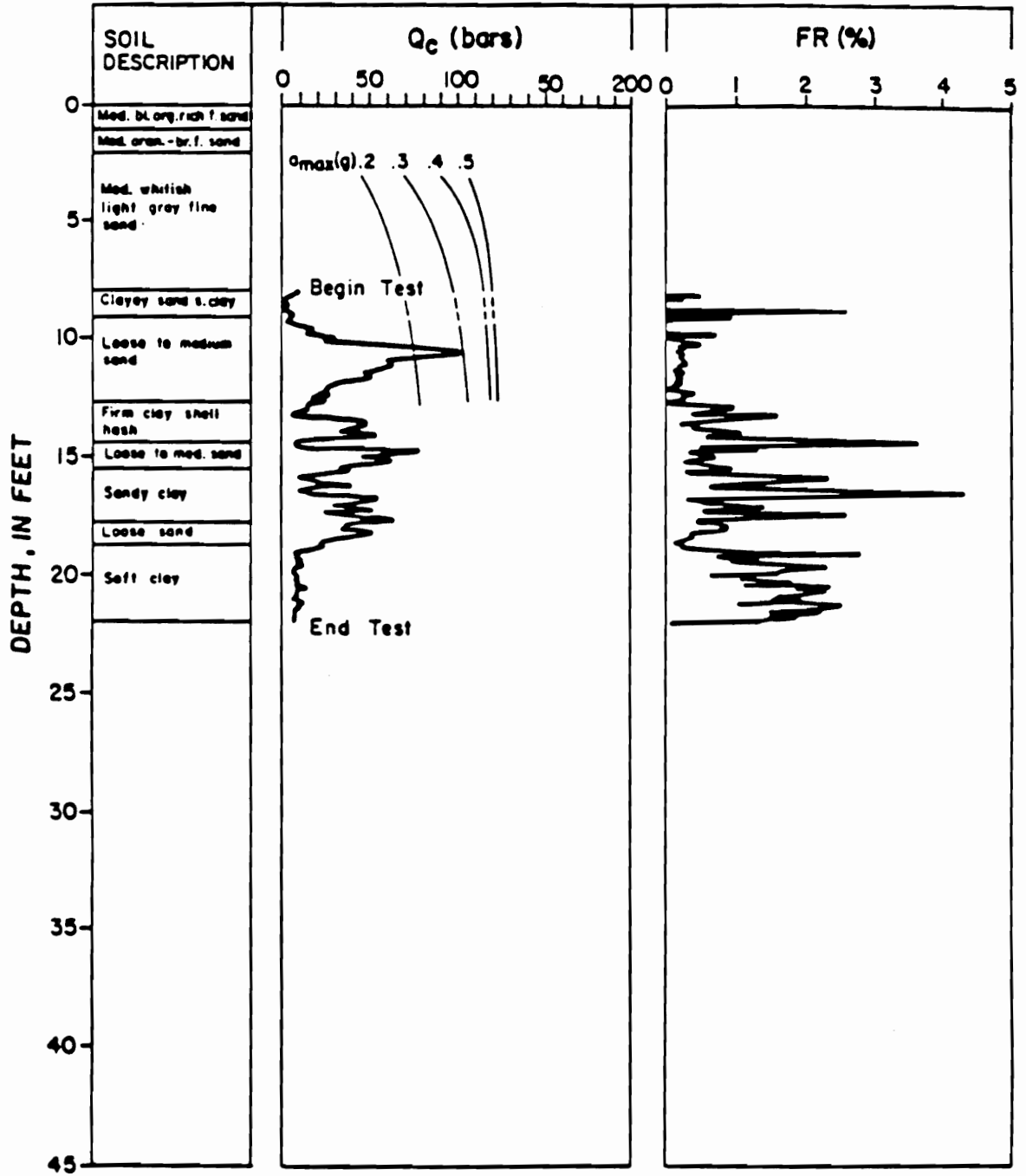
CPT RECORDS

SITE: SOD FARM STA. 400'

CHARLESTON LIQUEFACTION STUDY



SITE: SOD FARM STA. 600'

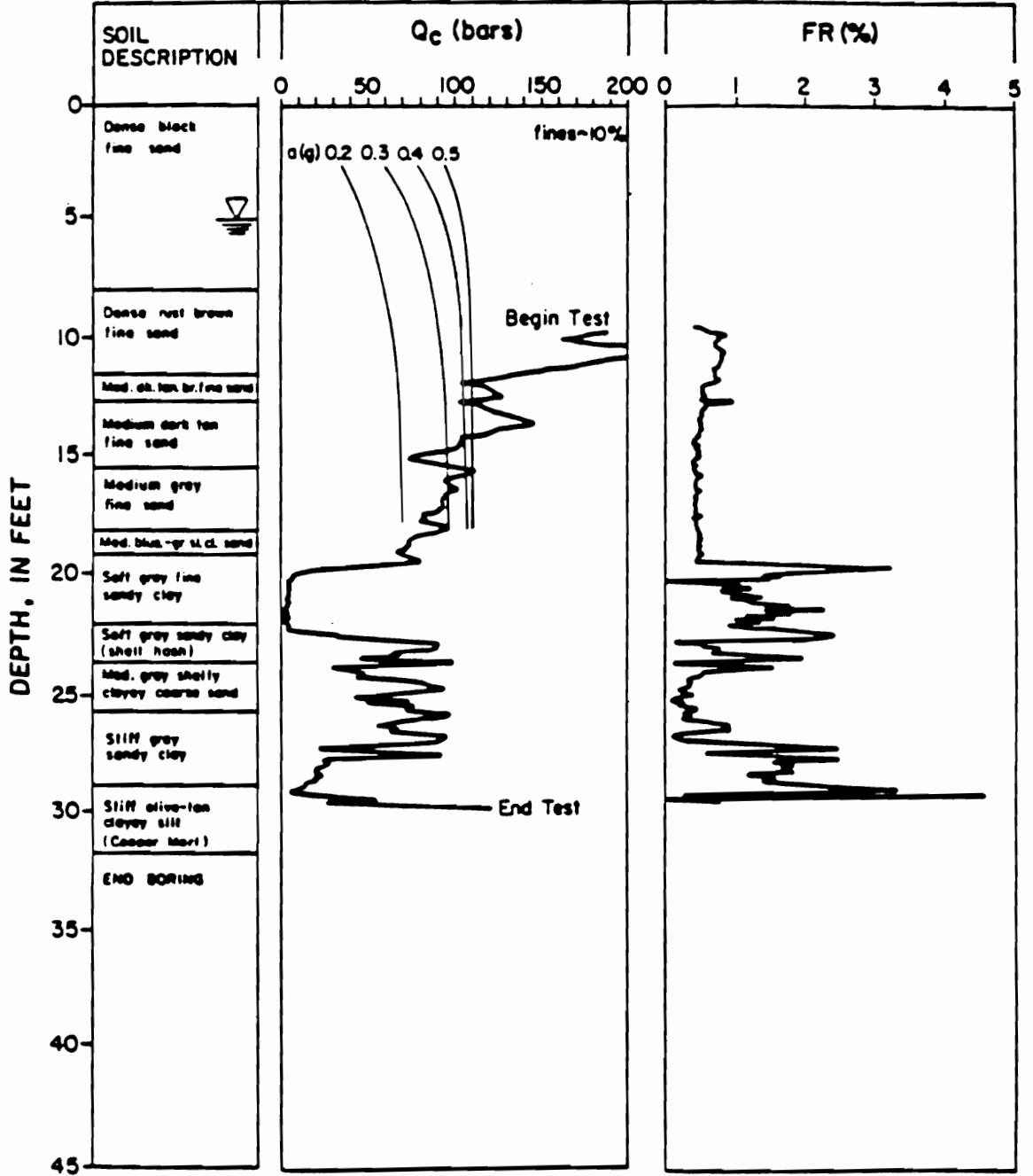


MONTAGUE SITE

CPT RECORDS

SITE: MONTAGUE

CHARLESTON LIQUEFACTION STUDY

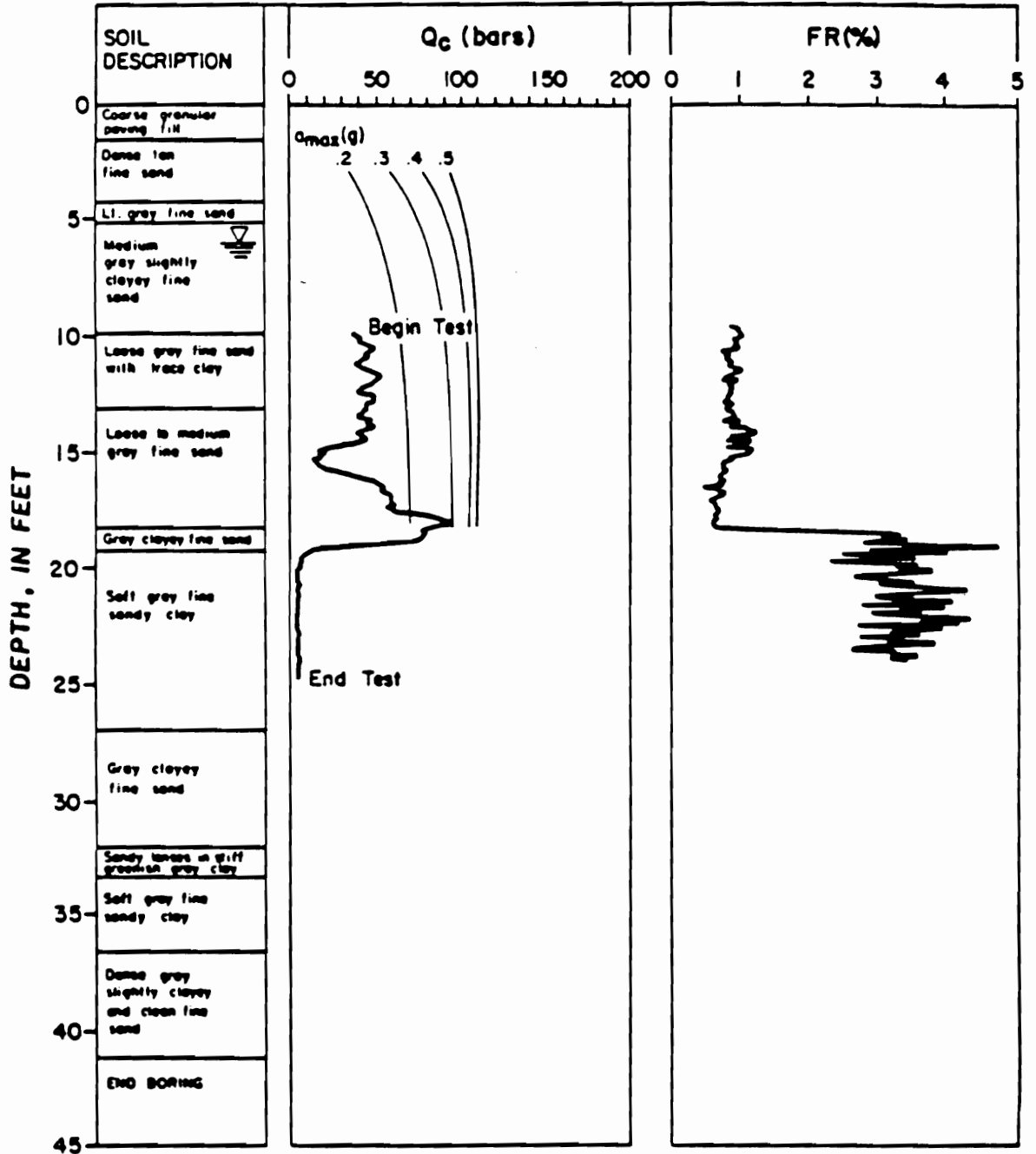


ELEVEN MILE POST SITE

CPT RECORDS

SITE : 11 MILE POST

CHARLESTON LIQUEFACTION STUDY

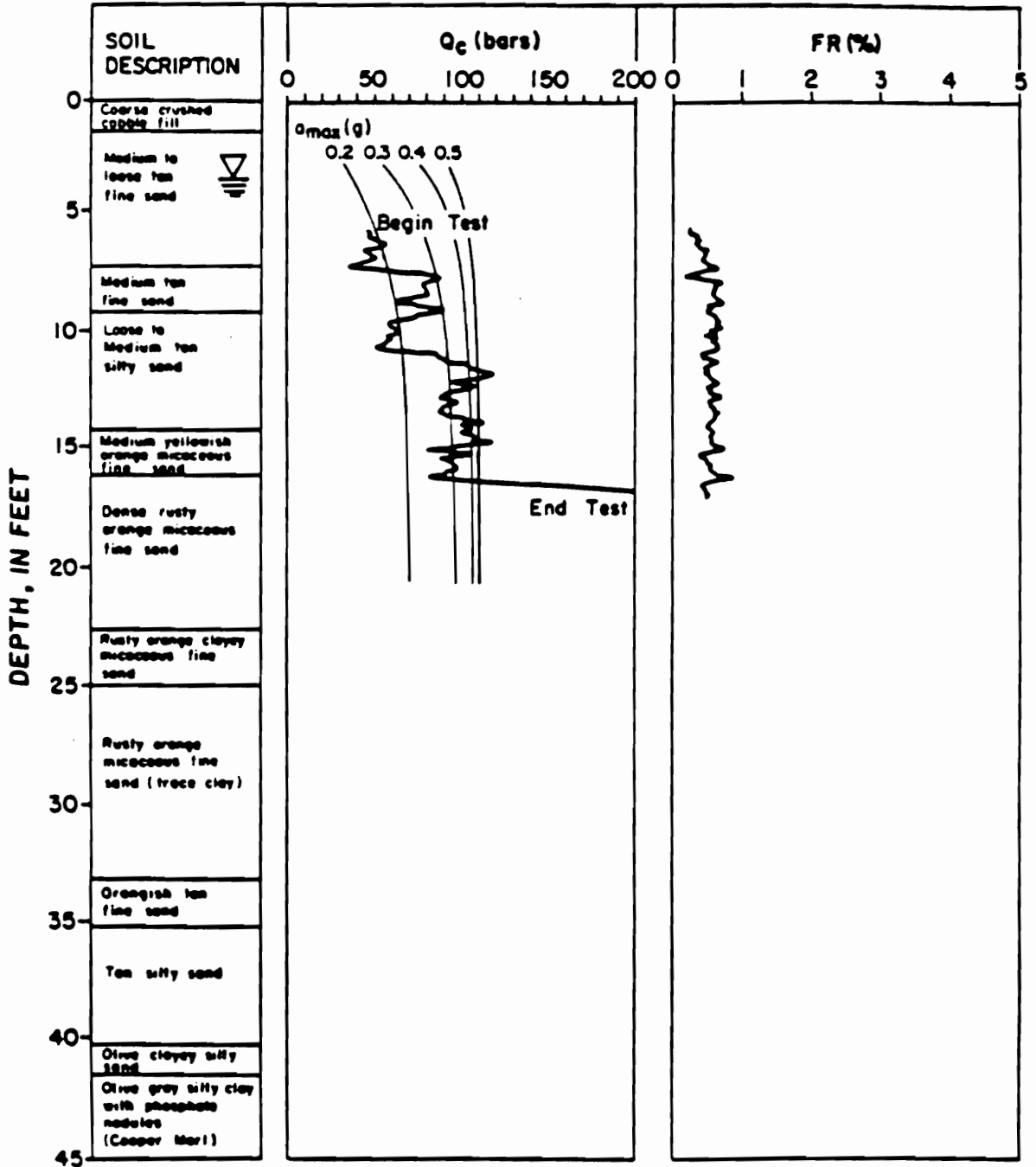


TEN MILE HILL SITES

CPT RECORDS

SITE : TEN MILE HILL

CHARLESTON LIQUEFACTION STUDY



APPENDIX B

CPT AND SPT RECORDS:

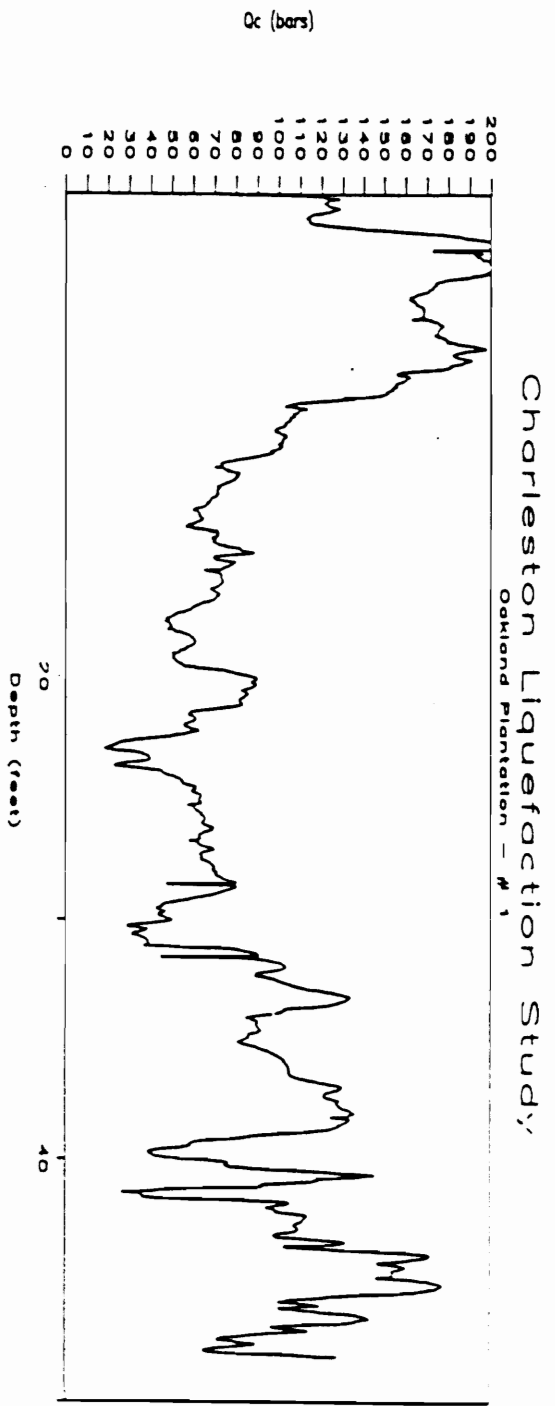
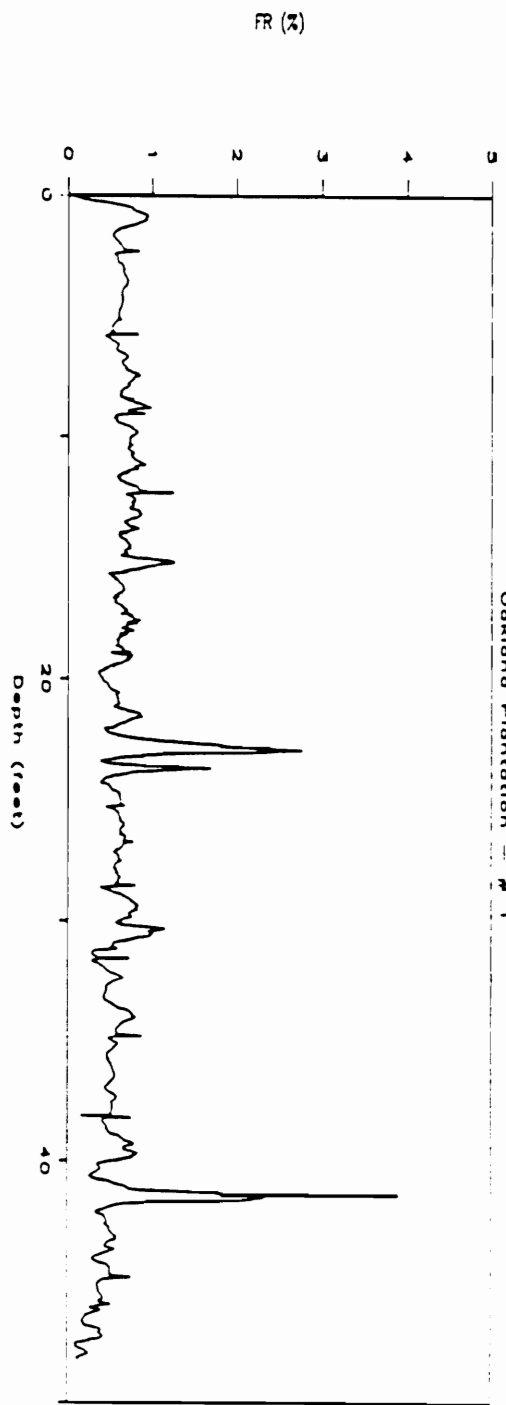
OAKLAND PLANTATION, MOUNT PLEASANT PITS, MARK CLARK EXPRESSWAY
BRIDGE, ST. JAMES-SANTEE SCHOOL, SOUTH TIBWIN, GEORGETOWN (KING &
PRINCE STREETS), PAWLEY'S ISLAND/PARKERSVILLE, AND EDISTO ISLAND

SITES

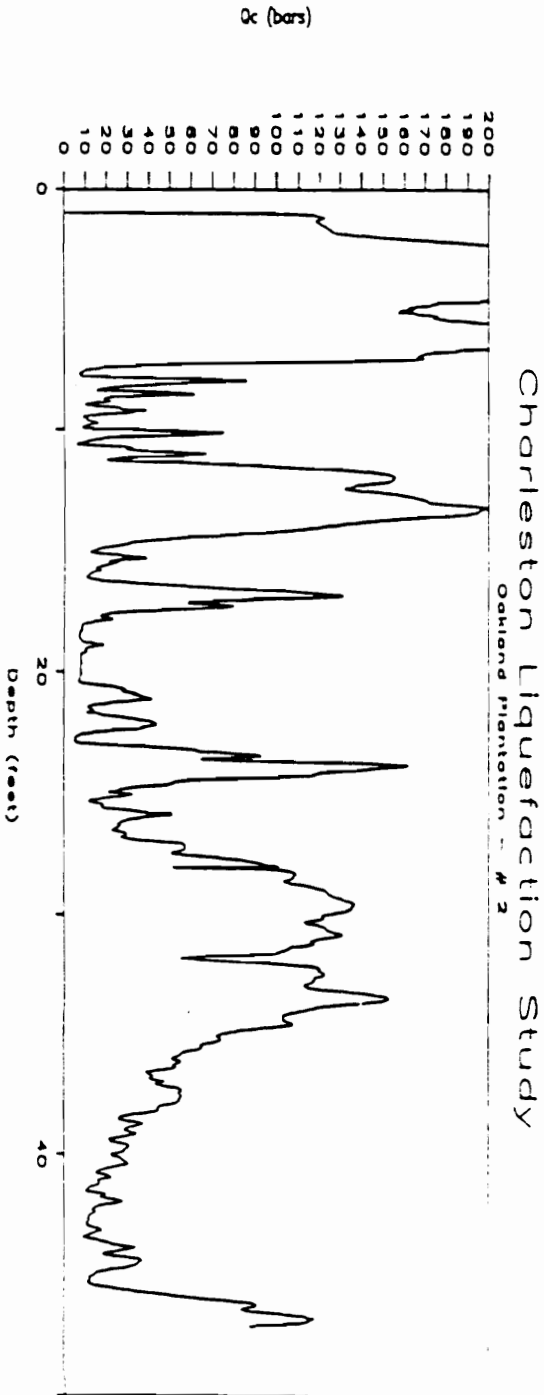
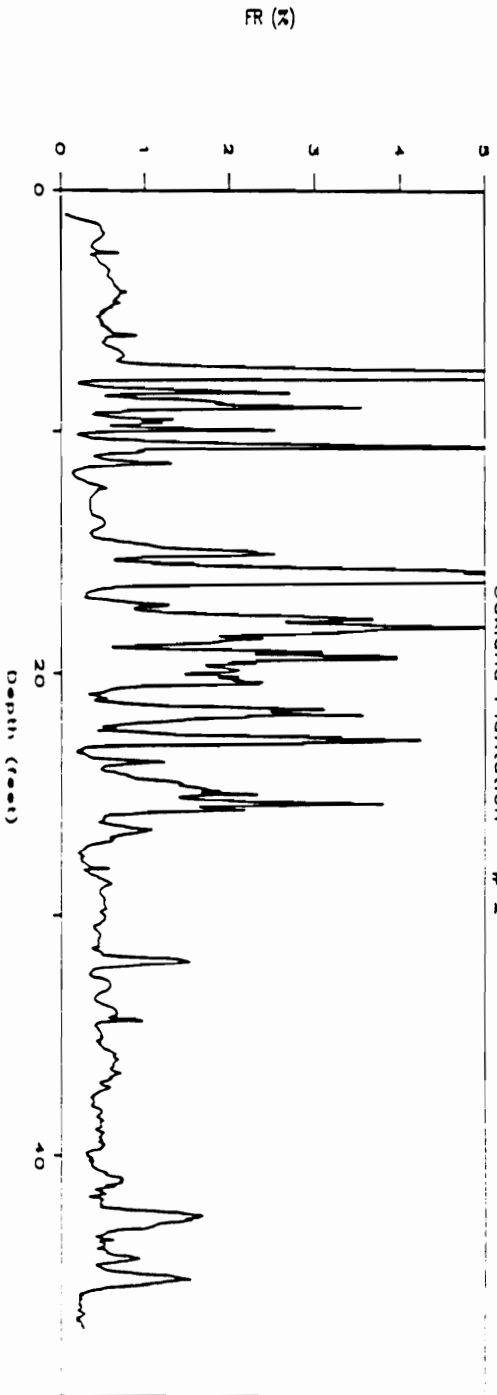
OAKLAND PLANTATION SITE

CPT AND SPT RECORDS

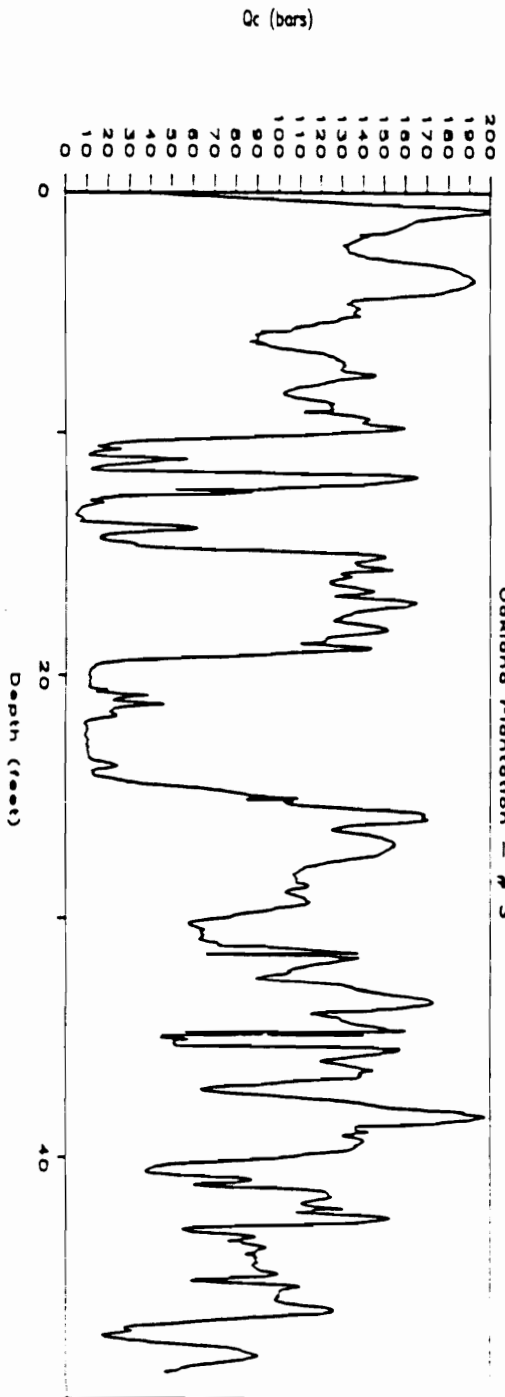
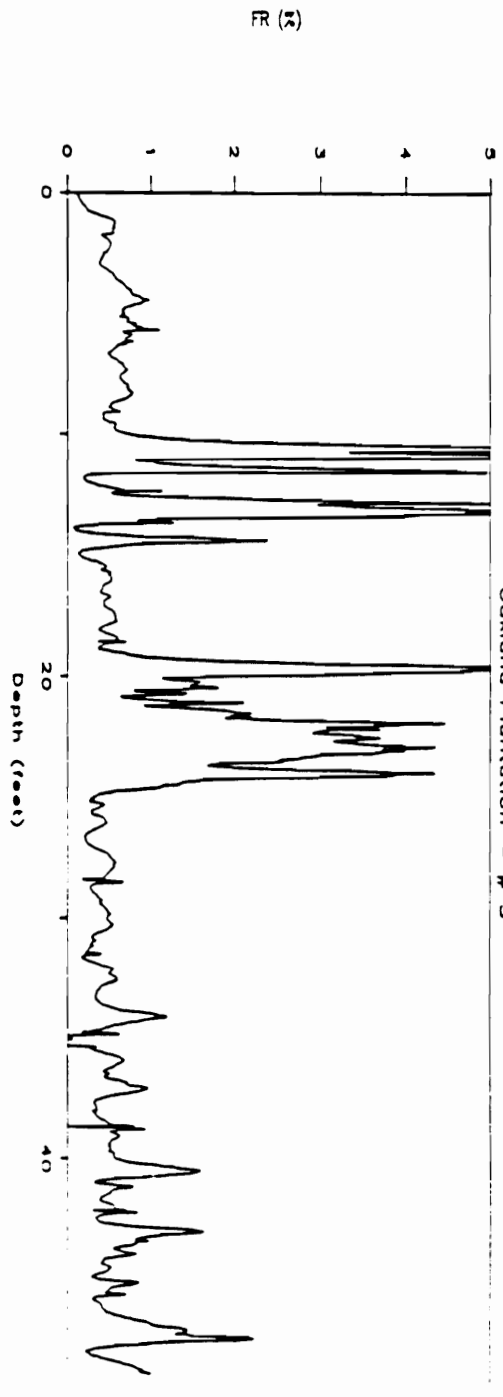
Charleston Liquefaction Study
Oakland Plantation - # 1



Charleston Liquefaction Study
Oakland Plantation - # 2

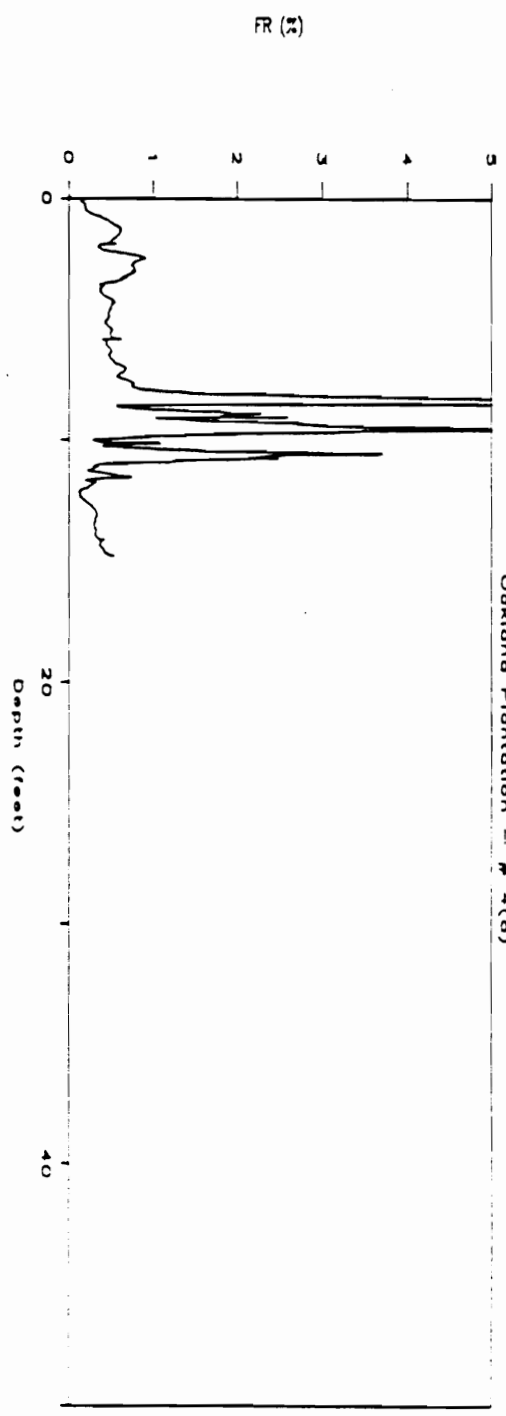


Charleston Liquefaction Study
Oakland Plantation - # 3

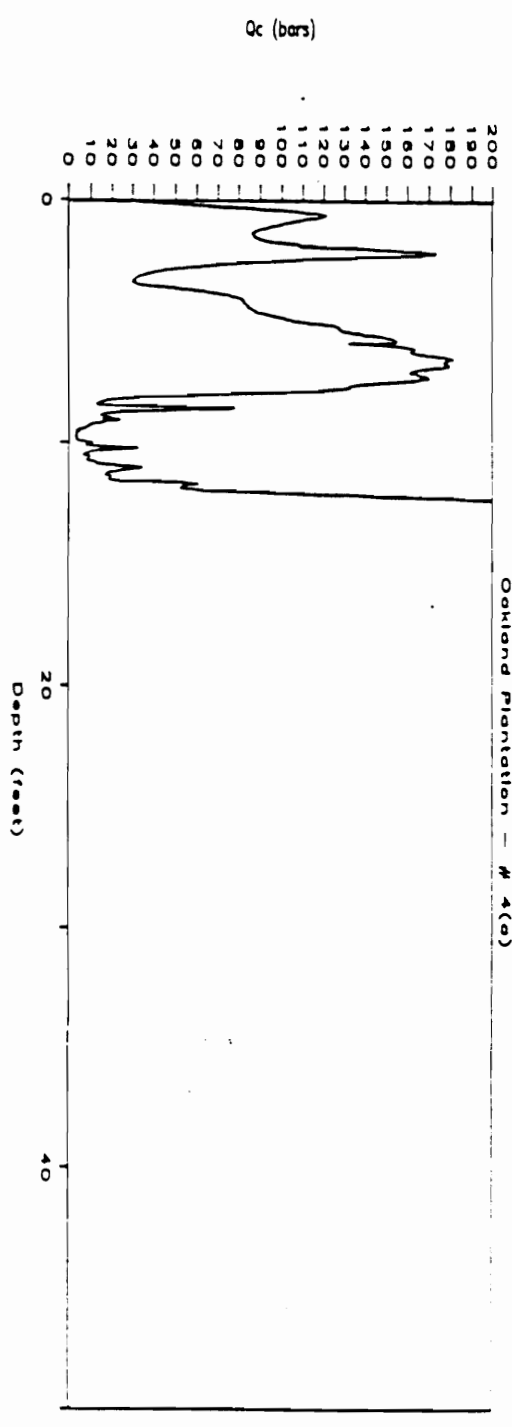


Charleston Liquefaction Study
Oakland Plantation - # 3

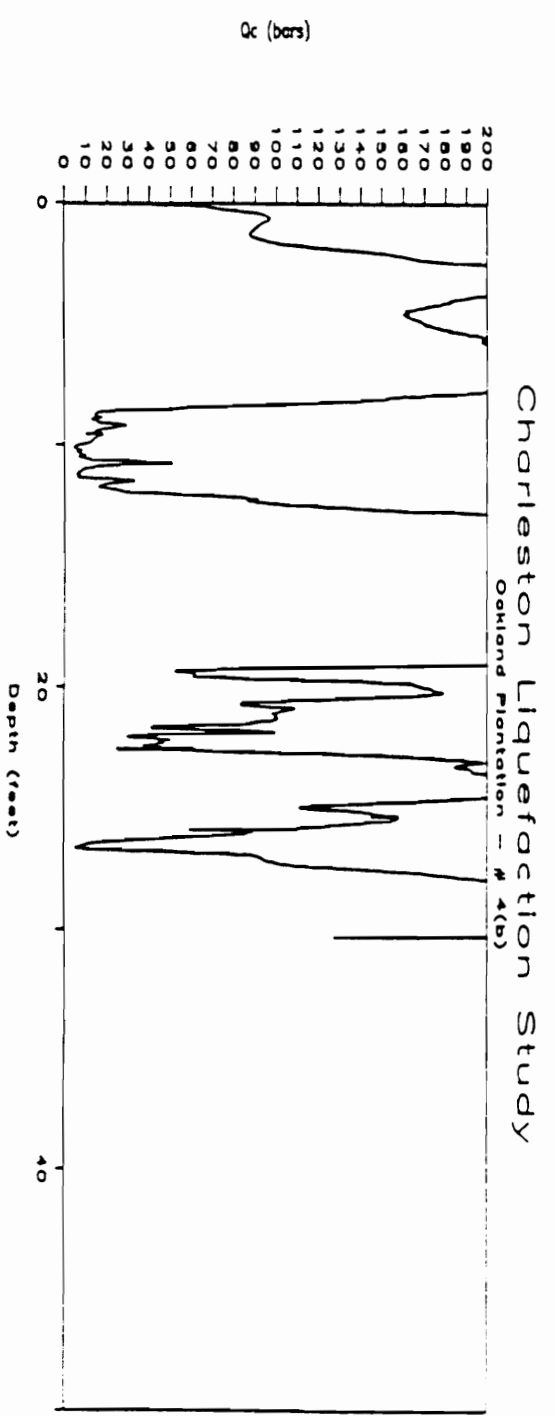
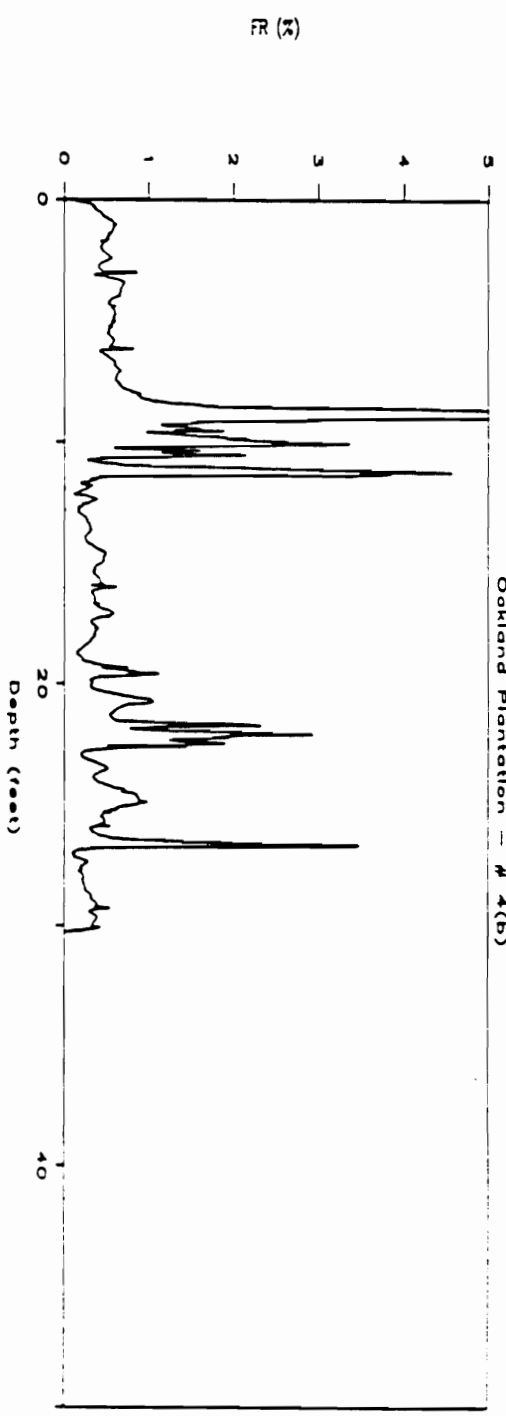
Charleston Liquefaction Study
Oakland Plantation - # 4(a)



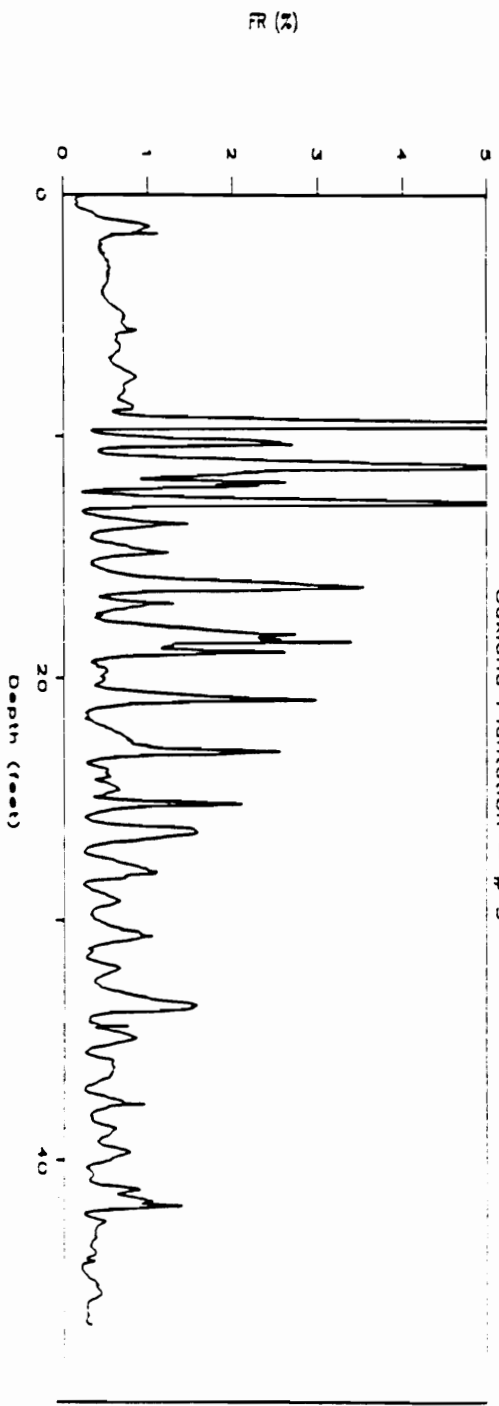
Charleston Liquefaction Study
Oakland Plantation - # 4(a)



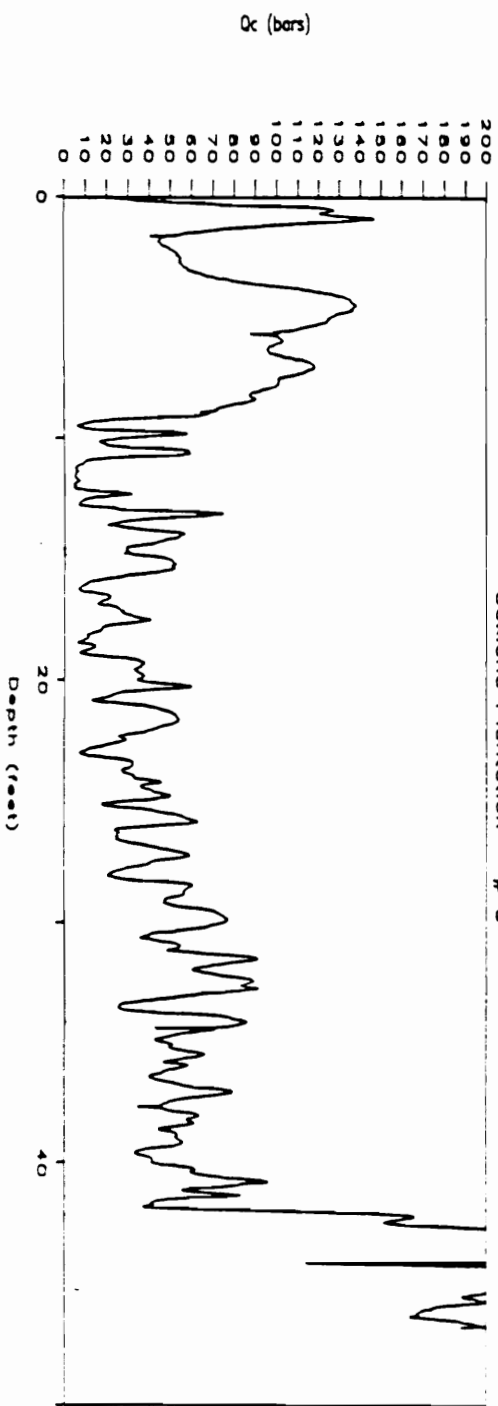
Charleston Liquefaction Study
Oakland Plantation - # 4(b)



Charleston Liquefaction Study
Oakland Plantation - # 5



Charleston Liquefaction Study
Oakland Plantation - # 5



LOG of BORING

Project: CHARLESTON LIQUEFACTION PROJECT - CAKLAND PLANTATION

Boring No. CAKLAND #5 S. C. I. Project No. Date 27 JULY 1988

Ground Surface Elev. Assumed 0.0" Datum Gr. Water Elev. 5'0"
(Minimum of 24 hrs. after completion)

Sample No.	SAMPLE		STRATUM		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	Elev. - Feet & Inches		Elev. - Feet & Inches			
	From	To	From	To		
1	1'0"	2'6"			LOOSE BLACK FINE SAND w/ORGANICS	6-4-4
2	2'6"	4'0"			MEDIUM TAN FINE SAND	3-4-7
3	5'0"	6'6"			MEDIUM TAN FINE SAND	A-6-7
4	7'6"	9'0"			MEDIUM TAN FINE SAND w/MICA	2-4-6
5	10'0"	11'6"			V. LOOSE GRAY FINE SILTY SAND	1-2-1
6	12'6"	14'0"			LOOSE GRAY MEDIUM SAND w/SHELL	1-4-6
7	15'0"	16'6"			V. LOOSE GRAY FINE SAND w/SHELL	2-2-1
8	17'6"	19'0"			V. LOOSE GRAY FINE SILTY SAND	1-1-1
9	20'0"	21'6"			LOOSE FINE GRAY SAND w/SLIGHT SHELL	1-3-2
10	25'0"	26'6"			LOOSE GRAY FINE SAND	1-2-4
11	30'0"	31'6"			LOOSE GRAY FINE SAND	3-5-2
12	35'0"	36'6"			LOOSE GRAY FINE SAND	3-4-4
13	40'0"	41'6"			MEDIUM GRAY FINE SAND	2-9-16
14	45'0"	46'6"			DENSE GRAY MEDIUM SAND	13-19-24
					DRILL TO COPPER MARL AT 70'	

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

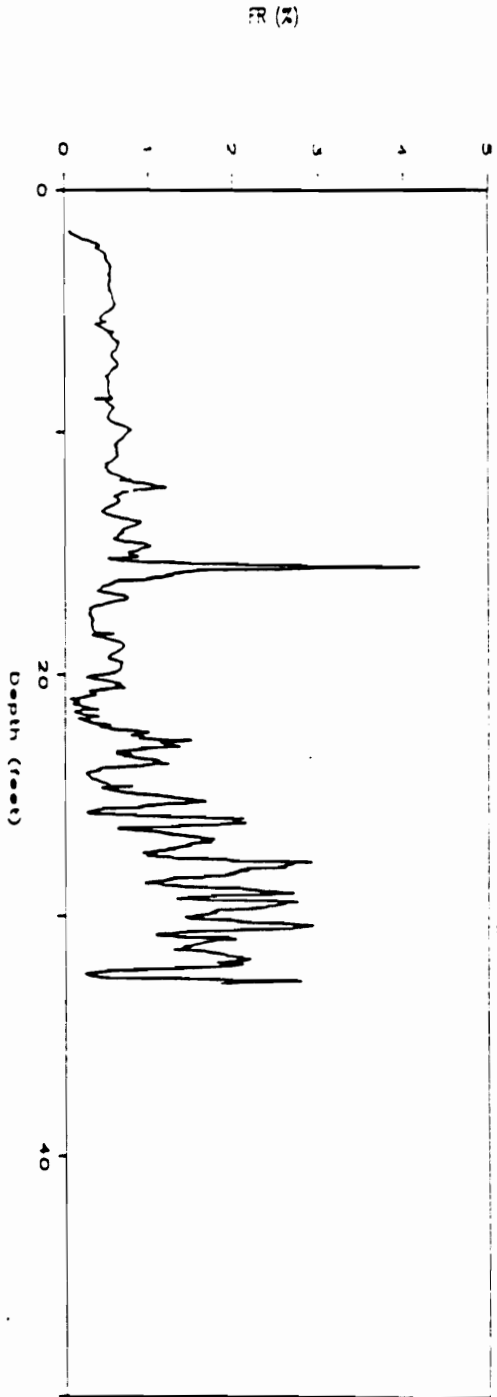
Remarks: DRILLING PERFORMED UNDER WATCHFUL EYE OF CURIOUS ALLIGATORS.

MT. PLEASANT PITS SITE

CPT AND SPT RECORDS

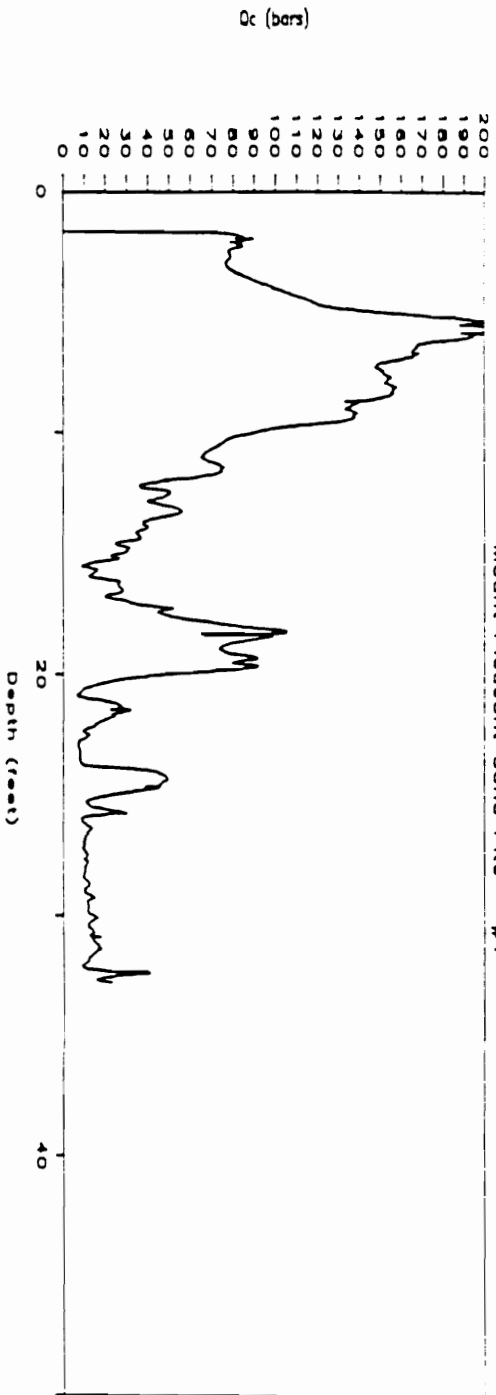
Charleston Liquefaction Study

Mount Pleasant Sand Pile - #1

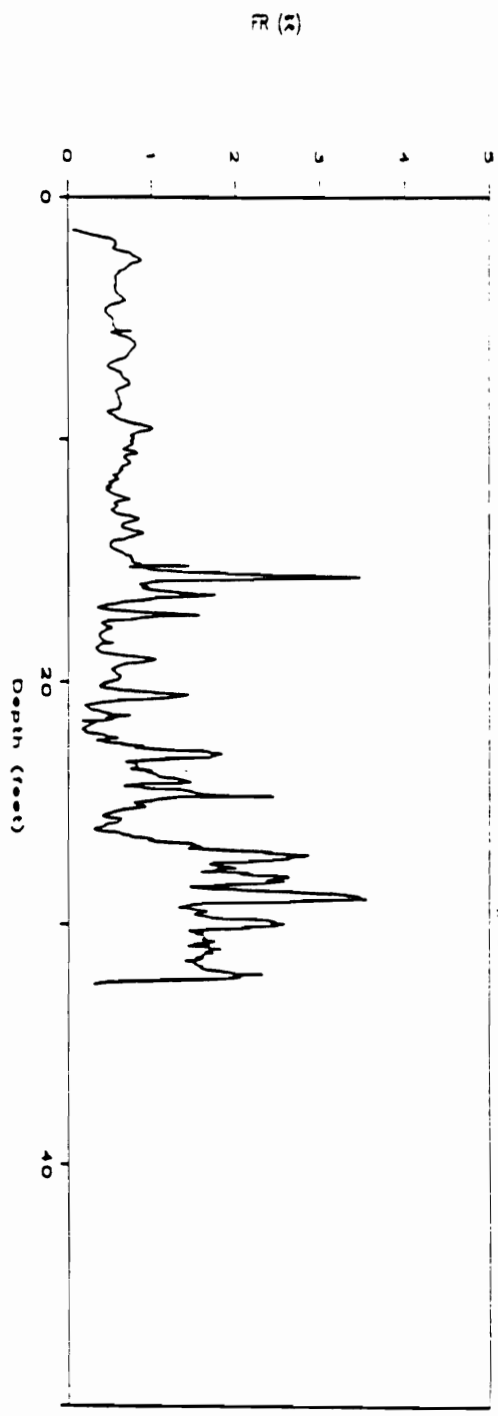


Charleston Liquefaction Study

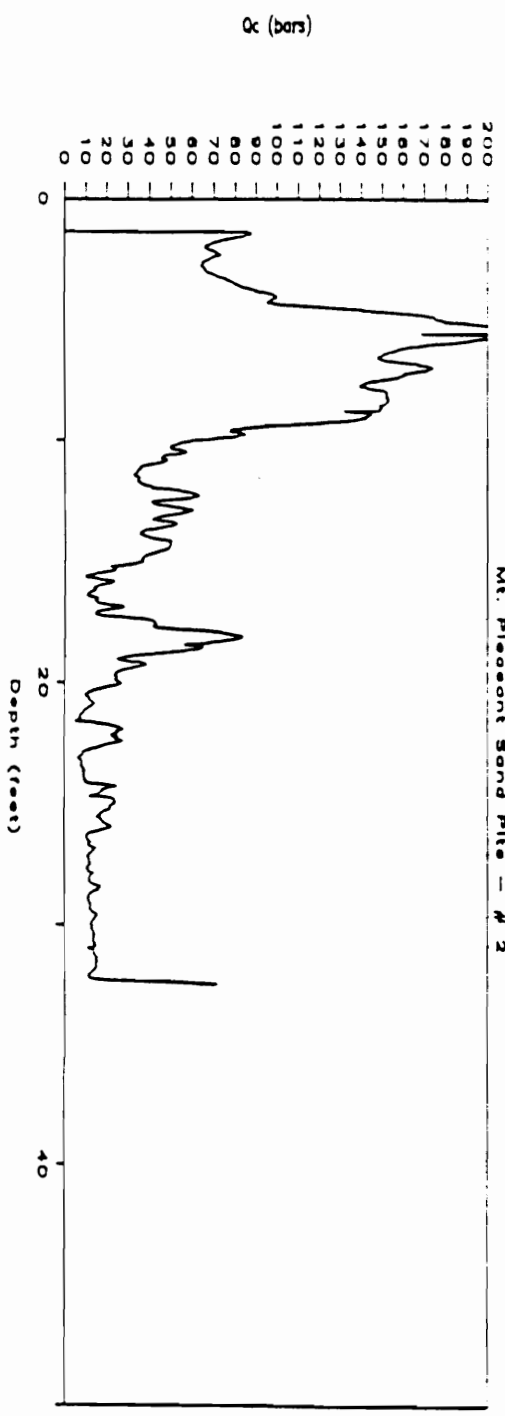
Mount Pleasant Sand Pile - #1



Charleston Earthquake Study
Mt. Pleasant Sand Pile - # 2

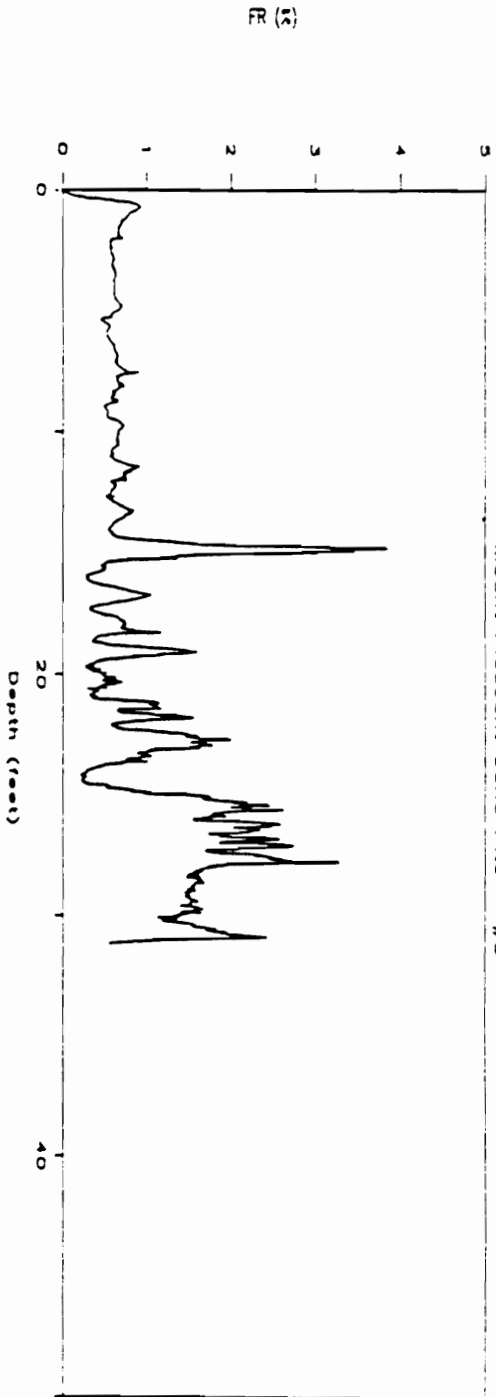


Charleston Earthquake Study
Mt. Pleasant Sand Pile - # 2



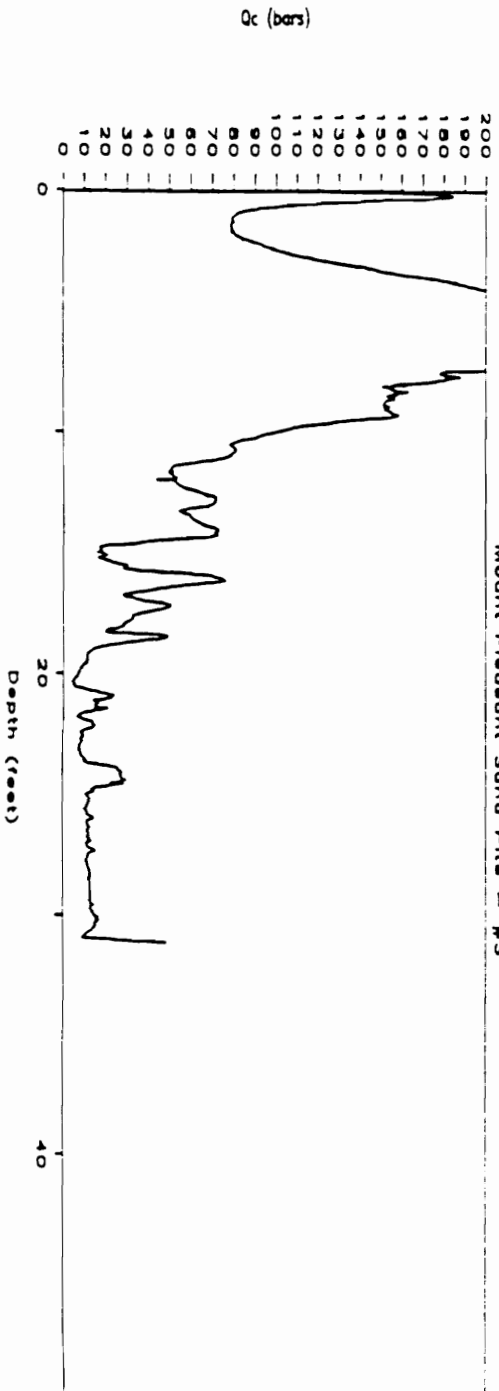
Charleston Liquefaction Study

Mount Pleasant Sand Pile - #3

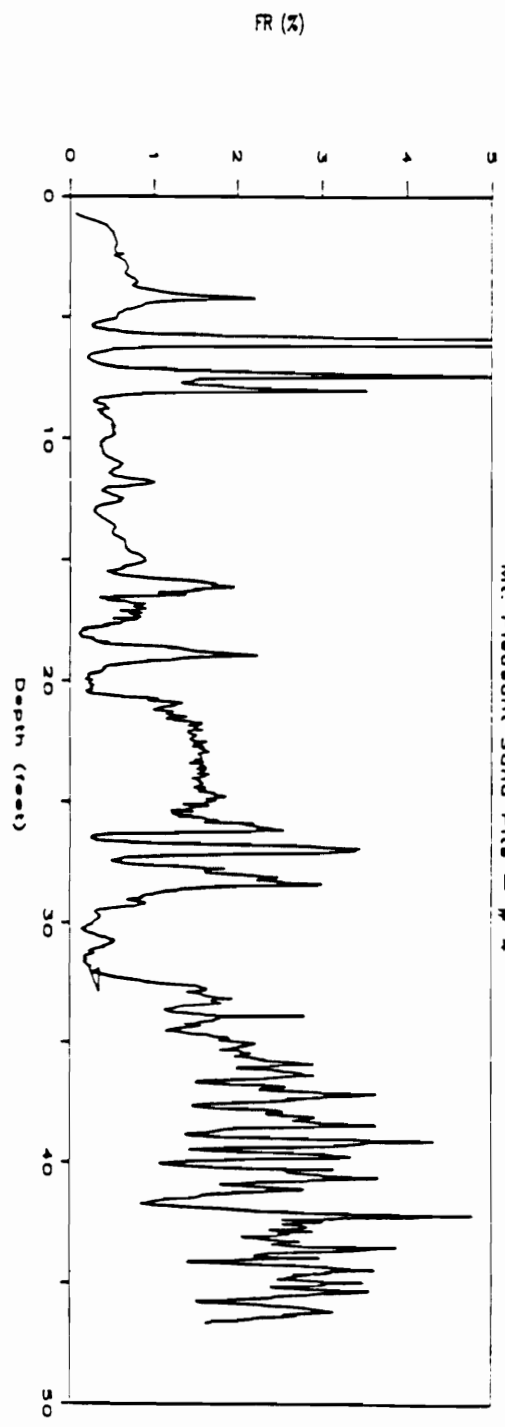


Charleston Liquefaction Study

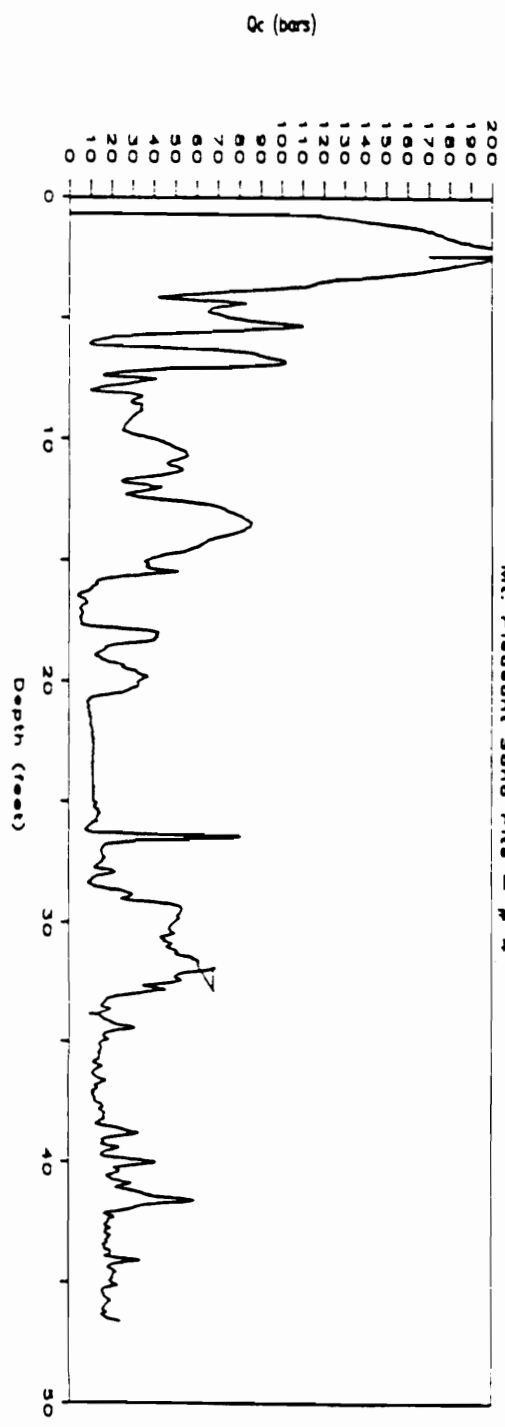
Mount Pleasant Sand Pile - #3



Charleston Liquefaction Study
Mt. Pleasant Sand Pile - # 4

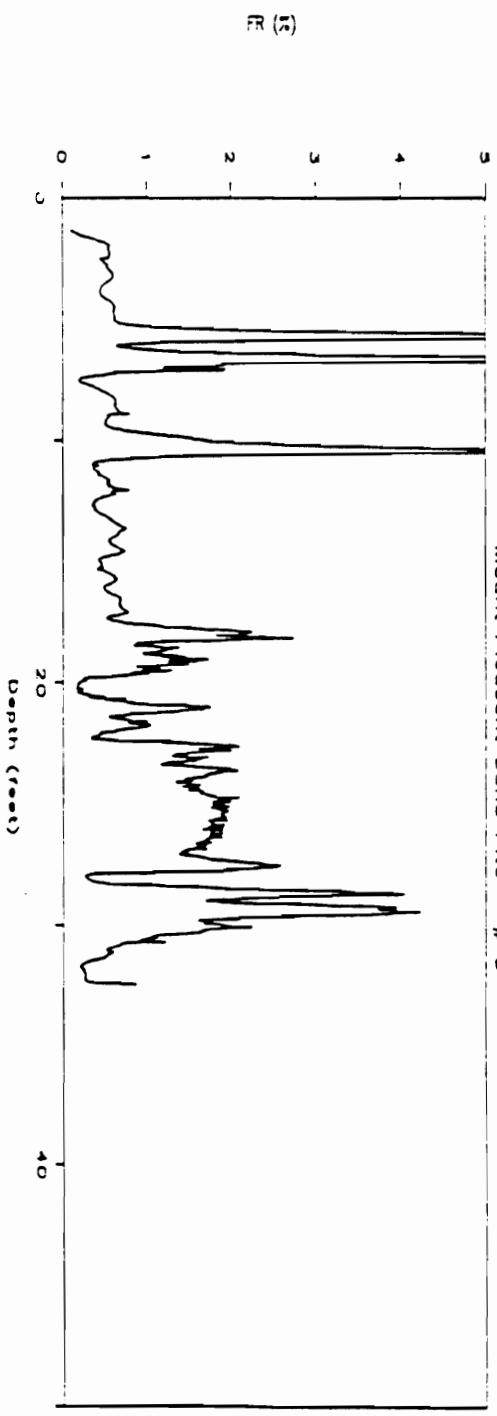


Charleston Liquefaction Study
Mt. Pleasant Sand Pile - # 4



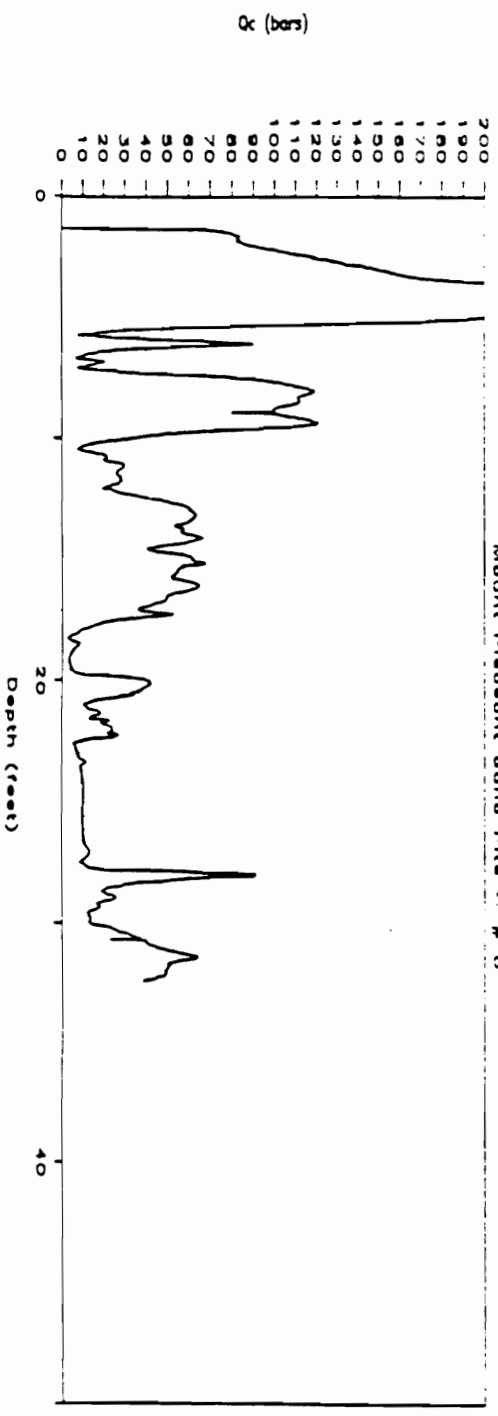
Charleston Earthquake Study

Mount Pleasant Sand Pile - # 3



Charleston Earthquake Study

Mount Pleasant Sand Pile - # 0



LOG of BORING

Project: CHARLESTON LIQUEFACTION PROJECT - MT. PLEASANT PITS

Boring No. MPPIT 2-2 S. C. I. Project No. - Date JULY 1988

Ground Surface Elev. Assumed 0'0" Datum - Gr. Water Elev. 9'0"
(Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRAIPIUS Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	From	To	From	To		
1	1'0"	2'6"			LOOSE GRAYISH-BROWN FINE SAND	4-5-4
2	2'3"	4'0"			LOOSE BROWN FINE SAND	3-3-5
3	5'0"	6'6"			LOOSE GRAY FINE SAND	2-3-3
4	7'6"	9'0"			LOOSE GRAY FINE SAND	4-4-6
5	10'0"	11'6"			V. LOOSE GRAY FINE SAND	1-1-1
6	12'6"	14'0"			V. LOOSE GRAY FINE SAND	2-1-2
7	15'6"	16'6"			LOOSE GRAY FINE SAND	2-4-3
8	17'6"	19'0"			V. LOOSE BLUISH-GRAY FINE SILTY SAND W/ SLIGHT SHELL CONTENT	2-2-2
9	20'0"	21'6"			V. LOOSE GRAY FINE SAND W/ MEDIUM SHELL CONTENT	2-2-2
10	22'6"	24'0"			V. SOFT BLUISH GRAY CLAY W/ TRACE OF SAND AND SHELL	1-1/2"
11	27'6"	29'0"			** STIFF GREENISH-BROWN SILTY SANDY CALCAREOUS MATERIAL (CLAY)	2-7-7

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

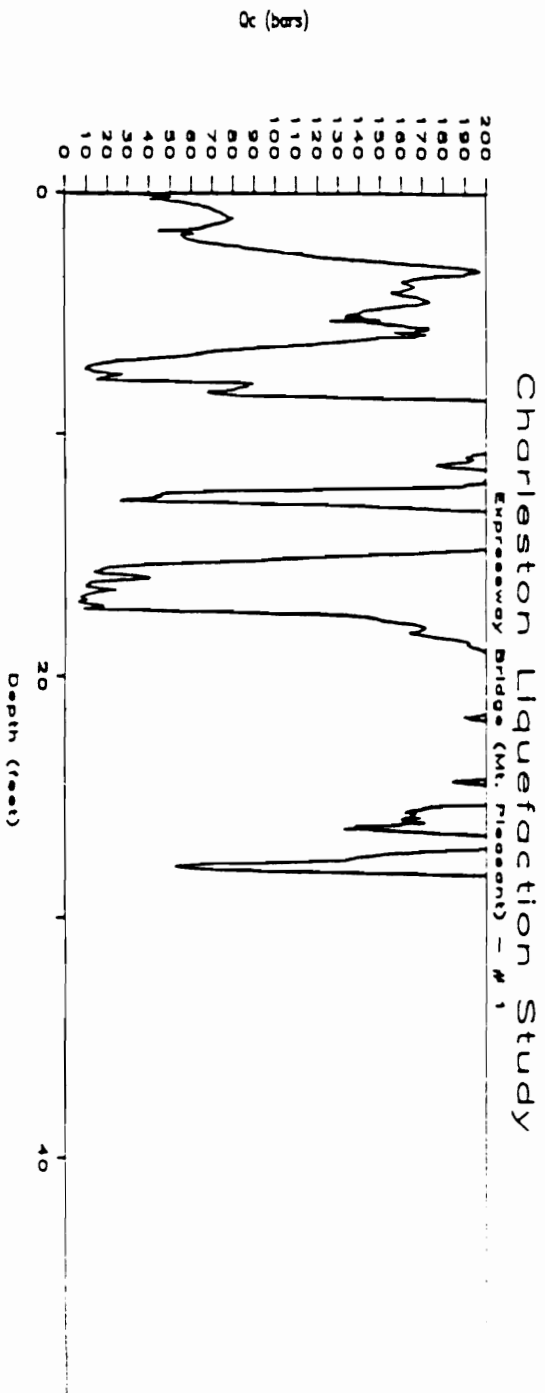
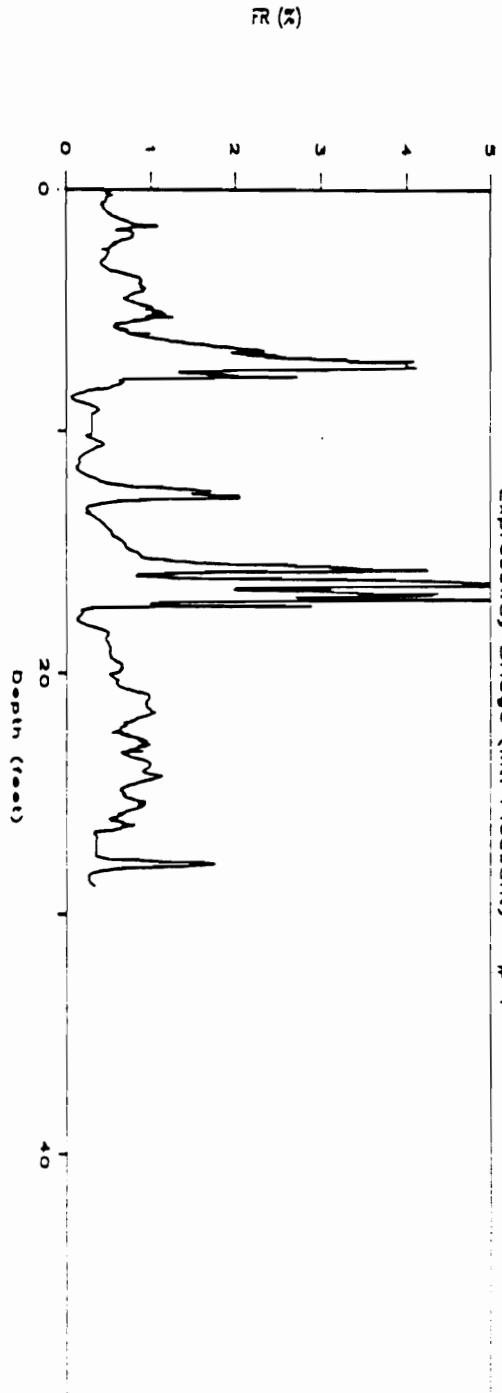
Remarks: ** COOPER MARL

TOO DAMNED HOT IN CHARLESTON!!

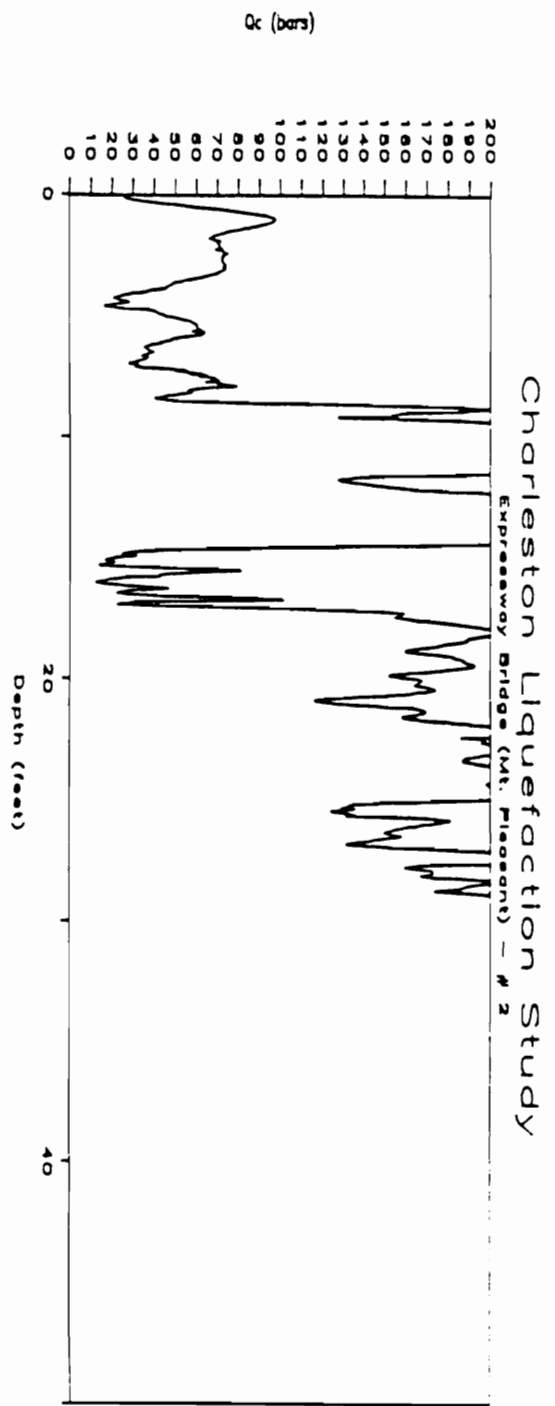
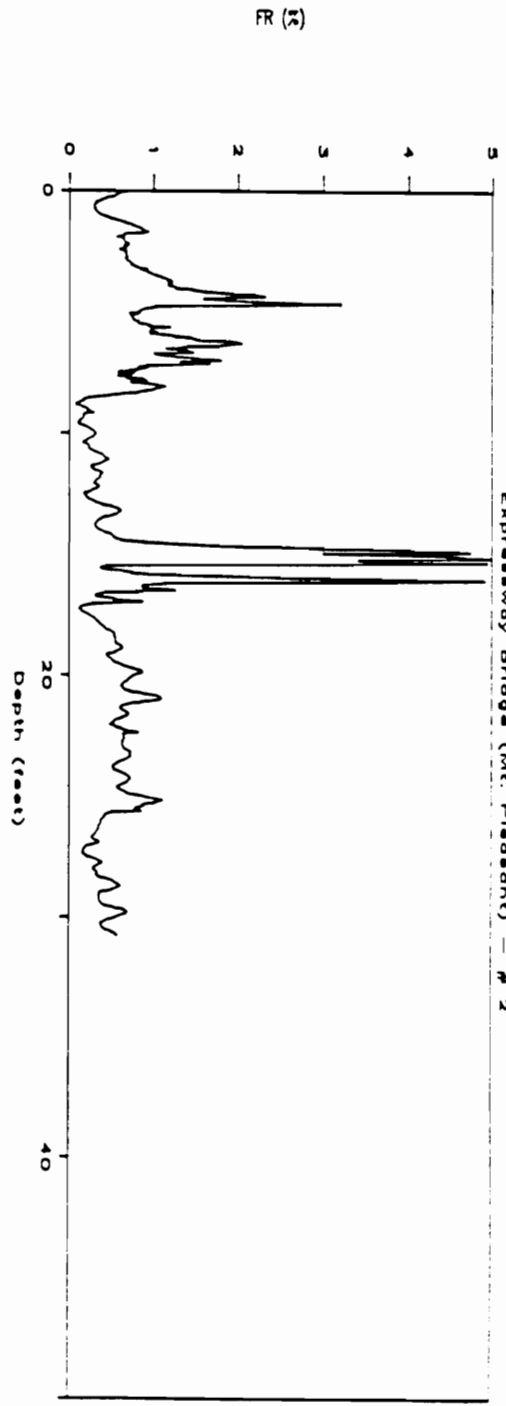
MARK CLARK EXPRESSWAY BRIDGE/
ISLE OF PALMS CONNECTOR SITE

CPT RECORDS

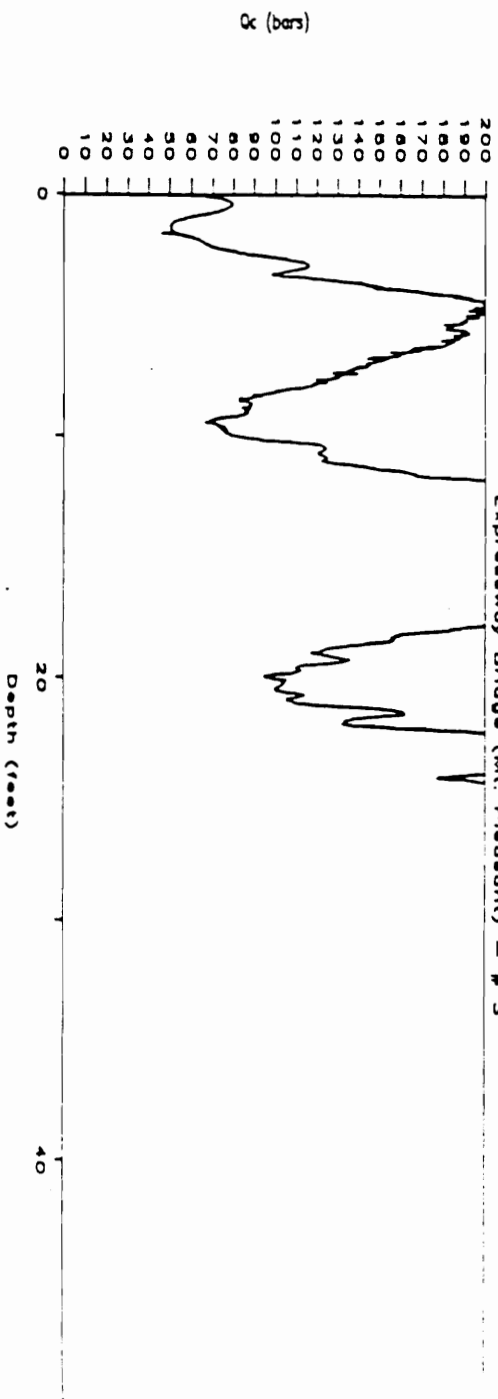
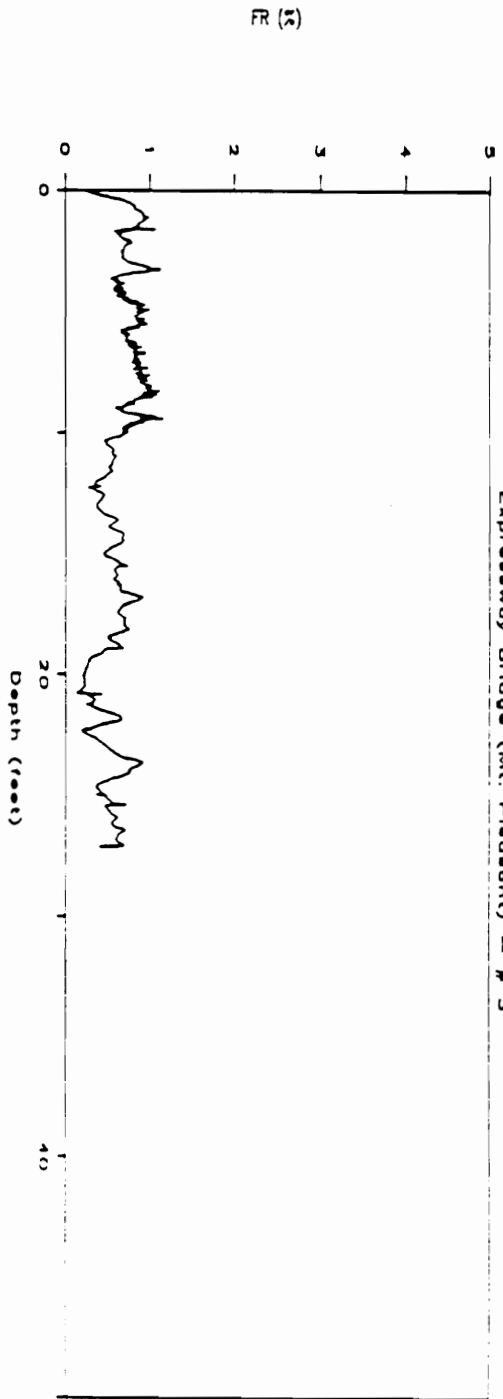
Charleston Liquefaction Study
Expressway Bridge (Mt. Pleasant) - # 1



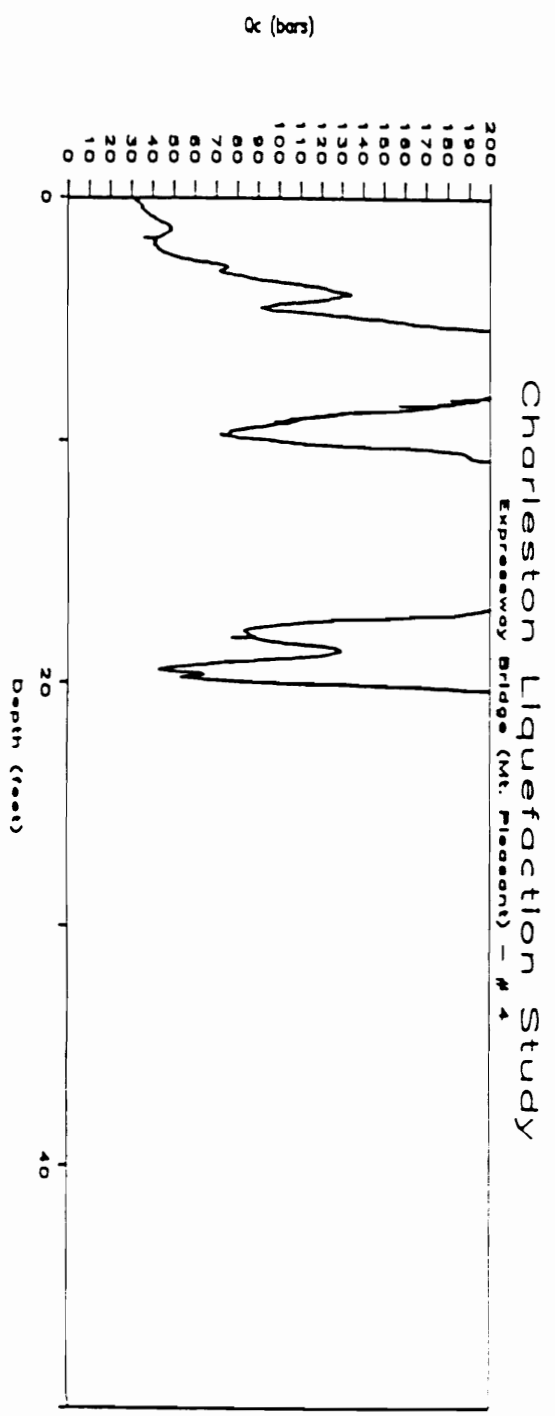
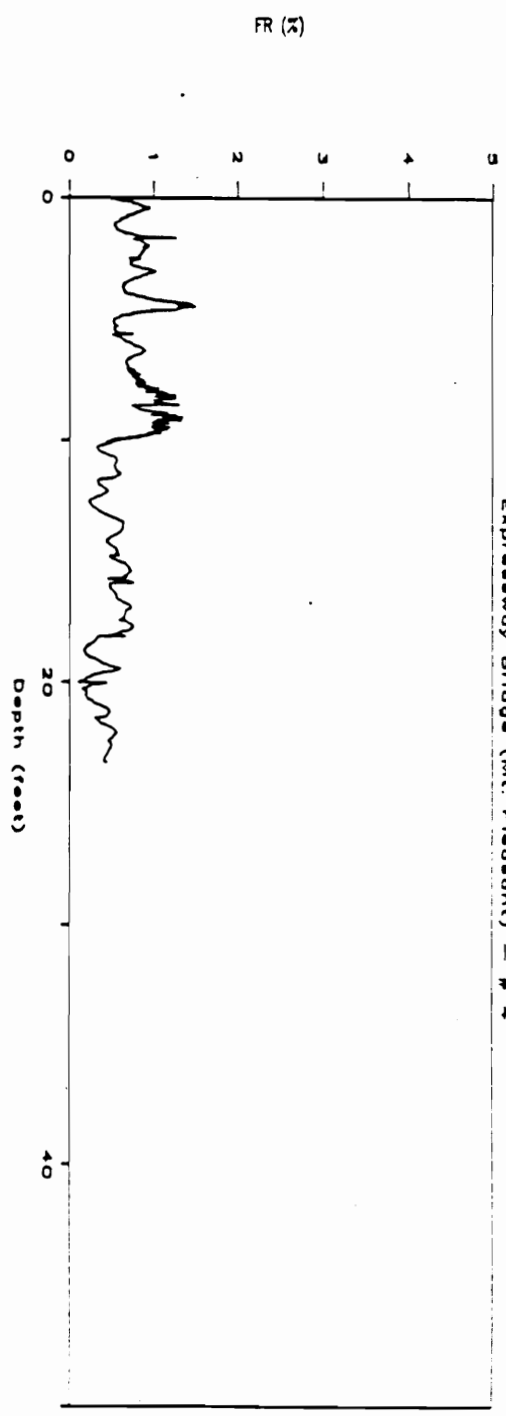
Charleston Liquefaction Study
Expressway Bridge (Mt. Pleasant) - # 2



Charleston Liquefaction Study Expressway Bridge (Mt. Pleasant) - # 3



Charleston Liquefaction Study
 Expressway Bridge (Mt. Pleasant) - # 4

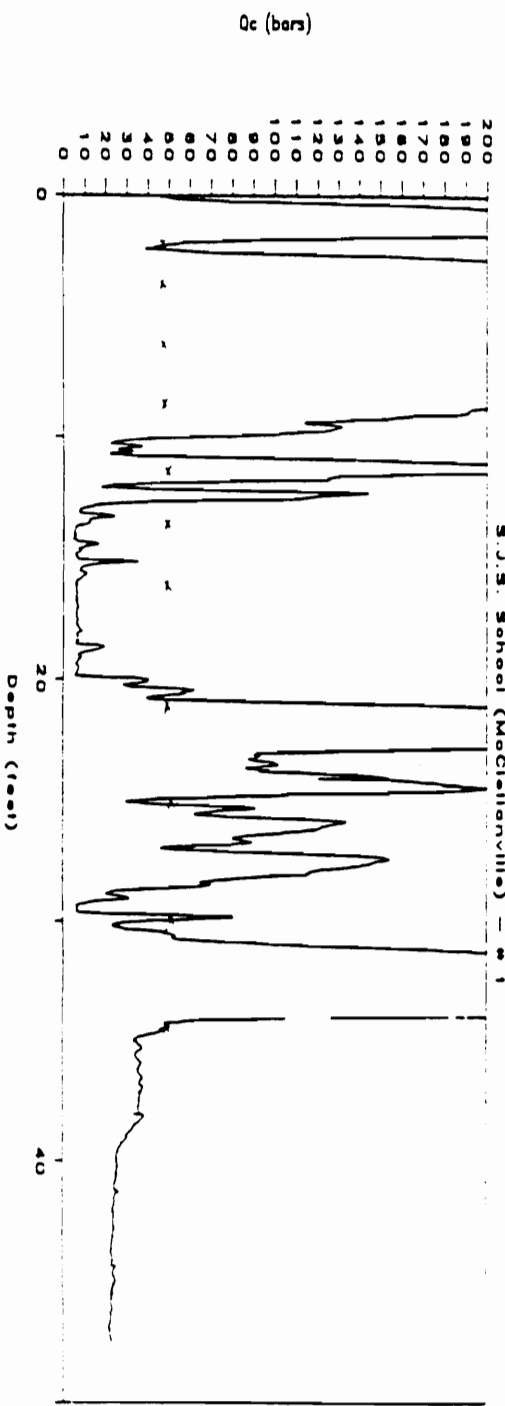
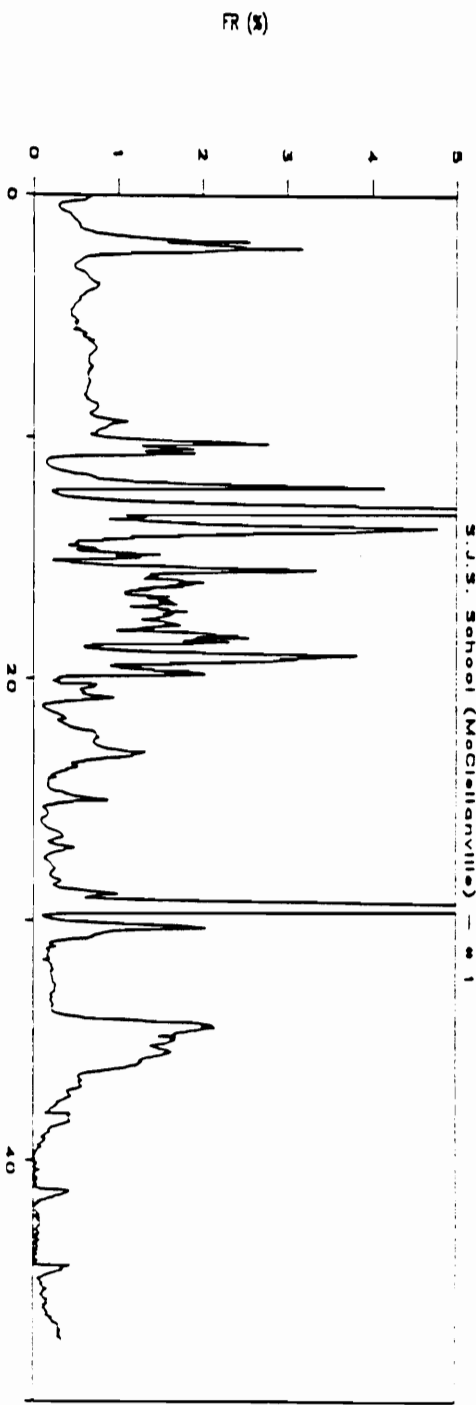


ST. JAMES - SANTEE SCHOOL SITE

(CPT SAMPLES)

Charleston Liquefaction Study

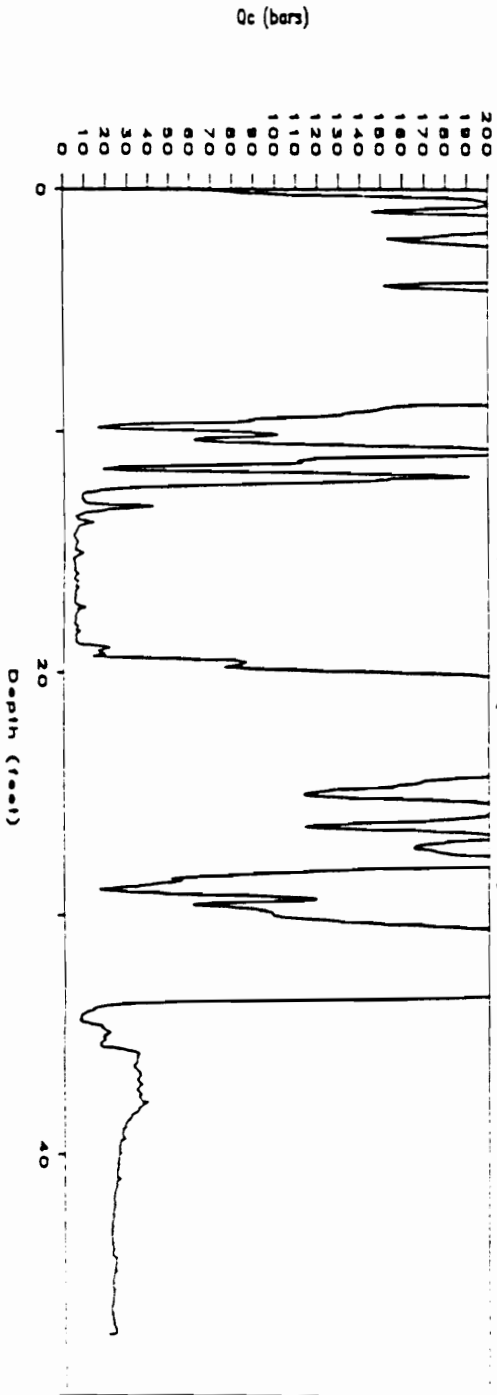
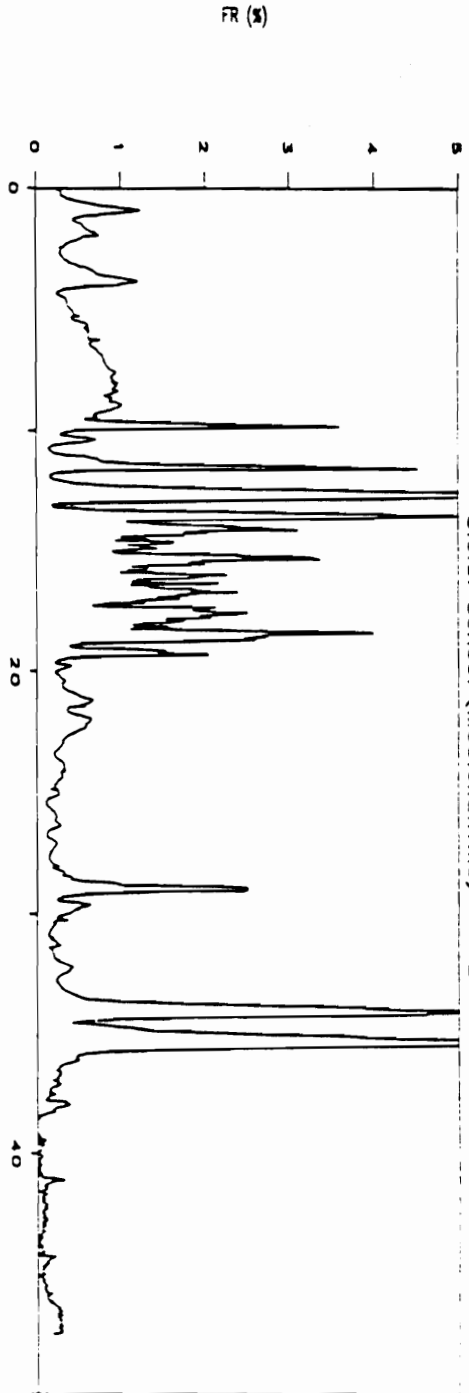
S.J.S. School (McClellanville) - #1



Charleston Liquefaction Study

S.J.S. School (McClellanville) - #1

Charleston Liquefaction Study
S.J.S. School (McClellanville) - #2

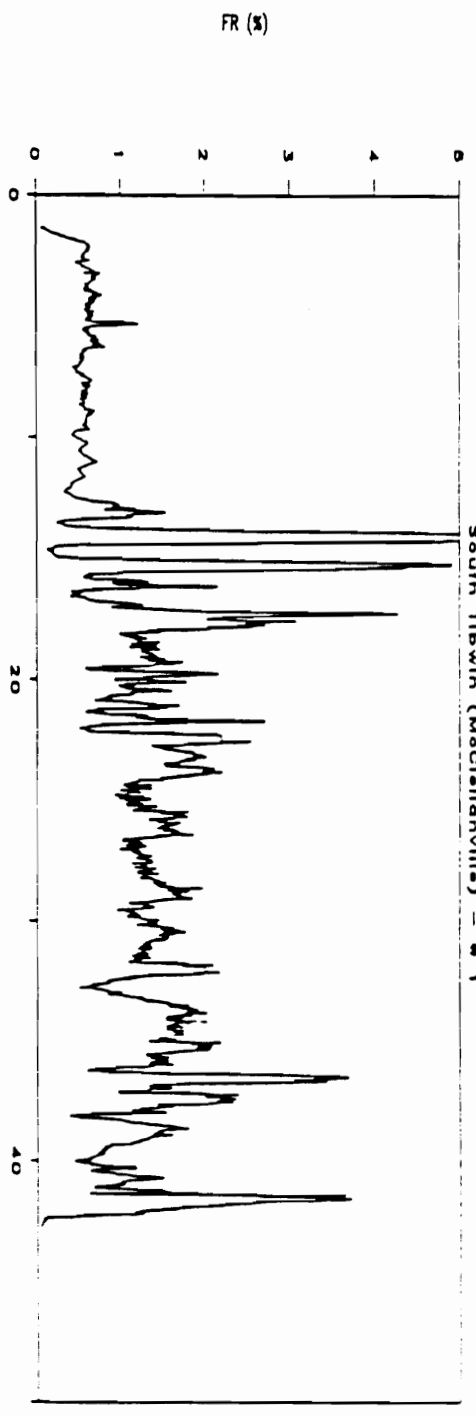


Charleston Liquefaction Study
S.J.S. School (McClellanville) - #2

SOUTH TIBWIN SITE
(CPT AND SPT SAMPLES)

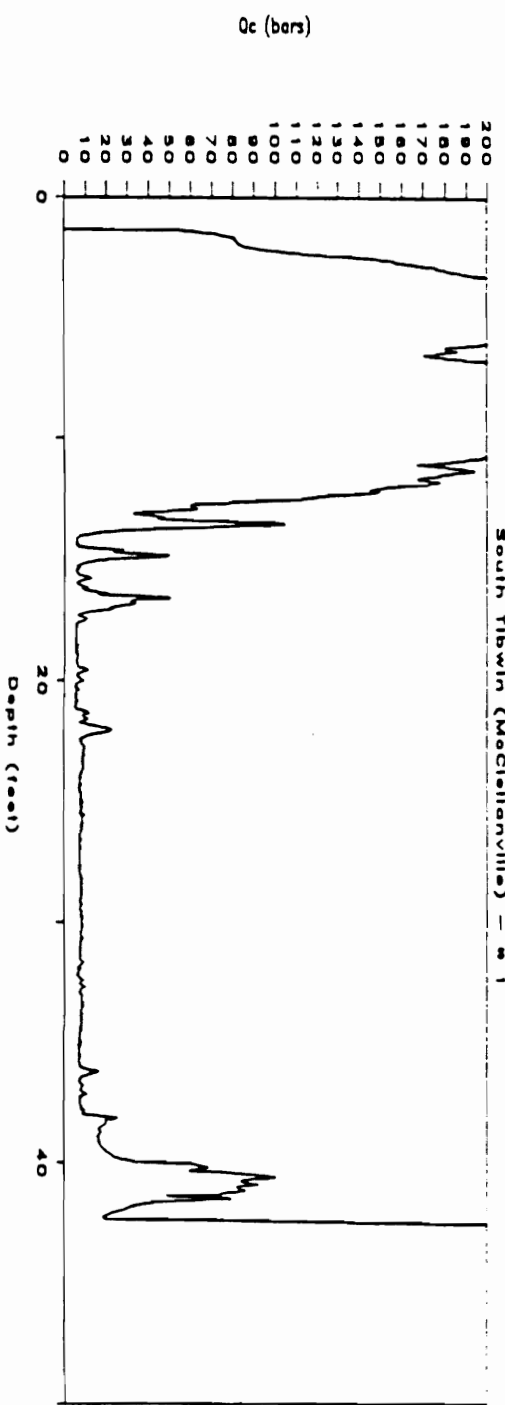
Charleston Liquefaction Study

South Tibwin (McClellanville) - #1



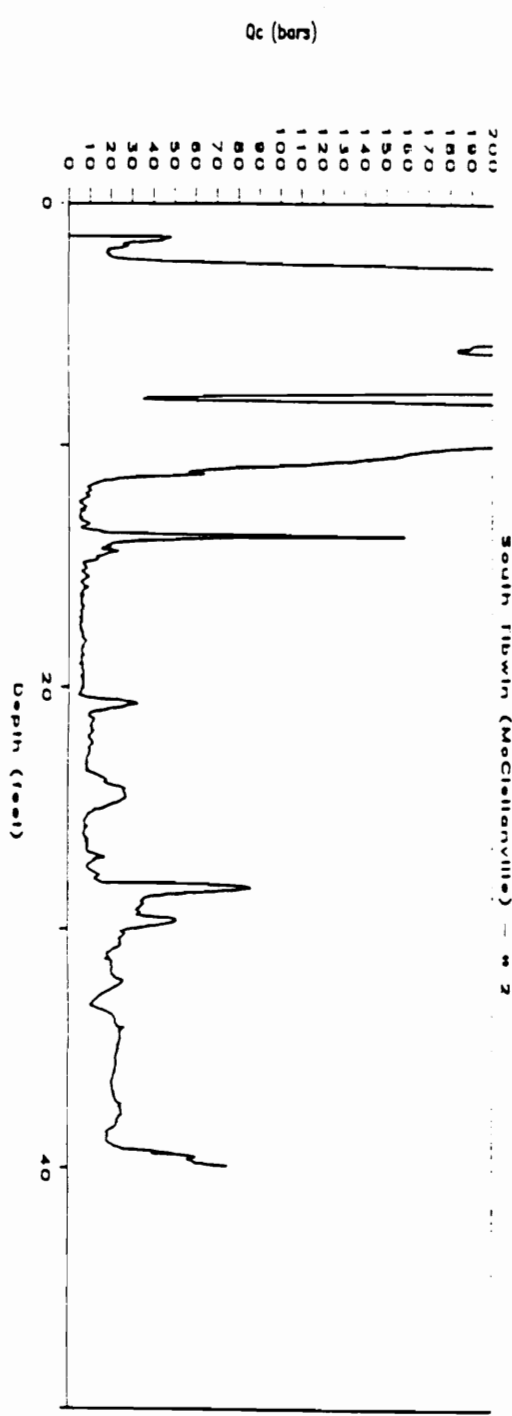
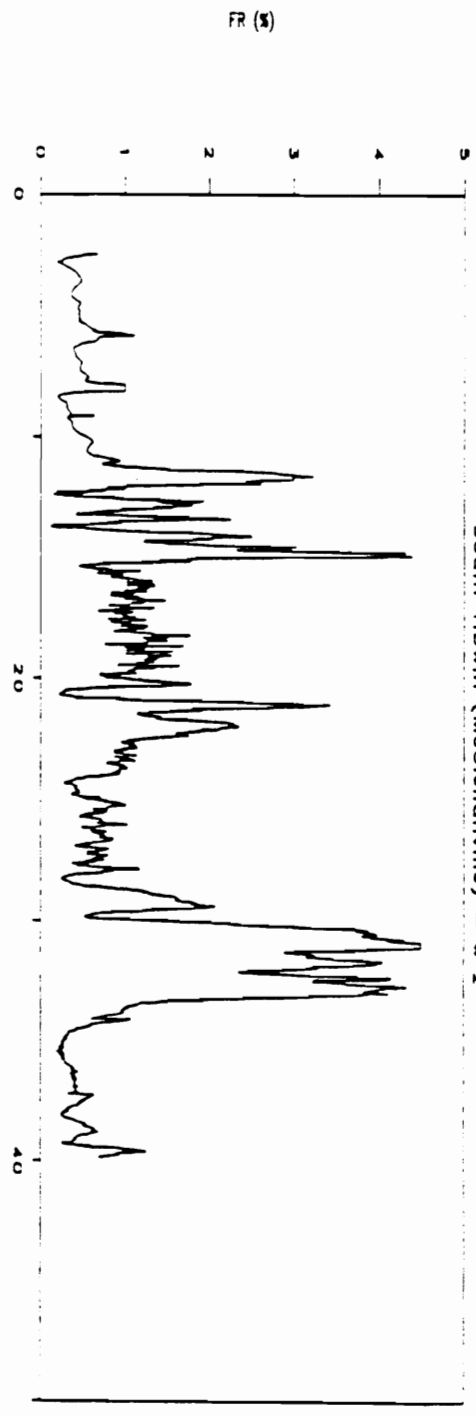
Charleston Liquefaction Study

South Tibwin (McClellanville) - #1



Charleston Liquefaction Study

South Tibwin (McClellanville) - # 2

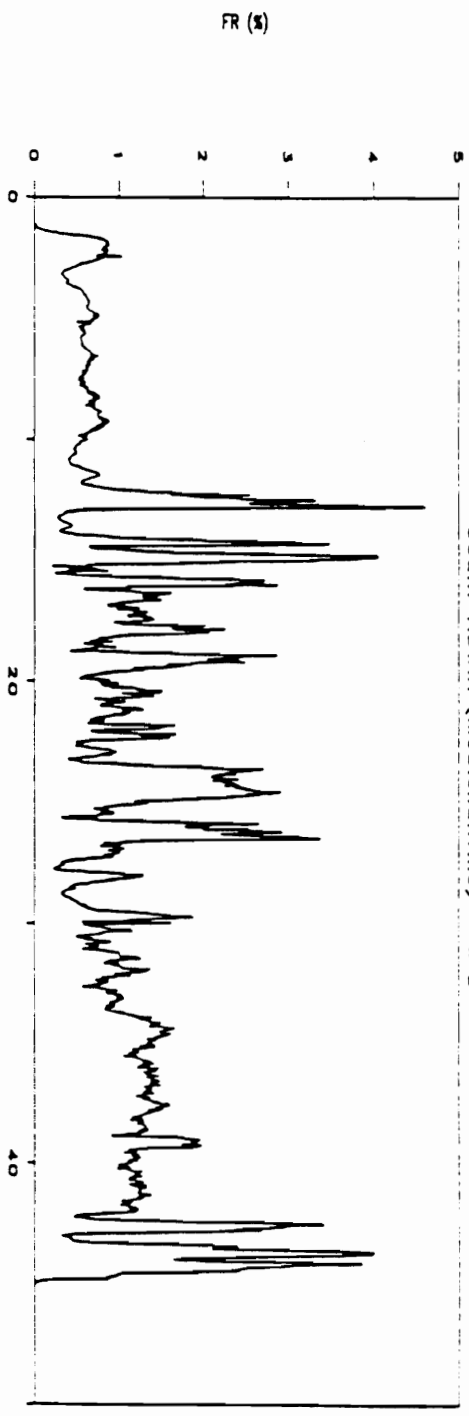


Charleston Liquefaction Study

South Tibwin (McClellanville) - # 2

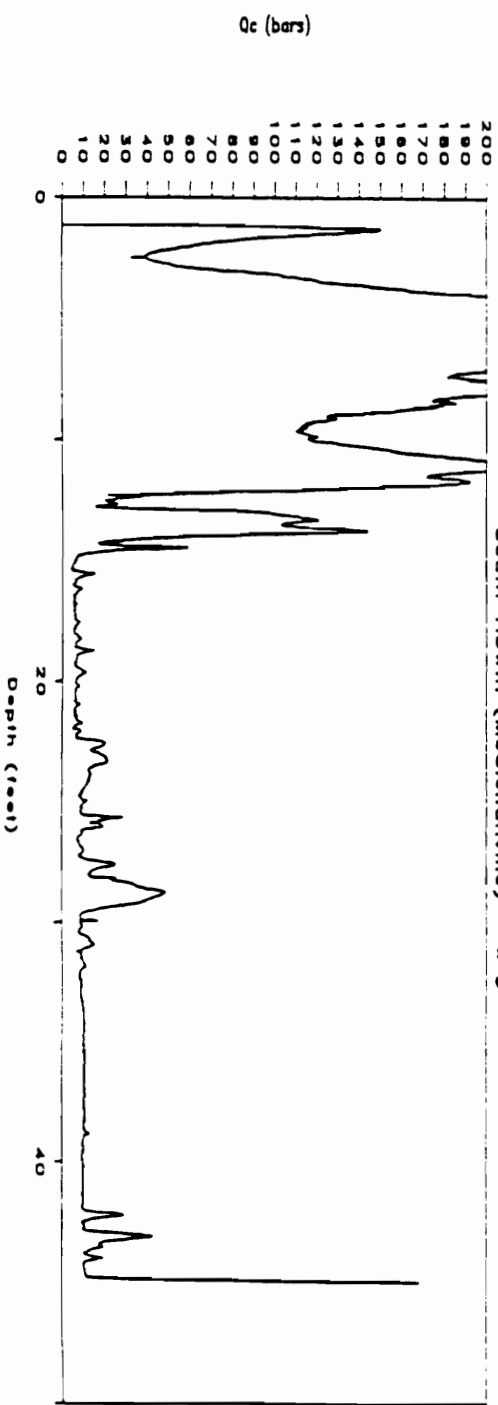
Charleston Liquefaction Study

South Tibwin (McClellanville) - # 3



Charleston Liquefaction Study

South Tibwin (McClellanville) - # 3



LOG of BORING

Project: CHARLESTON LIQUEFACTION PROJECT - SOUTH TIBWIN
 Boring No. South Tib 1 S. C. I. Project No. - Date JULY 1988
 Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. 4'6"
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 Inches
	From	To	From	To		
1	1'0"	2'6"	0'0"		LOOSE BROWN FINE SAND w/	2-3-2
				2'6"	ROOT	
2	2'6"	4'0"	2'6"	4'6"	V. LOOSE BROWN FINE SAND	2/18"
3	5'0"	6'6"	4'6"	7'0"	MEDIUM GRAY FINE SAND	5-12-14
4	7'6"	9'0"	7'0"		MEDIUM GRAY MEDIUM SAND w/	8-12-14
				9'6"	ROOT	
5	10'0"	11'6"	9'6"	12'0"	MEDIUM GRAY FINE SAND	6-7-6
6	12'6"	14'0"	12'0"		LOOSE GRAY FINE SAND w/	1-2-5
				14'6"	HEAVY SHELL CONTENT	
7	15'0"	16'6"	14'6"		LOOSE GRAY V. SILTY SAND w/	4-7-2
				17'0"	SLIGHT SHELL CONTENT	
8	17'6"	19'0"	17'0"		V. SOFT GRAY ORGANIC CLAY	1/18"
				19'6"	w/ SAND AND HEAVY SHELL HASH	
9	20'0"	21'6"	19'6"		V. LOOSE GRAY CLAYEY SAND	W.O.H. - 3/12"
				22'0"	w/ HEAVY SHELL HASH CONTENT	
10	22'6"	24'0"	22'0"		V. SOFT BLYSH-GRAY ORGANIC	W.O.H. - 2/6"
				24'6"	CLAY	
11	25'0"	26'6"	24'6"		V. SOFT BLYSH-GRAY ORGANIC	W.O.H. - 1/6"
					CLAY w/ SAND SEEMS	
12	30'0"	31'6"			V. SOFT BLYSH GRAY ORGANIC	W.O.H. - 1/6"
				34'0"	CLAY w/ SAND SEEMS	
13	35'0"	36'6"	34'0"	36'0"	LOOSE GRAY COARSE SAND AND	2-2-4
					* * GREENISH-BROWN SILTY SANDY	
					CALCAREOUS CLAY	
14	40'0"	41'6"			HARD COOPER MARL	11-17-19

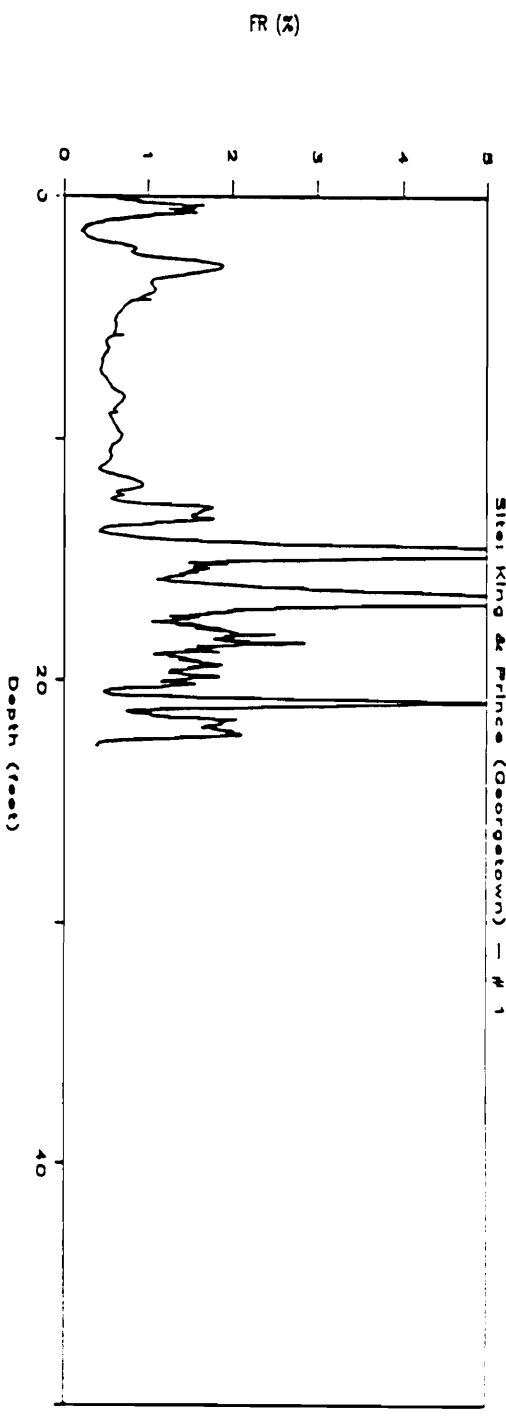
Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

Remarks: ** COOPER MARL

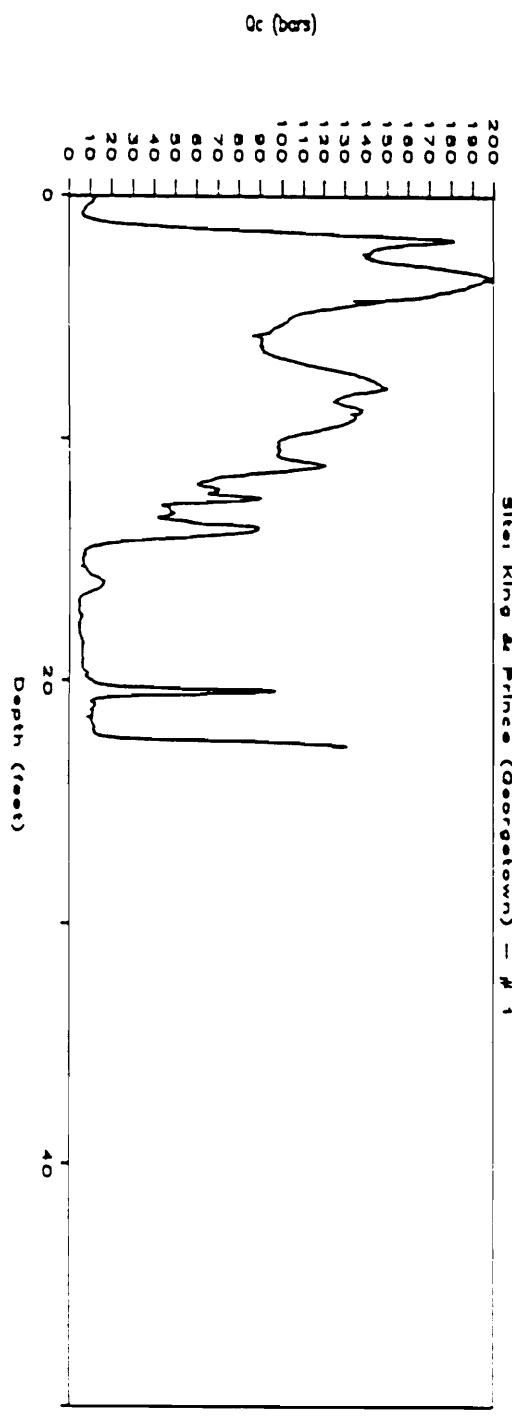
GEORGETOWN SITE (KING & PRINCE STREETS)

CPT AND SPT RECORDS

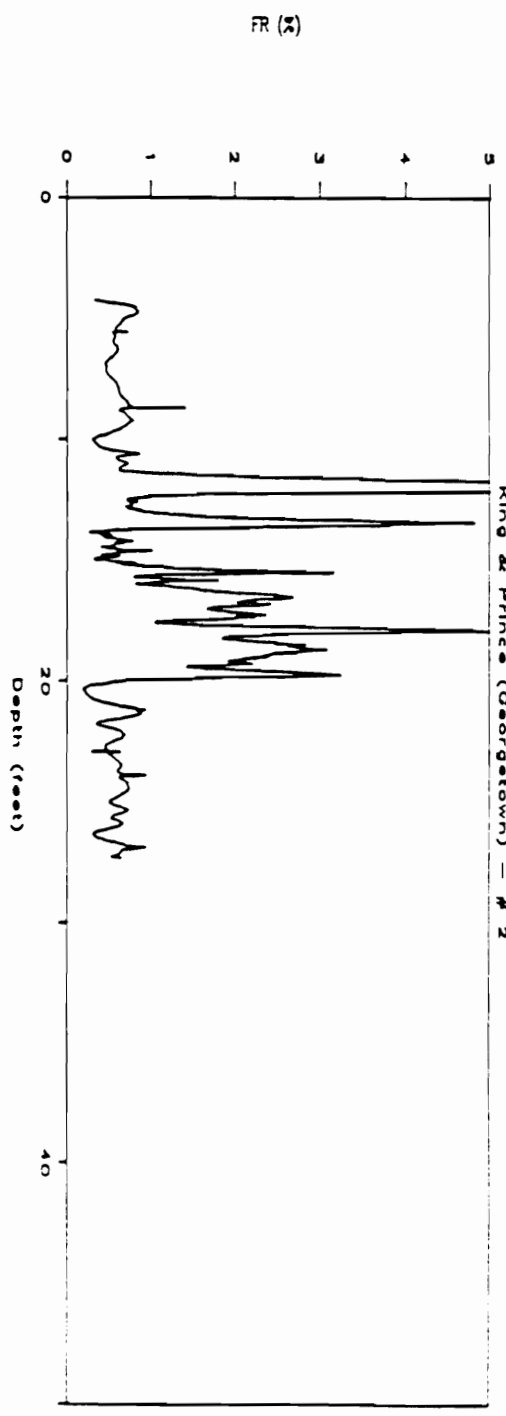
Charleston Earthquake Study
 Site: King & Prince (Georgetown) - # 1



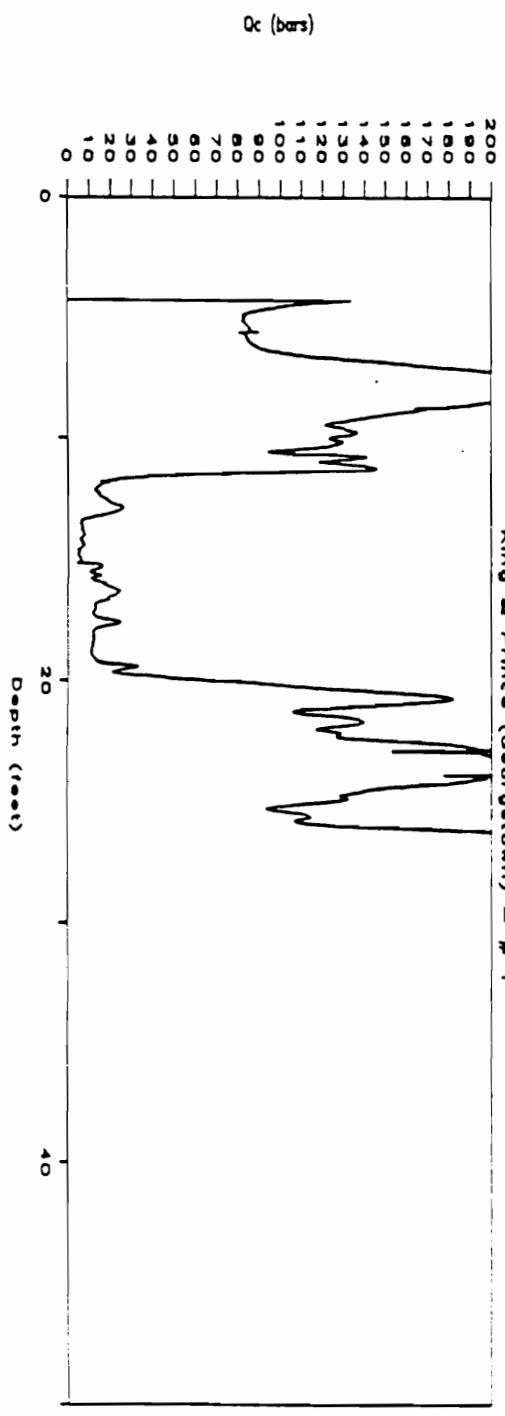
Charleston Earthquake Study
 Site: King & Prince (Georgetown) - # 1



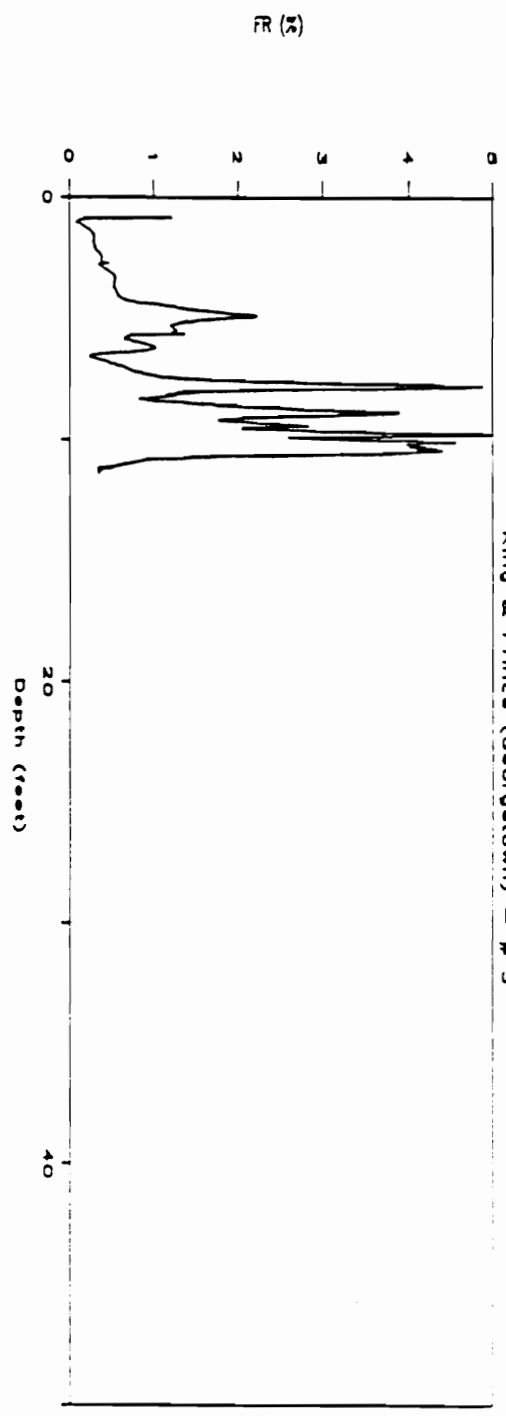
Charleston Liquefaction Study
 King & Prince (Georgetown) - # 2



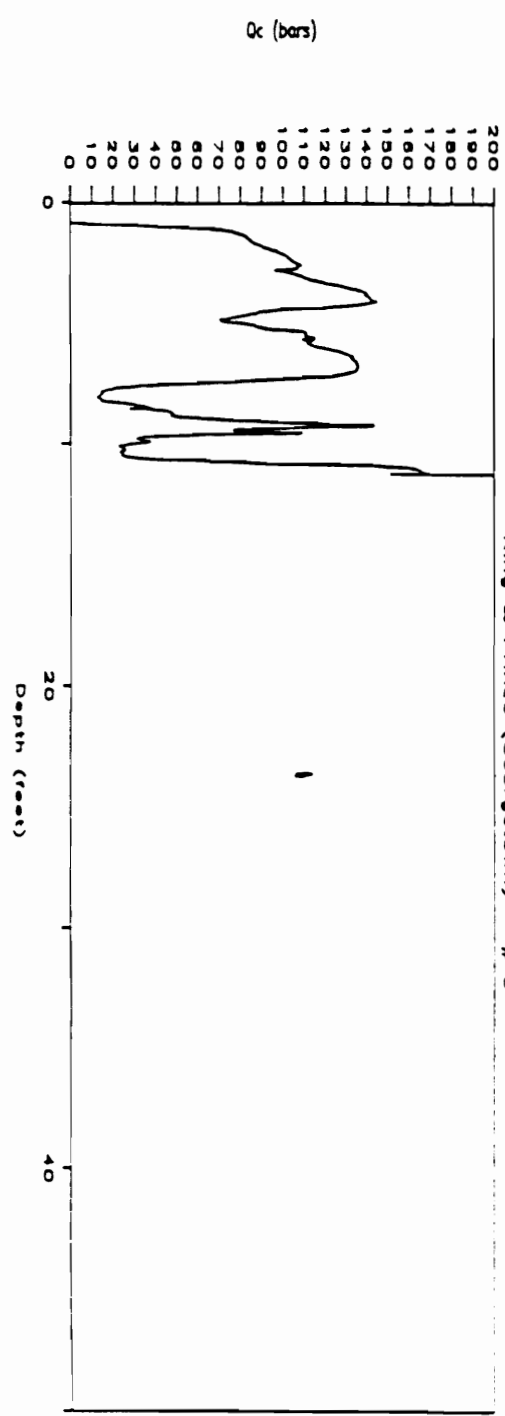
Charleston Liquefaction Study
 King & Prince (Georgetown) - # 1



Charleston Liquefaction Study
King & Prince (Georgetown) - # 3



Charleston Liquefaction Study
King & Prince (Georgetown) - # 3

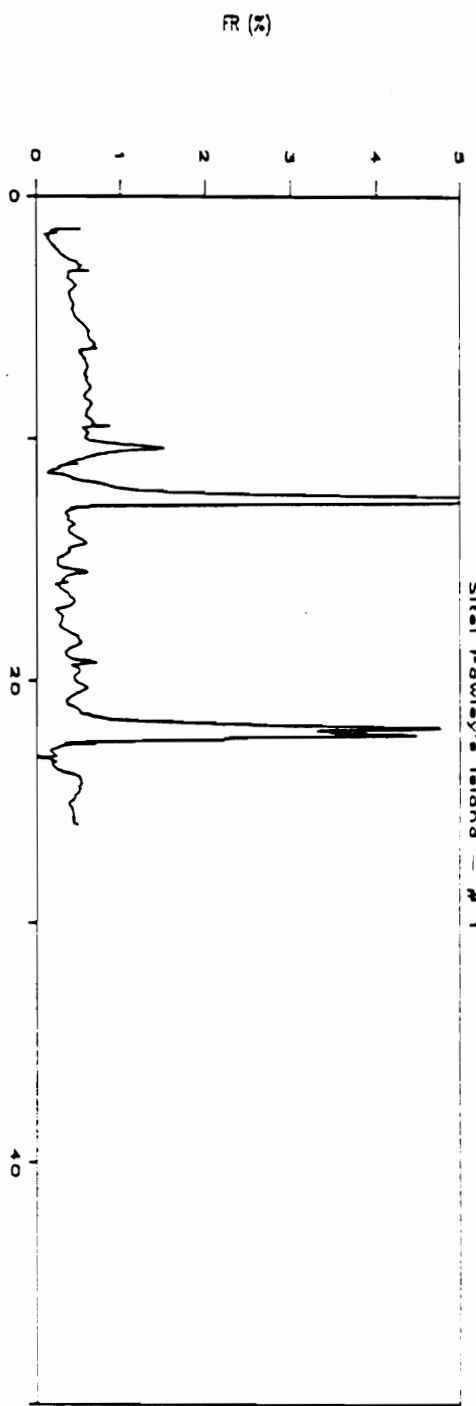


PAWLEY'S ISLAND/PARKERSVILLE SITE

CPT RECORDS

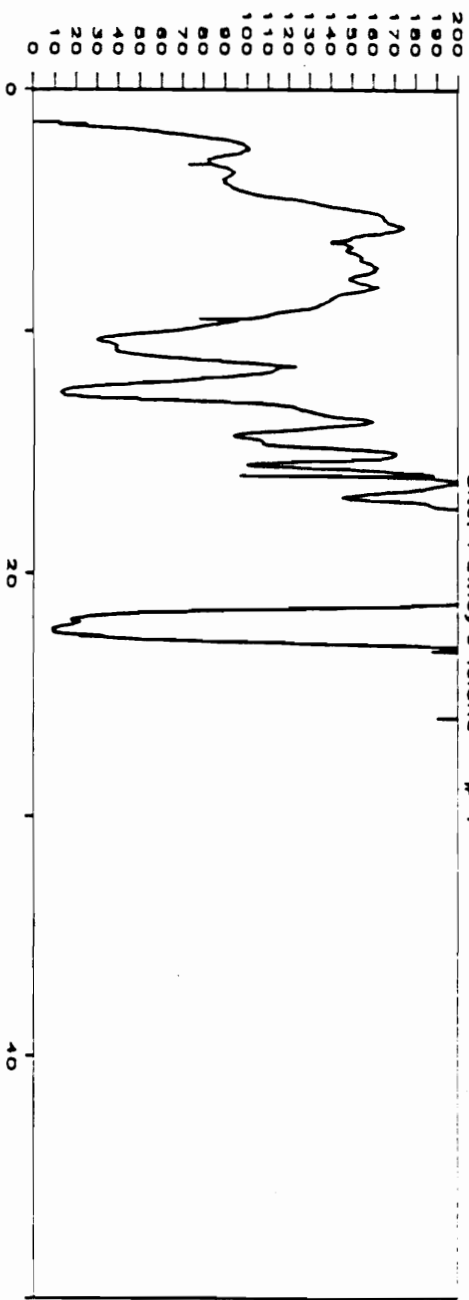
Charleston Liquefaction Study

Site: Pawley's Island - # 1



Charleston Liquefaction Study

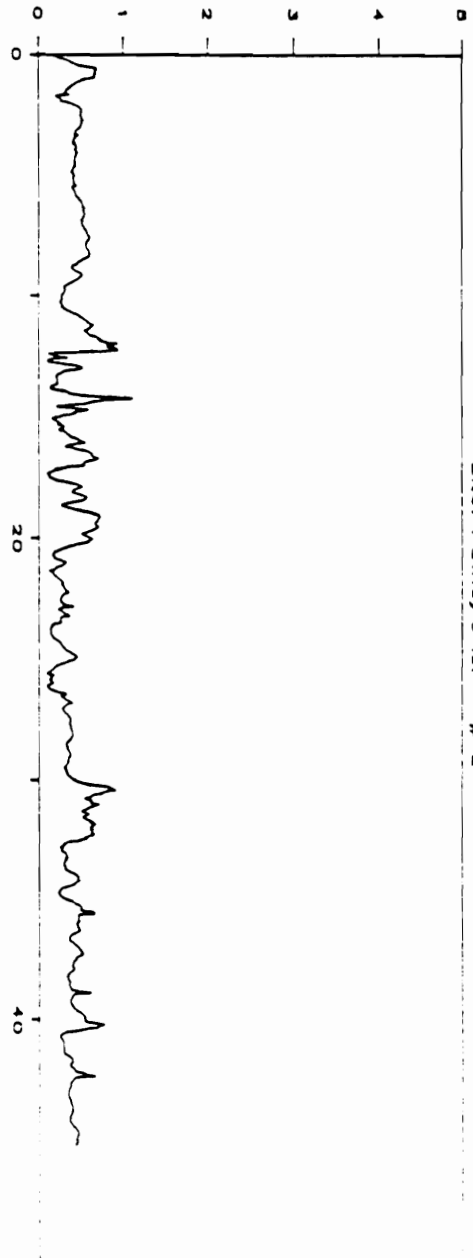
Site: Pawley's Island - # 1



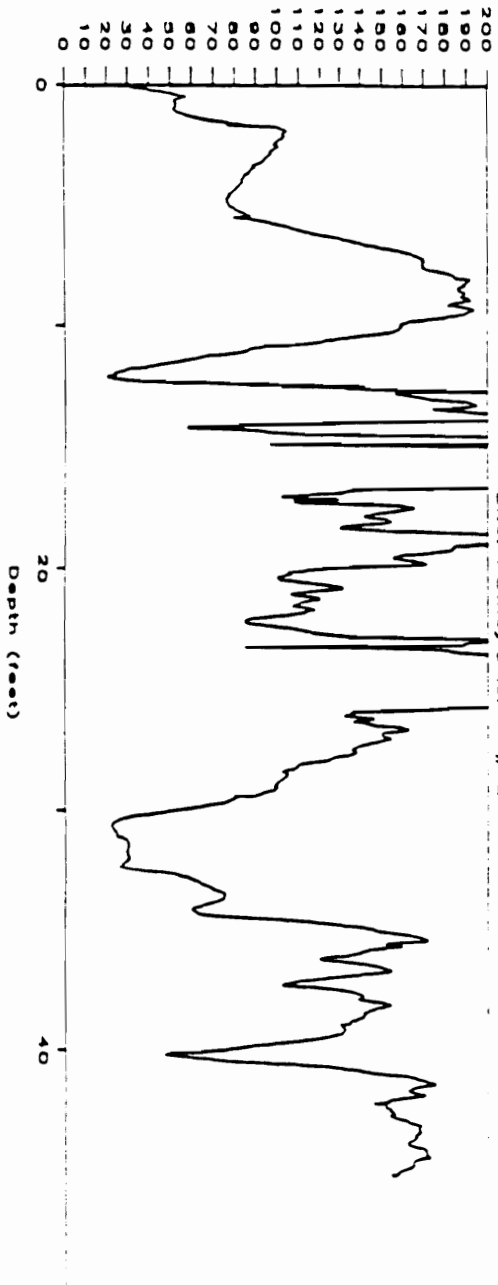
Charleston Liquefaction Study

Site: Pawley's Is. - # 2

FR (%)



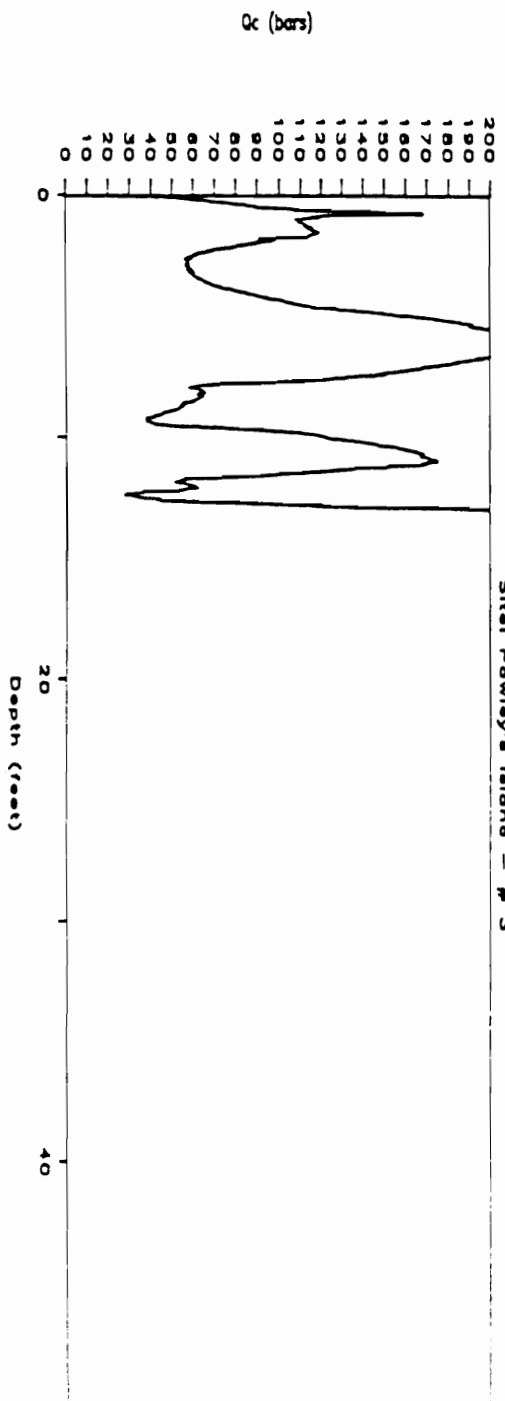
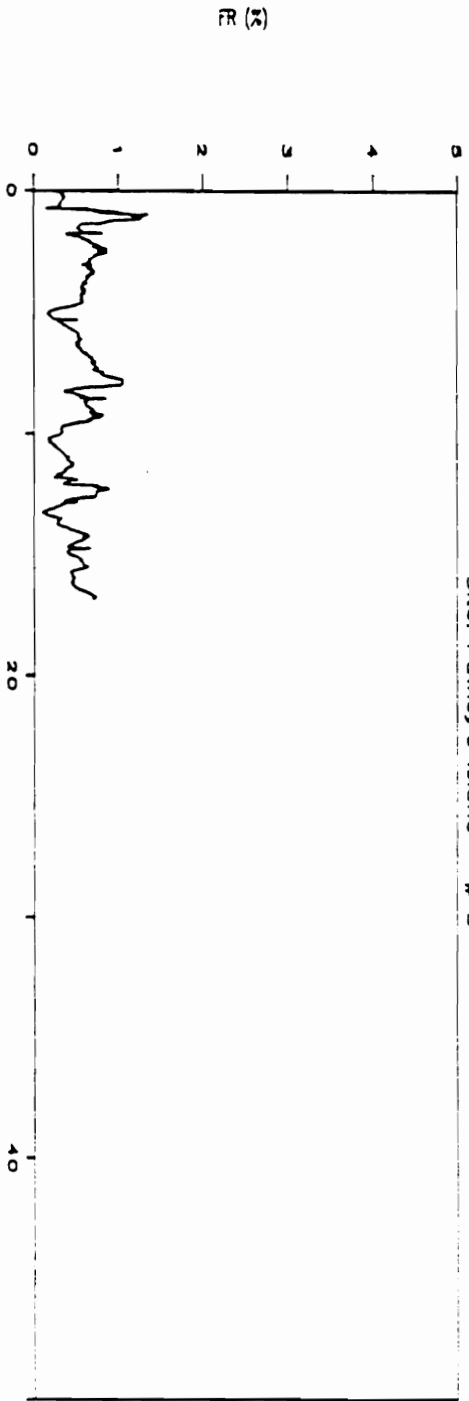
Qc (bars)



Charleston Liquefaction Study

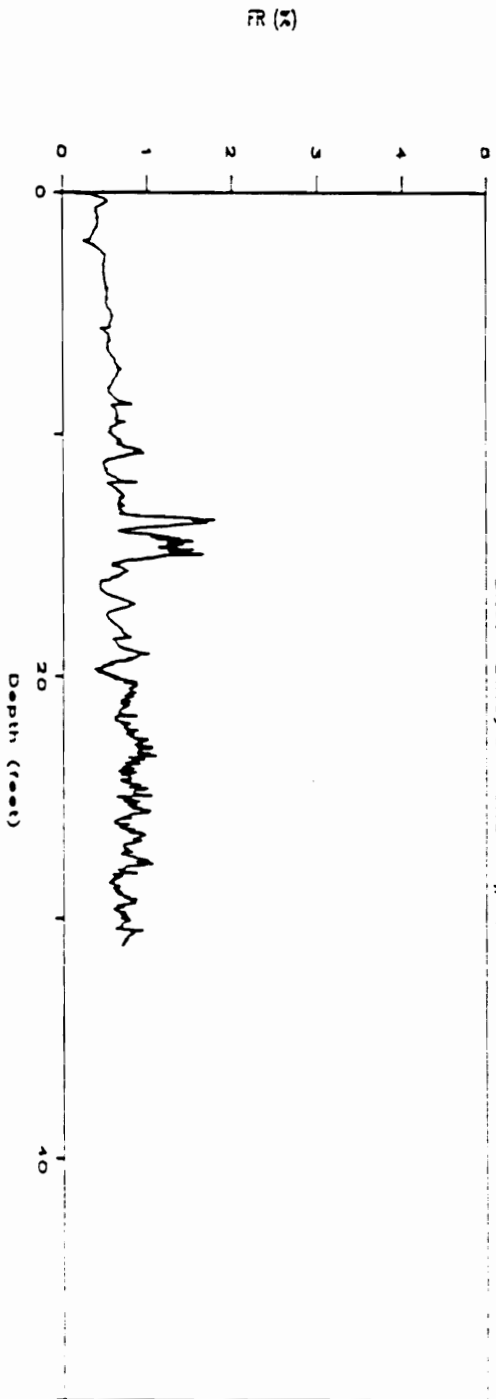
Site: Pawley's Is. - # 2

Charleston Liquefaction Study
Site: Pawley's Island - # 3



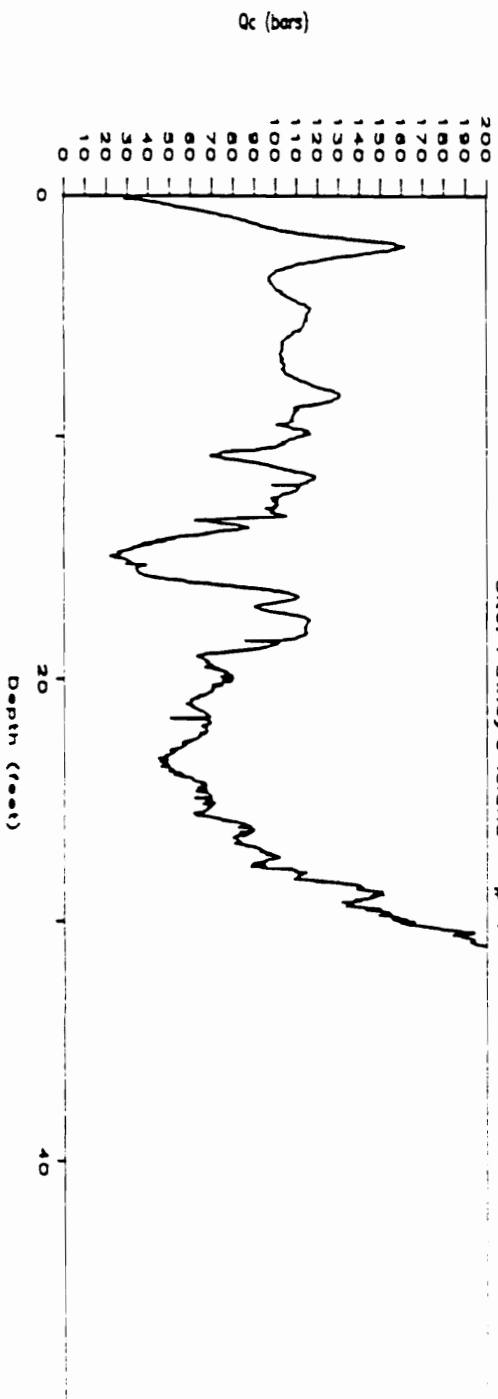
Charleston Liquefaction Study

Site: Pawley's Island - # 4



Charleston Liquefaction Study

Site: Pawley's Island - # 4



LOG of BORING

Project: CHARLESTON LIQUEFACTION PROJECT GEORGETOWN (KING & PRUCE ST.)

Boring No. King/Pruce 1 S. C. I. Project No. - Date JULY 1988

Ground Surface Elev. Assumed 0'0" Datum - Gr. Water Elev. 5'0"
 (Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	From	To	From	To		
1	1'0"	2'6"			MEDIUM BROWN-TAN FINE SAND	3-9-12
2	2'6"	4'0"			MEDIUM BROWN FINE SAND	8-11-14
3	5'0"	6'6"			MEDIUM BROWN FINE SAND W/ TRACE CLAY	3-5-6
4	7'6"	9'0"			MEDIUM BROWN FINE SAND	5-6-9
5	10'0"	11'6"			MEDIUM ORANGE-TAN FINE SAND	3-6-7
6	12'6"	14'0"			MEDIUM ORANGE-TAN FINE SAND	4-6-5
7	15'0"	16'6"			SOFT DARK BROWN FEAT	2-1-2
8	17'6"	19'0"			SOFT GRAY CLAY	Mo.H-6/6"
9	20'0"	21'6"			V. SOFT GRAY CLAY	Mo.H-2/6"
10	22'6"	24'0"			STIFF GRAY CLAY	6-7-8
					↓	
					DRILL TO HARD CLAY AT 36'6"	

Testing and Sampling in accordance with ASTM D. 1586-67 (1974)

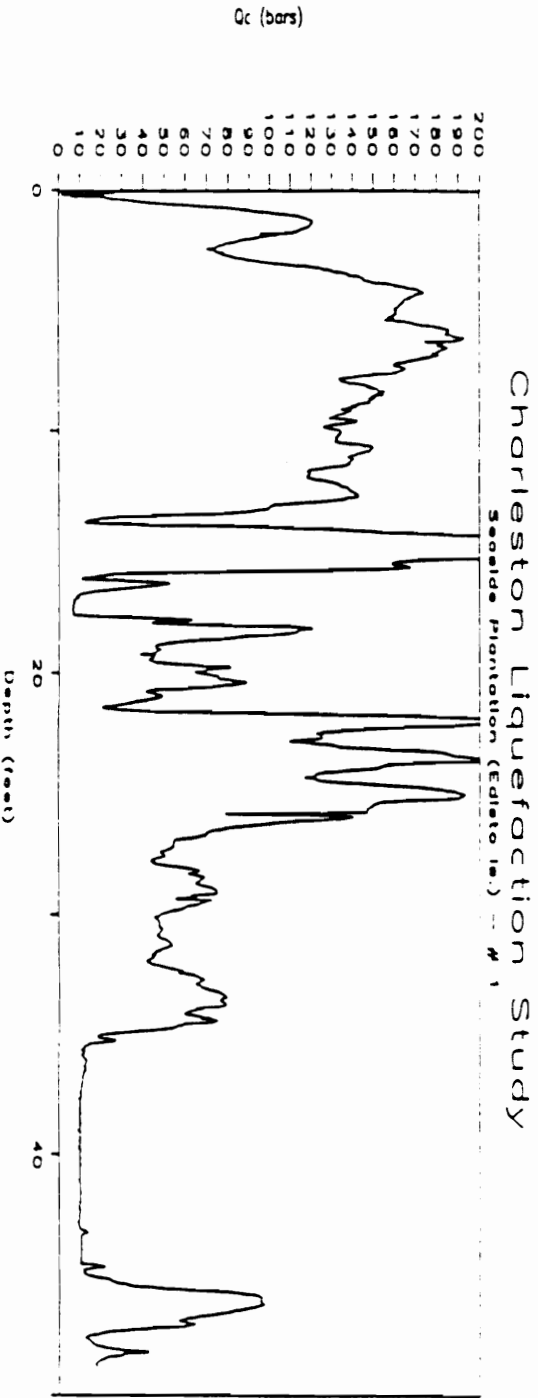
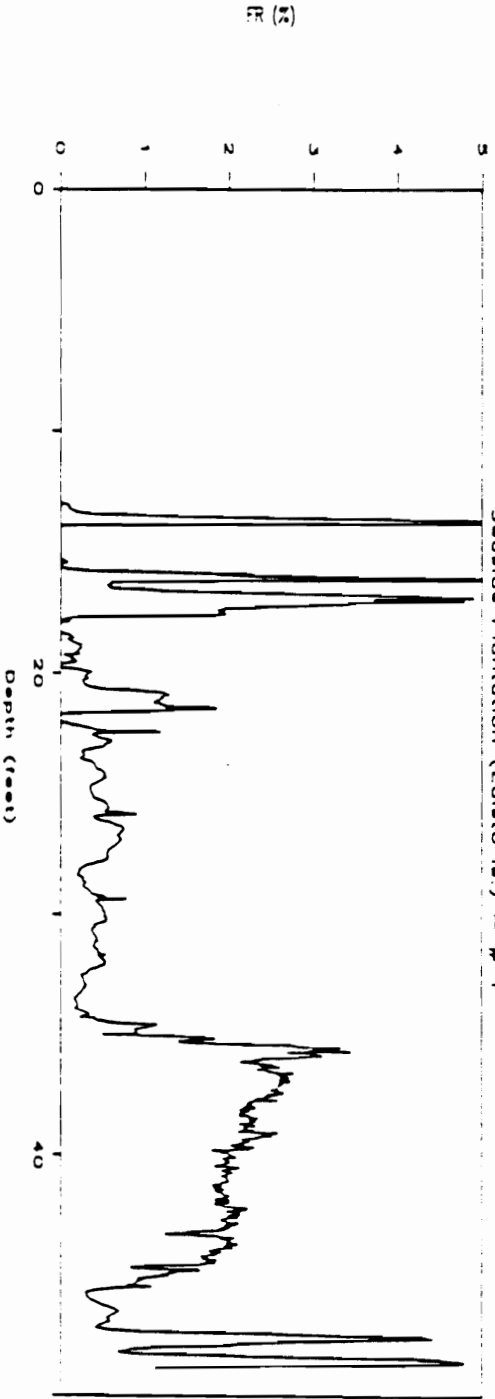
Remarks: _____

EDISTO ISLAND SITE

CPT AND SPT RECORDS

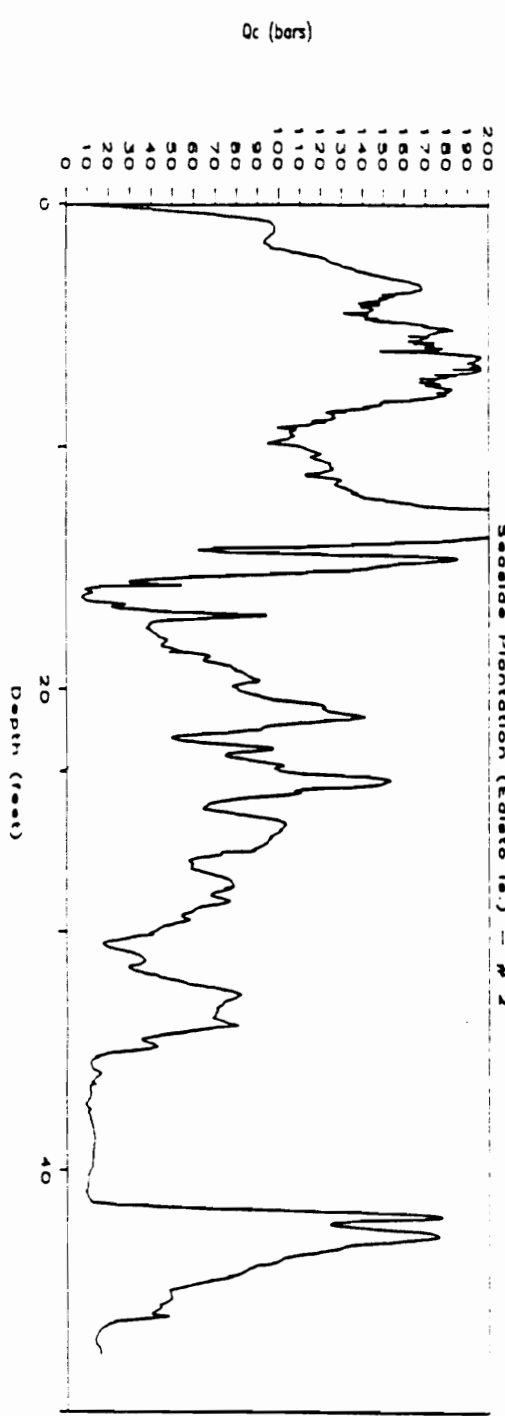
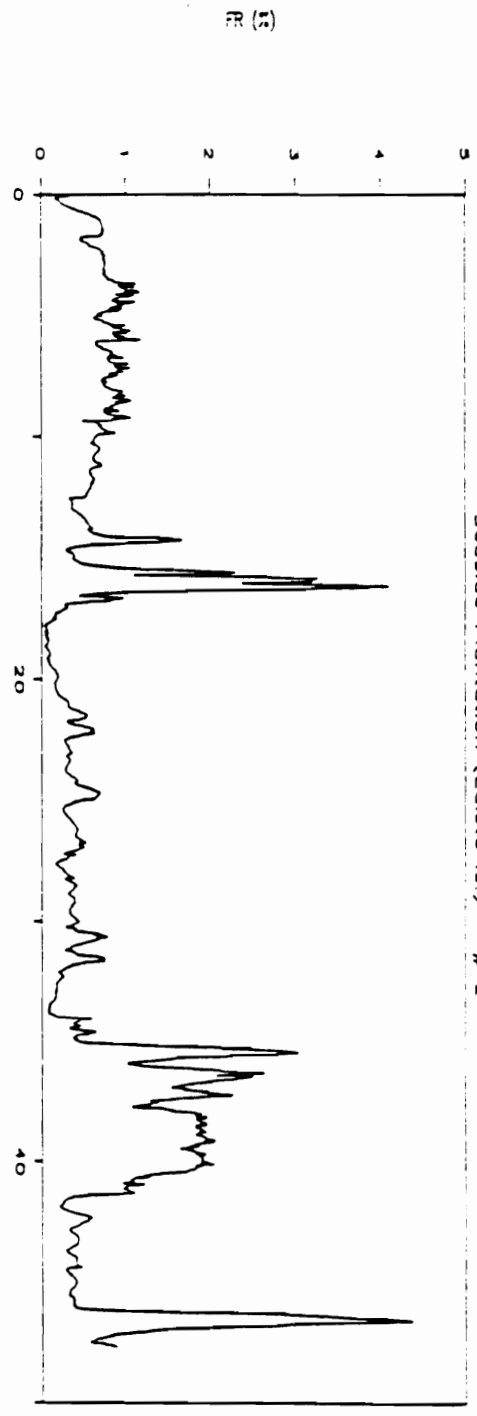
Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 1



Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 2

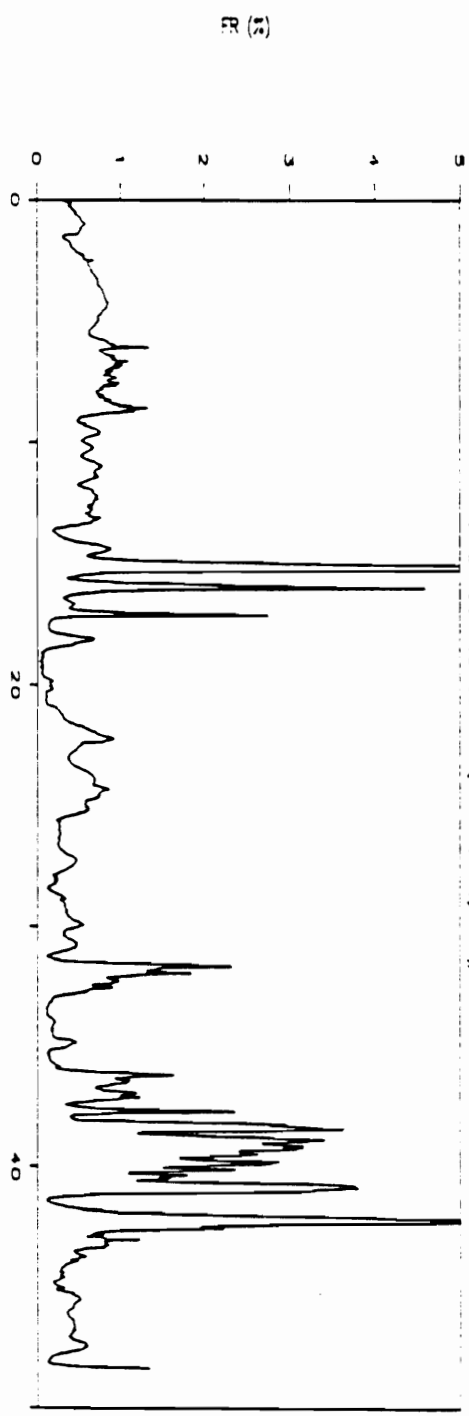


Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 2

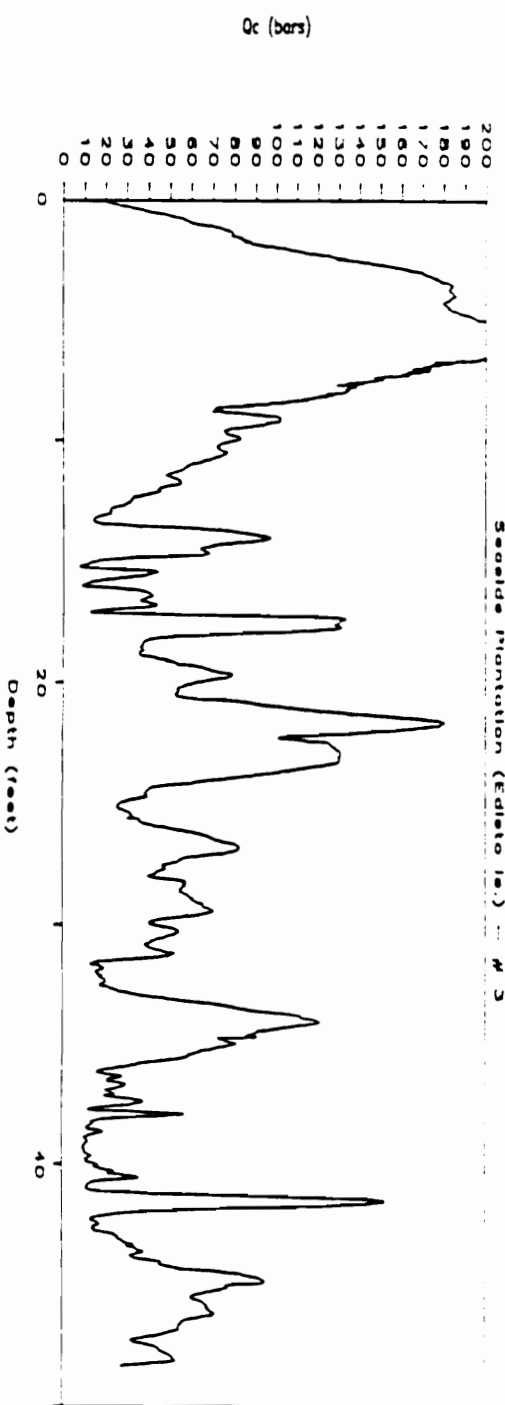
Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 3

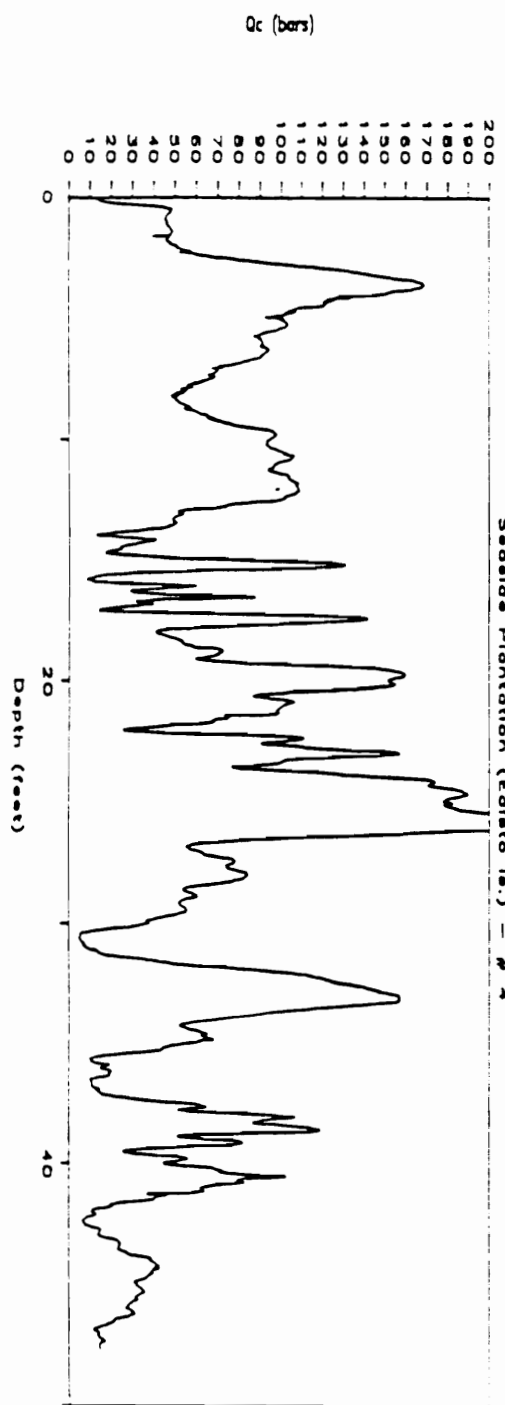
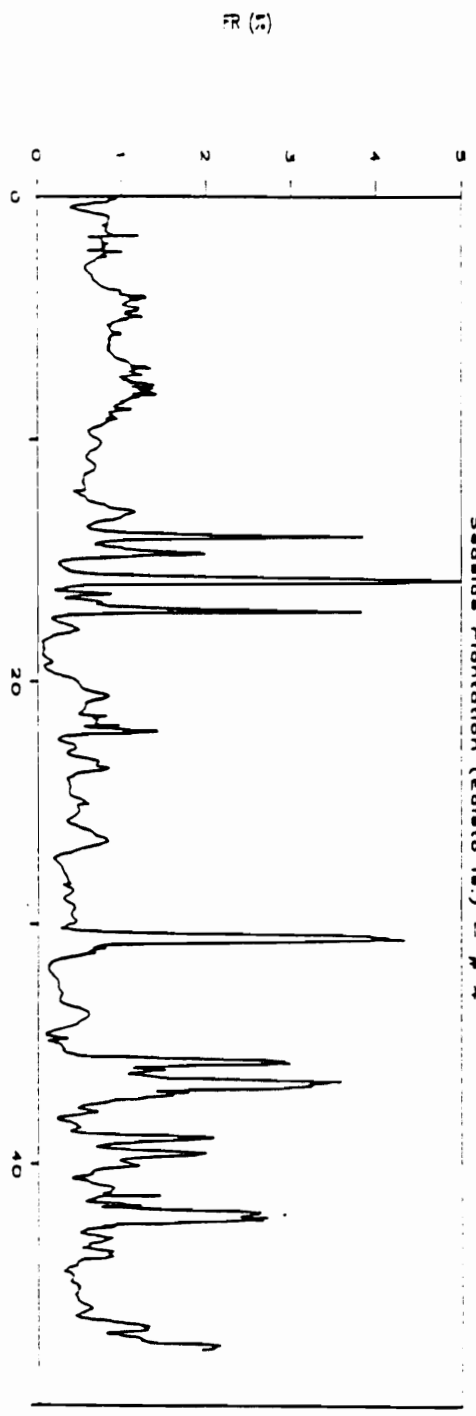


Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 3

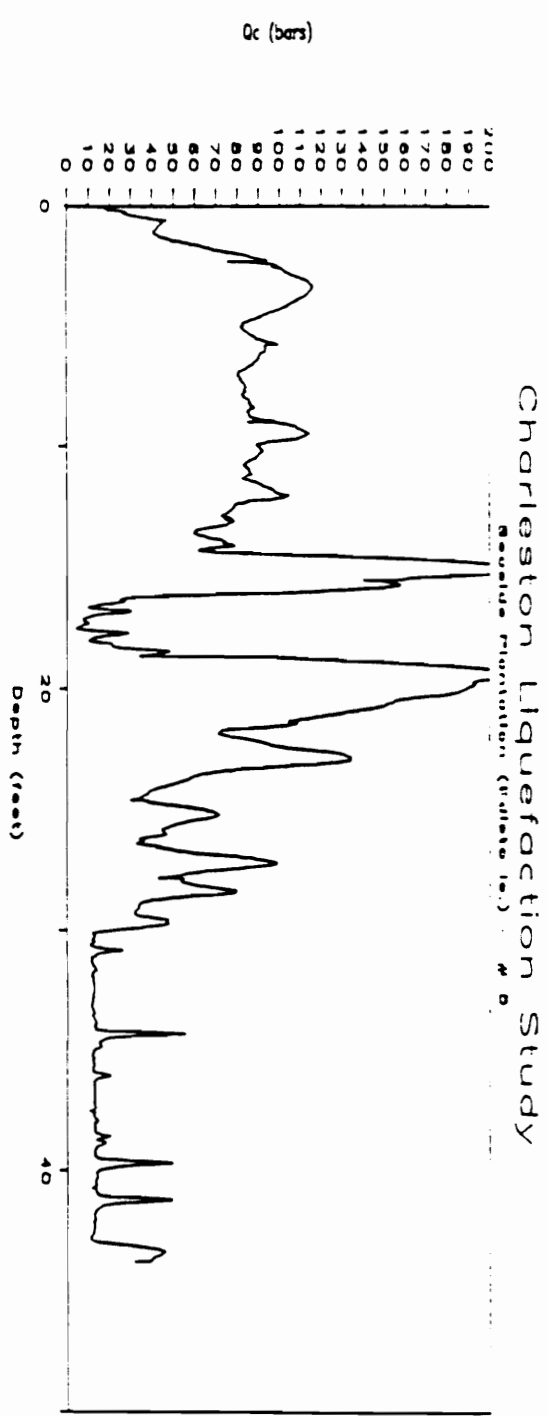
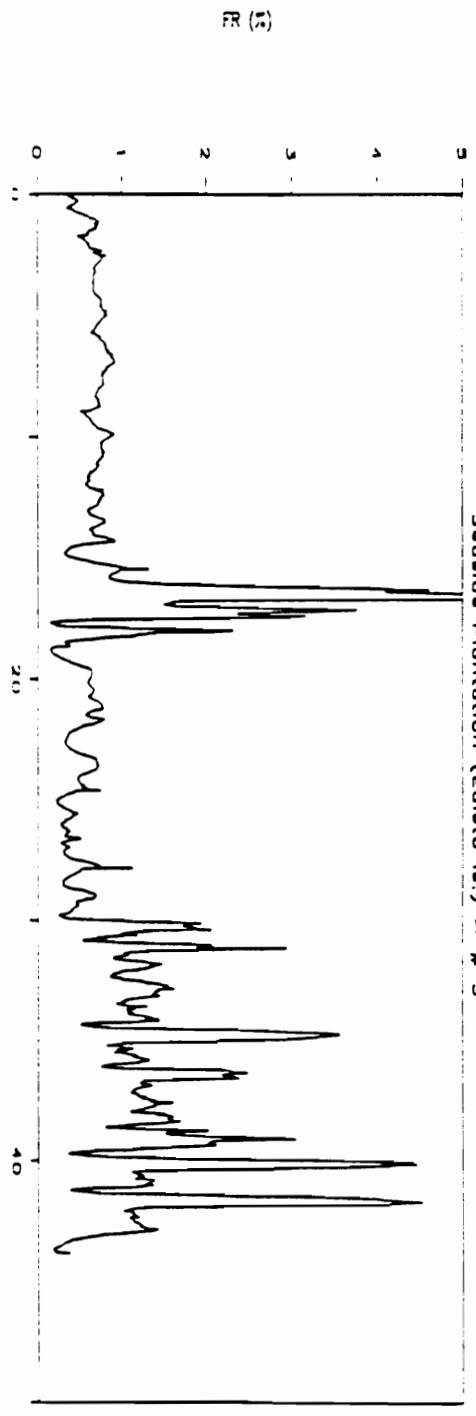


Charleston Liquefaction Study Seaside Plantation (Edisto Is.) - # 4



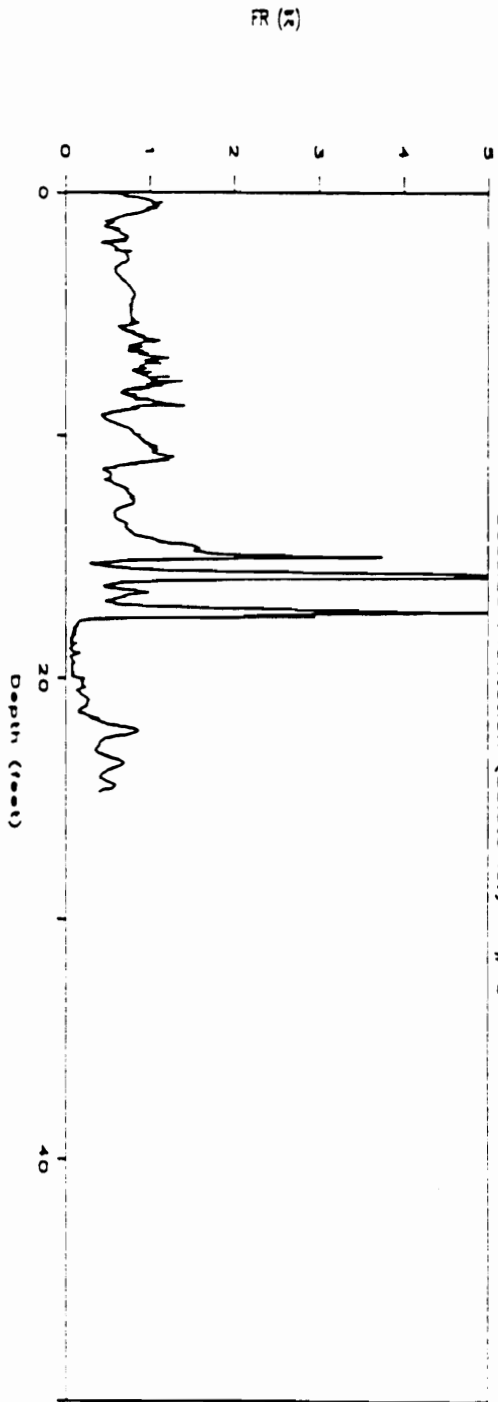
Charleston Liquefaction Study Seaside Plantation (Edisto Is.) - # 4

Charleston Liquefaction Study
 Seaside Plantation (Edisto Is.) - # 5



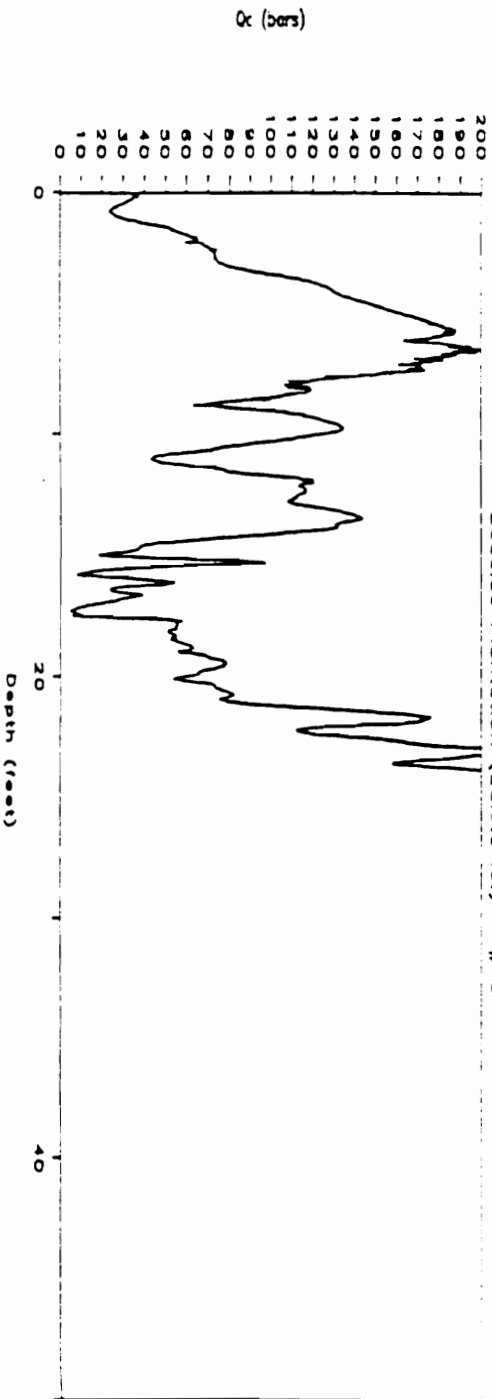
Charleston Liquefaction Study

Seaside Piontation (Edisto Is.) - # 6



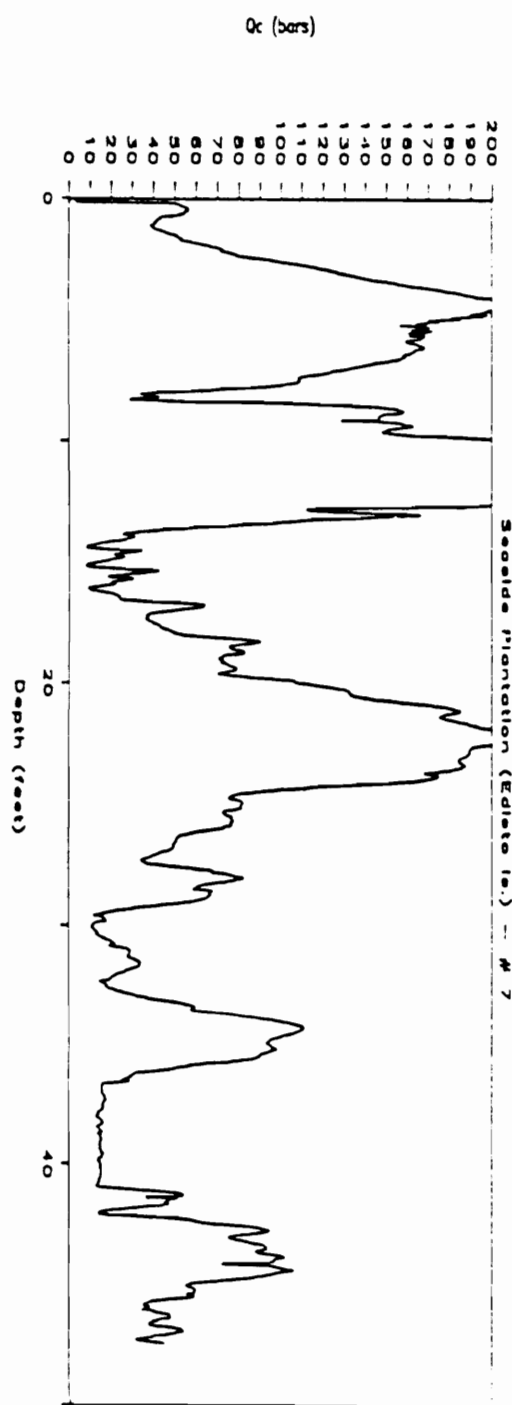
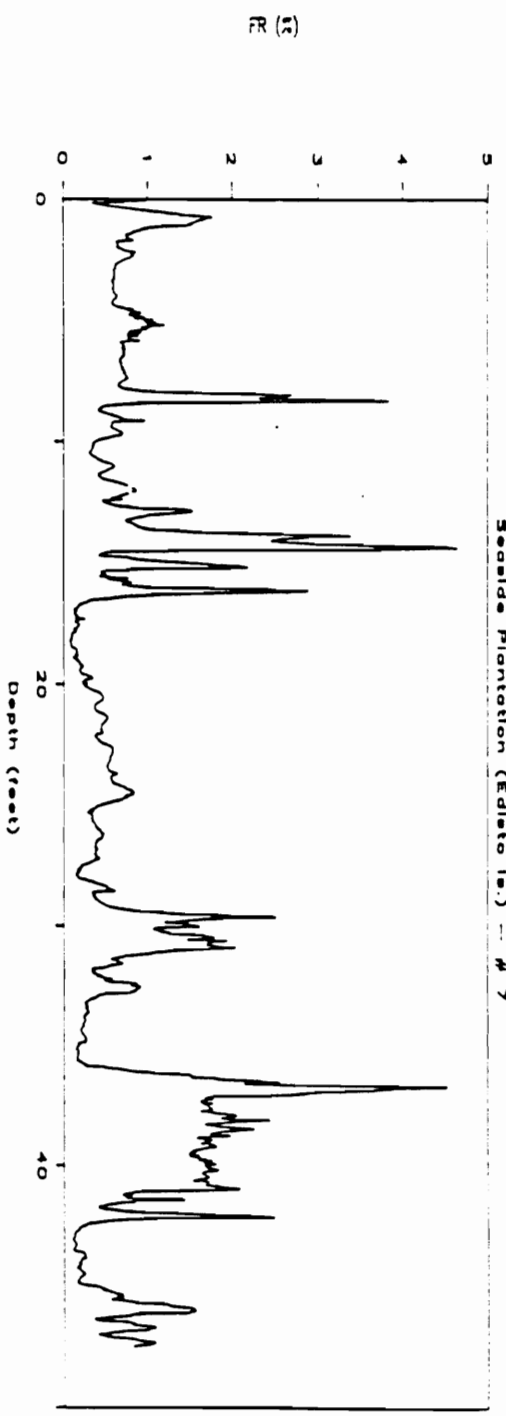
Charleston Liquefaction Study

Seaside Piontation (Edisto Is.) - # 6



Charleston Liquefaction Study

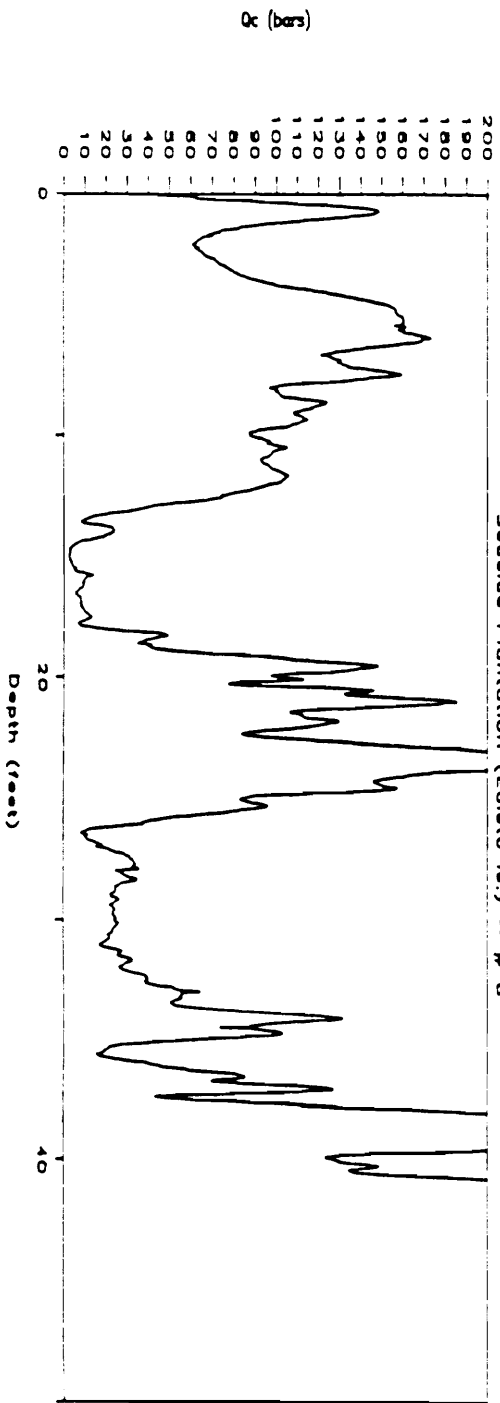
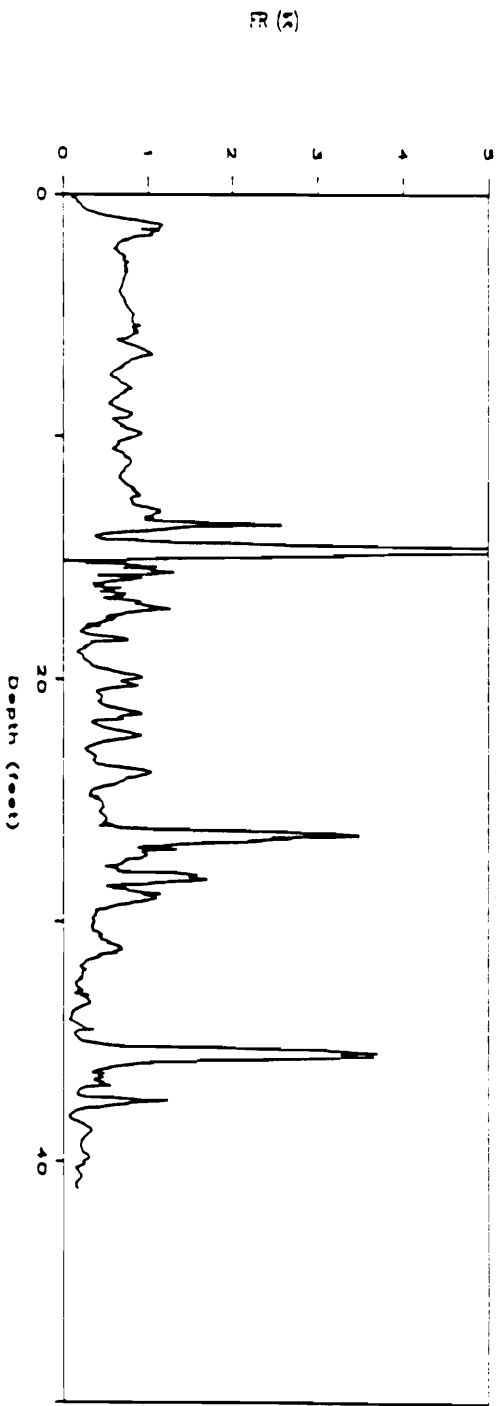
Seaside Plantation (Edisto Is.) - # 7



Charleston Liquefaction Study

Seaside Plantation (Edisto Is.) - # 7

Charleston Liquefaction Study Seaside Plantation (Edisto Is.) -- # 8



LOG of BORING

Project: CHARLESTON LINGUAFRAXIUM PROJECT - SEASIDE PLANTATION (EDISTO IS., S.C.)

Boring No. SSIDE PLAN 1 S. C. I. Project No. _____ Date JULY 1958

Ground Surface Elev. Assumed 0'0" Datum _____ Gr. Water Elev. 3'0"
(Minimum of 24 hrs. after completion)

Sample No.	SAMPLE Elev. - Feet & Inches		STRATUM Elev. - Feet & Inches		VISUAL FIELD CLASSIFICATION	Blows Per 6 inches
	From	To	From	To		
1	1'0"	2'6"			MEDIUM BROWN FINE SAND W/ GRAINICS	5-5-7
2	2'6"	4'0"			MEDIUM TAN FINE SAND	4-8-9
3	5'0"	6'6"			MEDIUM TAN FINE SAND	3-6-9
4	7'6"	9'0"			MEDIUM GRAY TAN FINE SAND	3-5-8
5	10'0"	11'6"			LOOSE GRAY FINE SAND W/ MICA	3-4-3
6	12'6"	14'0"			LOOSE GREEN/GRAY FINE SAND W/ MICA	3-2-7
7	15'0"	16'6"			MEDIUM GRAY FINE SAND W/ MICA'S SHELLS	6-7-6
8	17'6"	19'0"			LOOSE GRAY MEDIUM SAND W/ SHELL HASH	4-5-3
9	20'0"	21'6"			DENSE GRAY FINE SAND W/ SHELL HASH	2-11-25
10	25'0"	26'6"			MEDIUM GRAY FINE SAND W/ SOME SHELL	6-8-9
11	30'0"	31'6"			MEDIUM GRAY FINE SAND W/ SHELL HASH	3-5-6
12	35'0"	36'6"			LOOSE GRAY COARSE SAND W/ SHELL HASH	2-3-3
	36'6"	38'0"			REVERSED SHELDON TUBE SAMPLE (CLAY)	
13	40'0"	41'6"			V. SOFT GRAY CLAY	1 1/2" - 3/6"
14	45'0"	46'6"			MEDIUM GRAY FINE SAND W/ SHELL	7-10-12
15	50'0"	51'6"			** GREENISH BROWN SILTY SANDS	2-5-6
					CALCAREOUS MATERIAL (STIFF)	
	52'0"	54'2"			REVERSED SHELDON TUBE SAMPLE (MARL)	

Testing and Sampling in accordance with ASTM D. 1586-7 (1974)

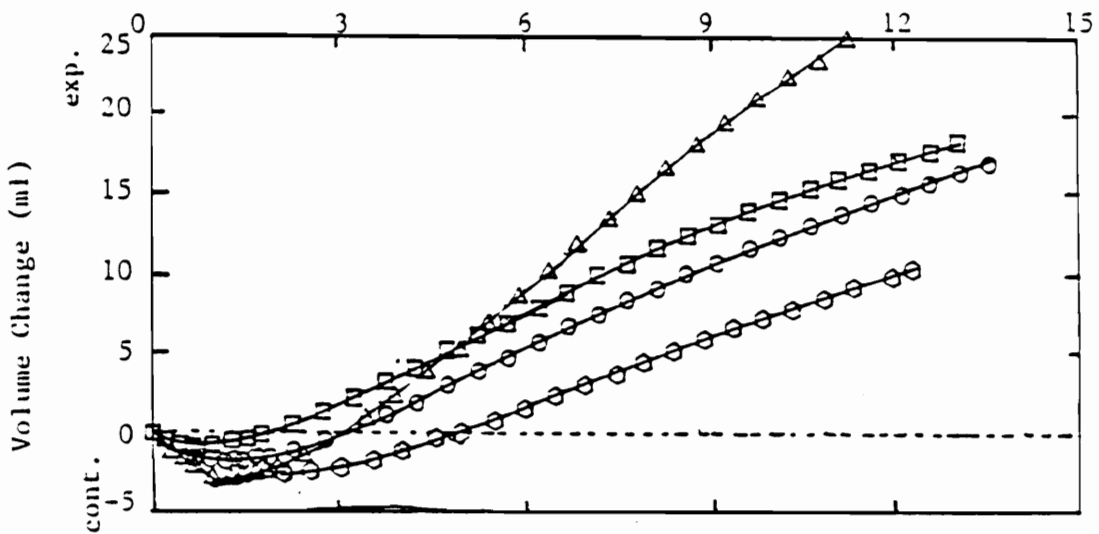
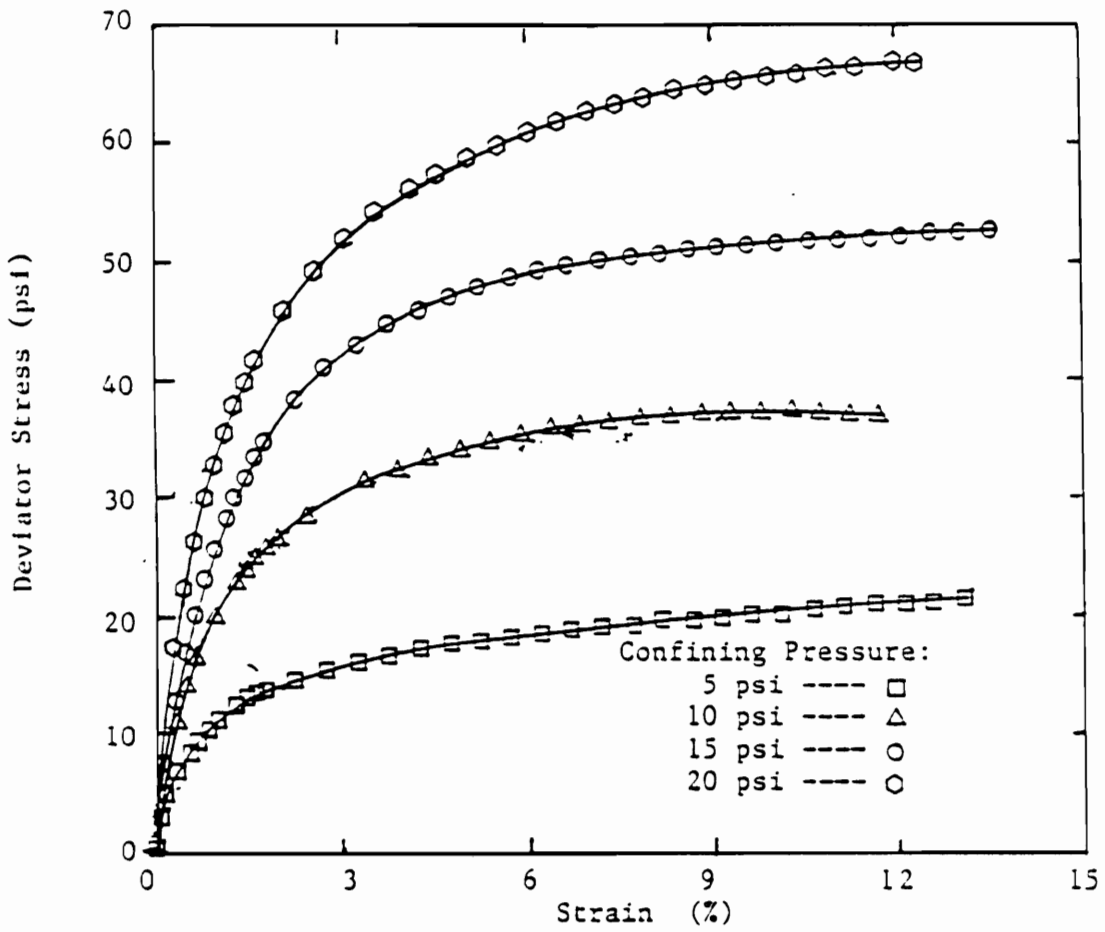
Remarks: ** COOPER MARL

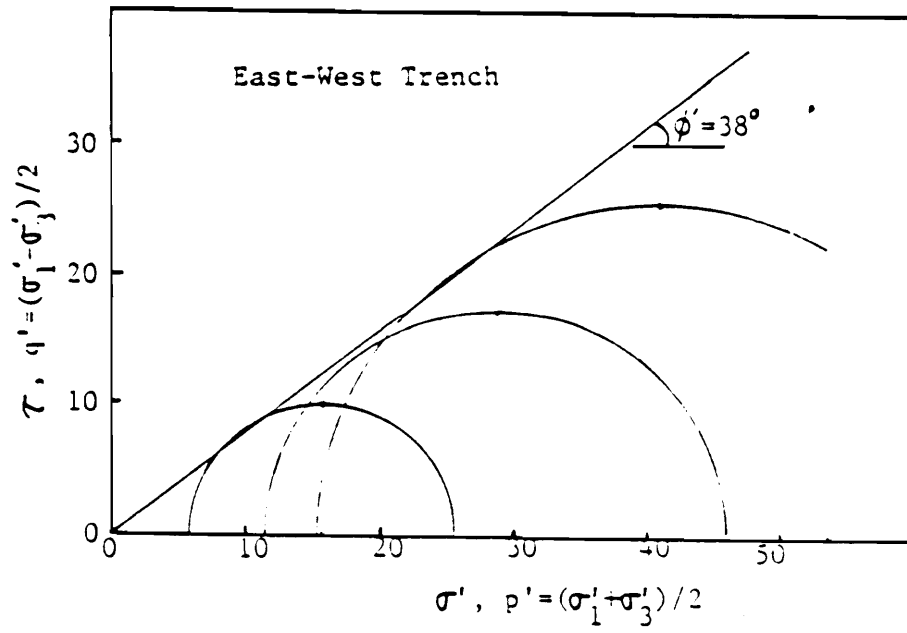
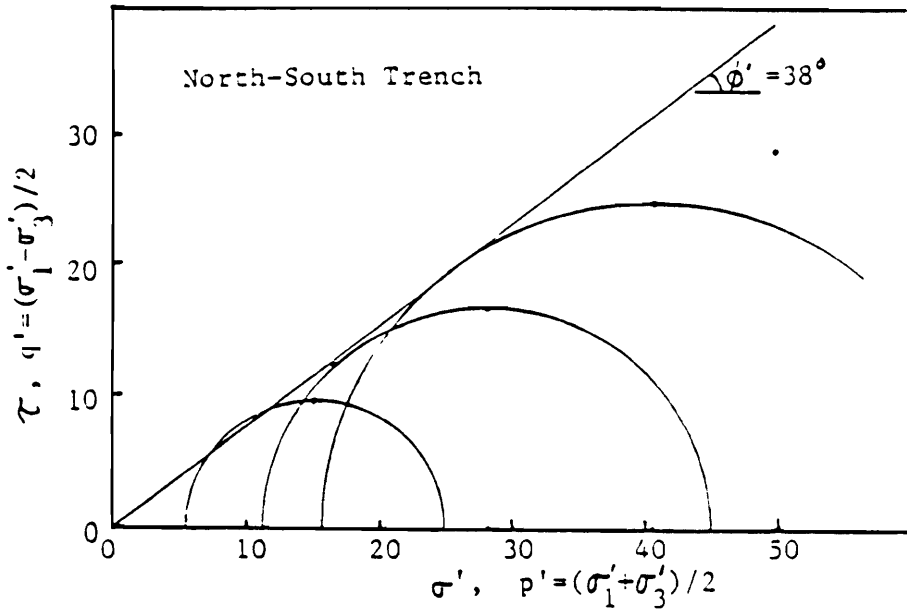
MUSQUITO HEAVEN IS AT EDISTO ISLAND!

APPENDIX C

STATIC AND CYCLIC TRIAXIAL TESTS ON CHARLESTON SANDS, CLAYS AND
MARLS

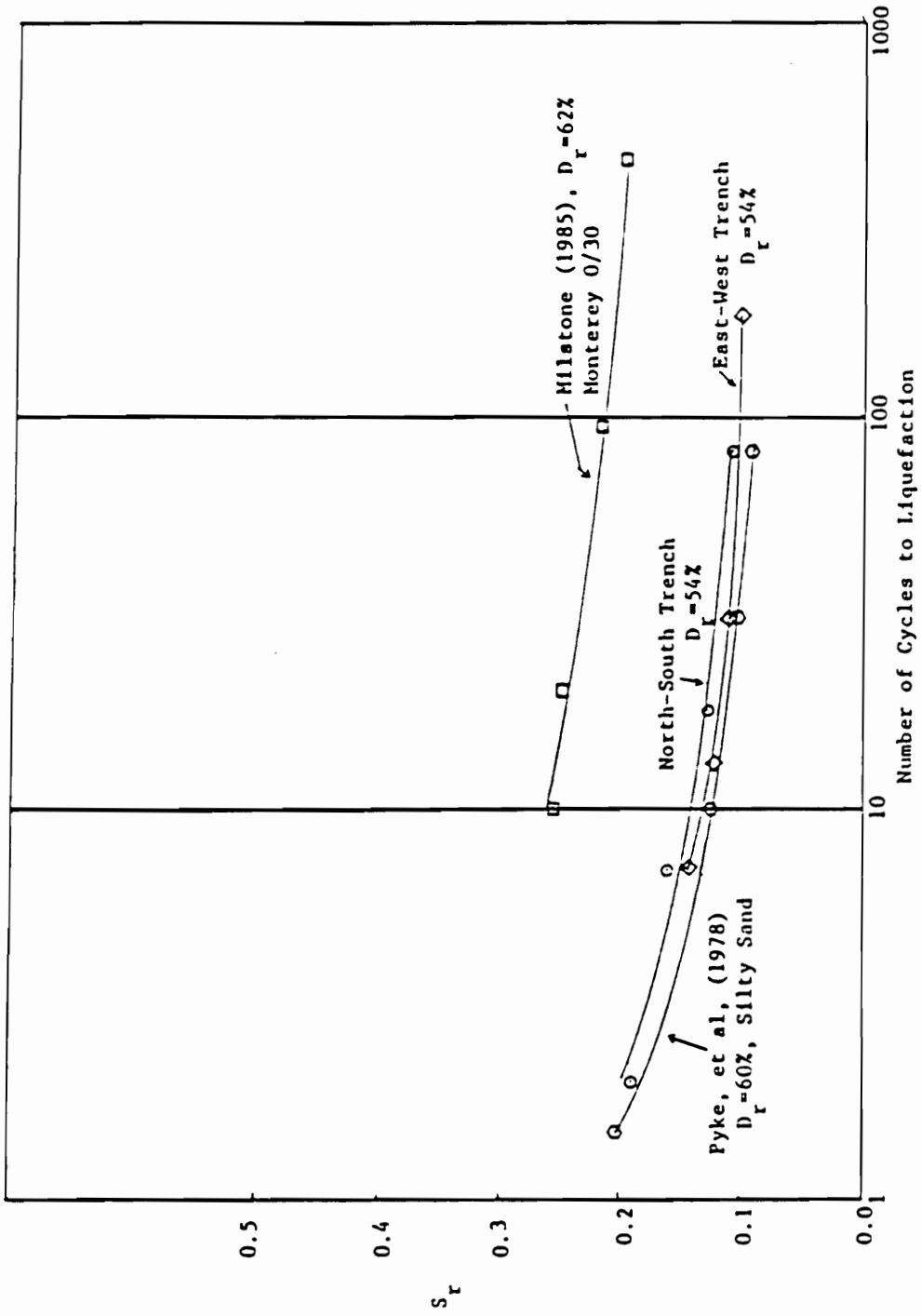
CD TESTS - HOLLYWOOD, S.C. SANDS





CYCLIC TRIAXIAL TESTS -- HOLLYWOOD S.C. SANDS

(EAST-WEST AND NORTH-SOUTH TRENCHES)

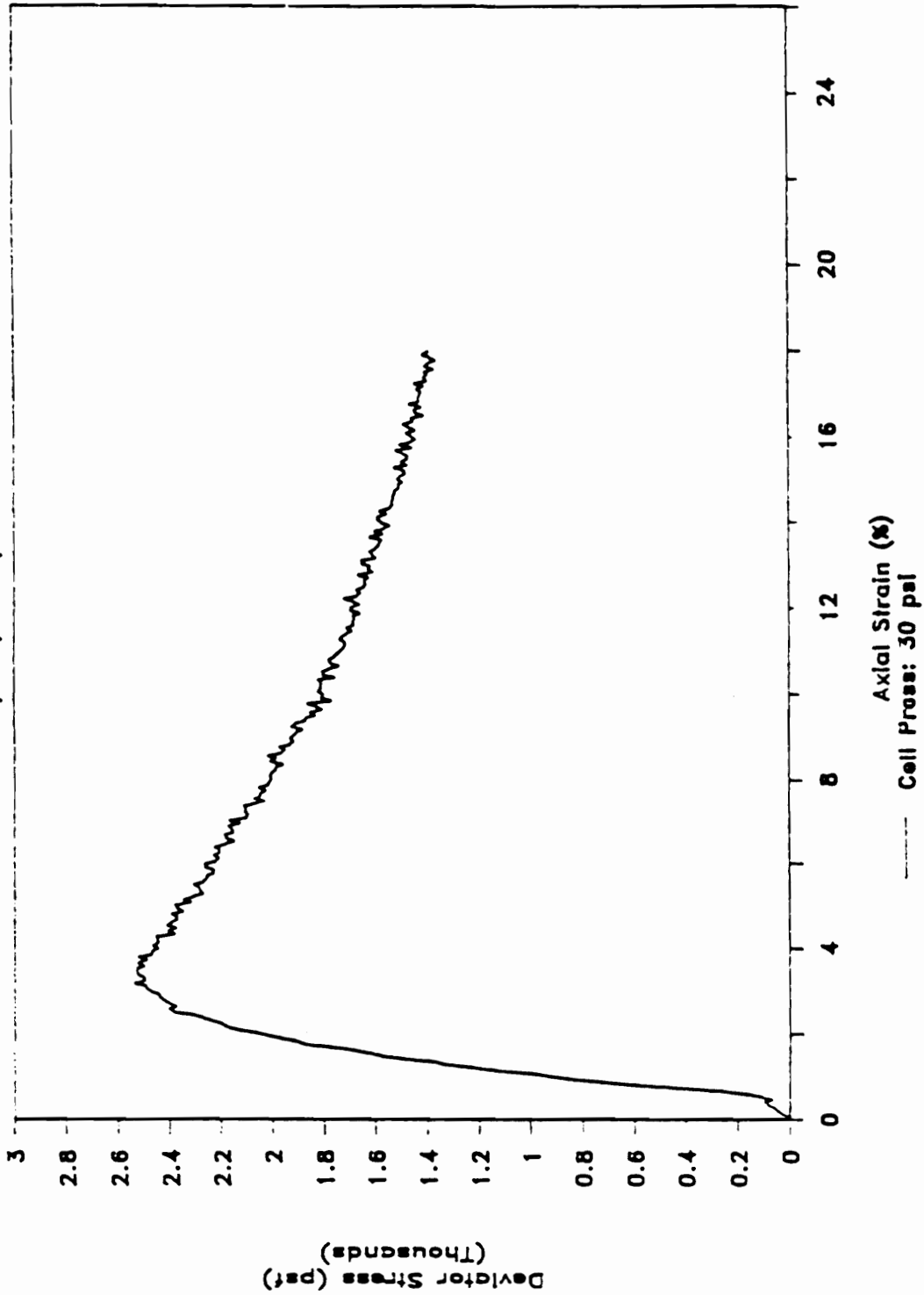


UU TESTS -- CHARLESTON CLAYS AND MARLS

(VARIOUS SITES)

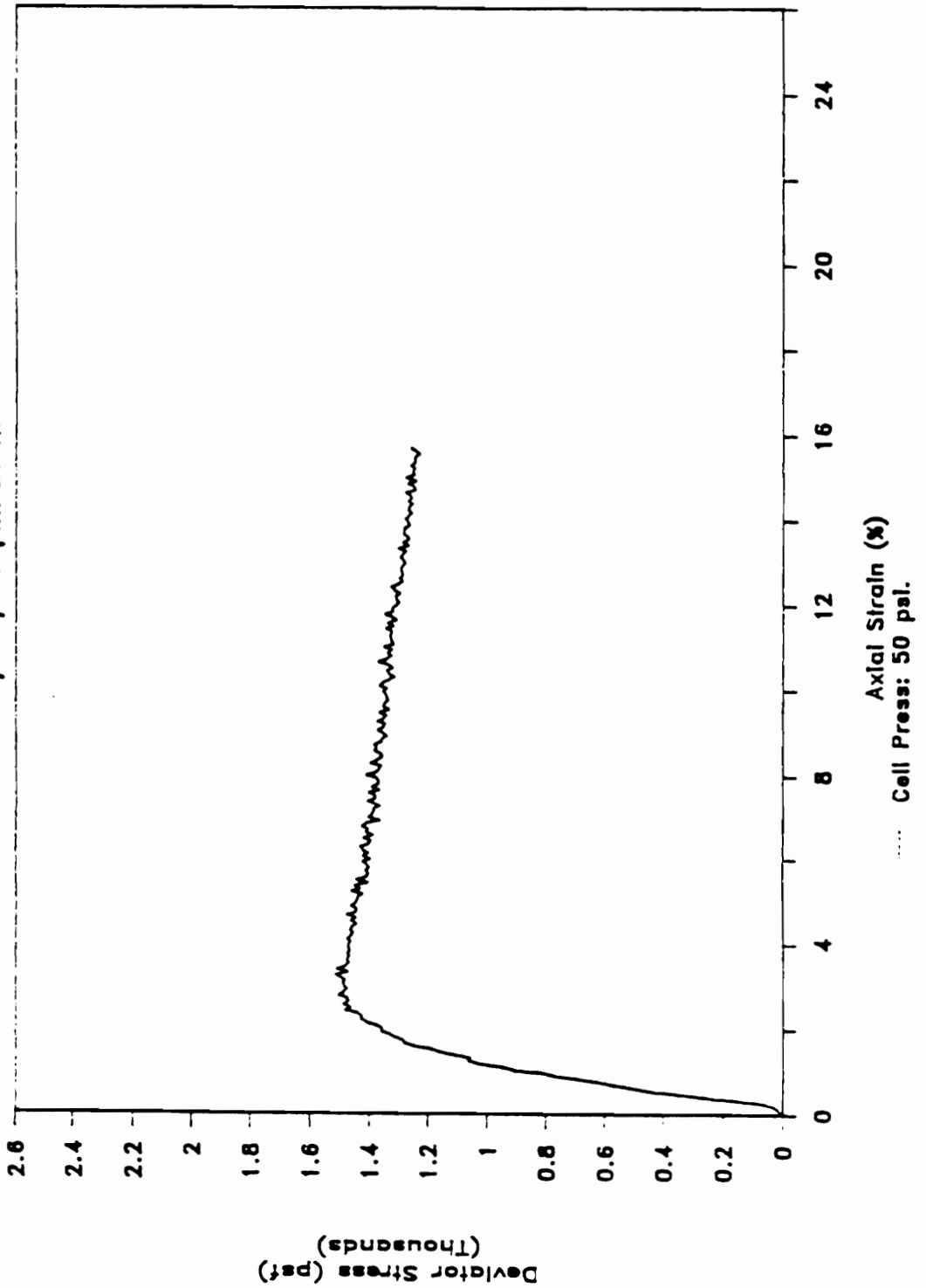
Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 24.5 ft.



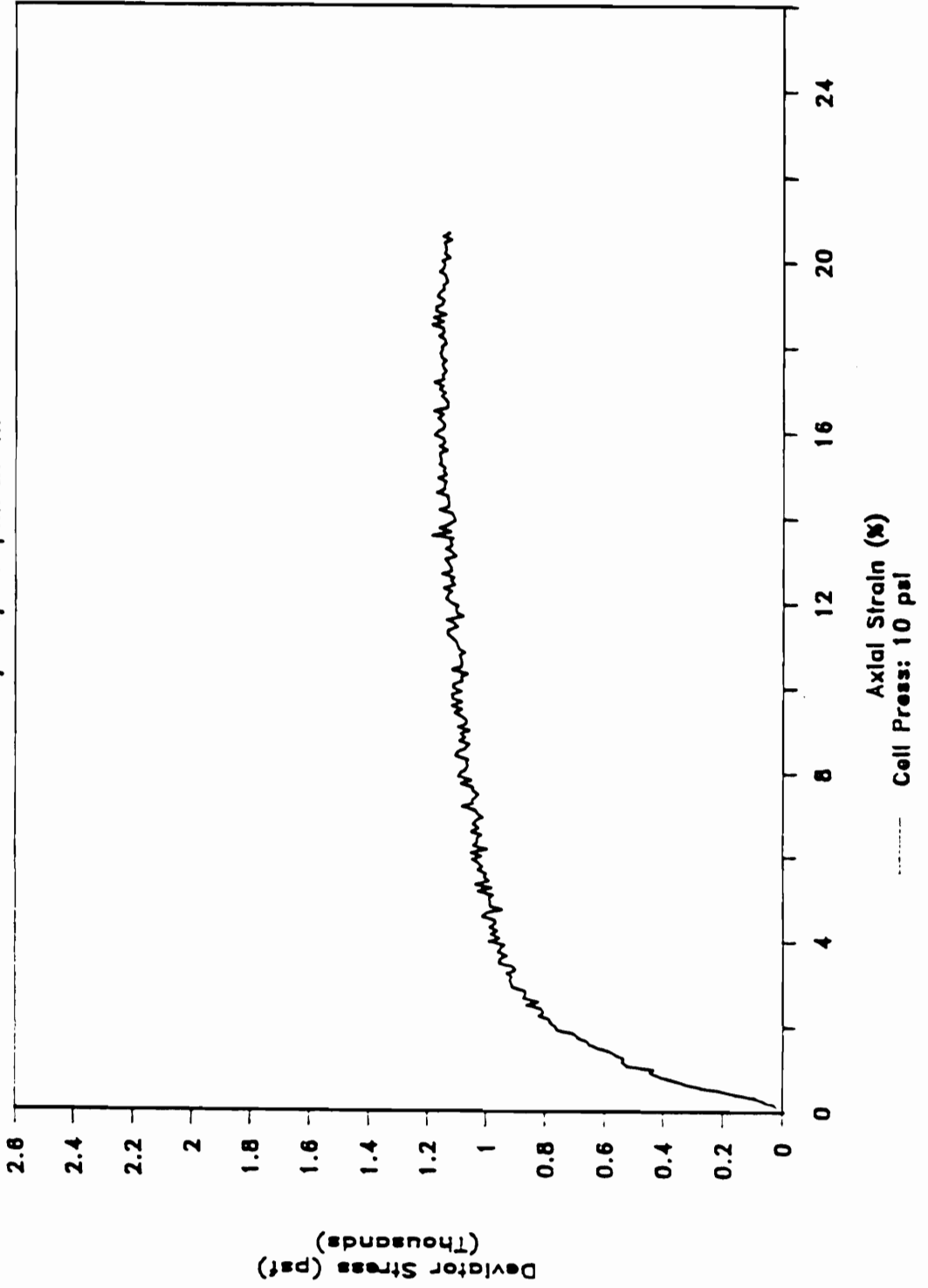
Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 25 ft.



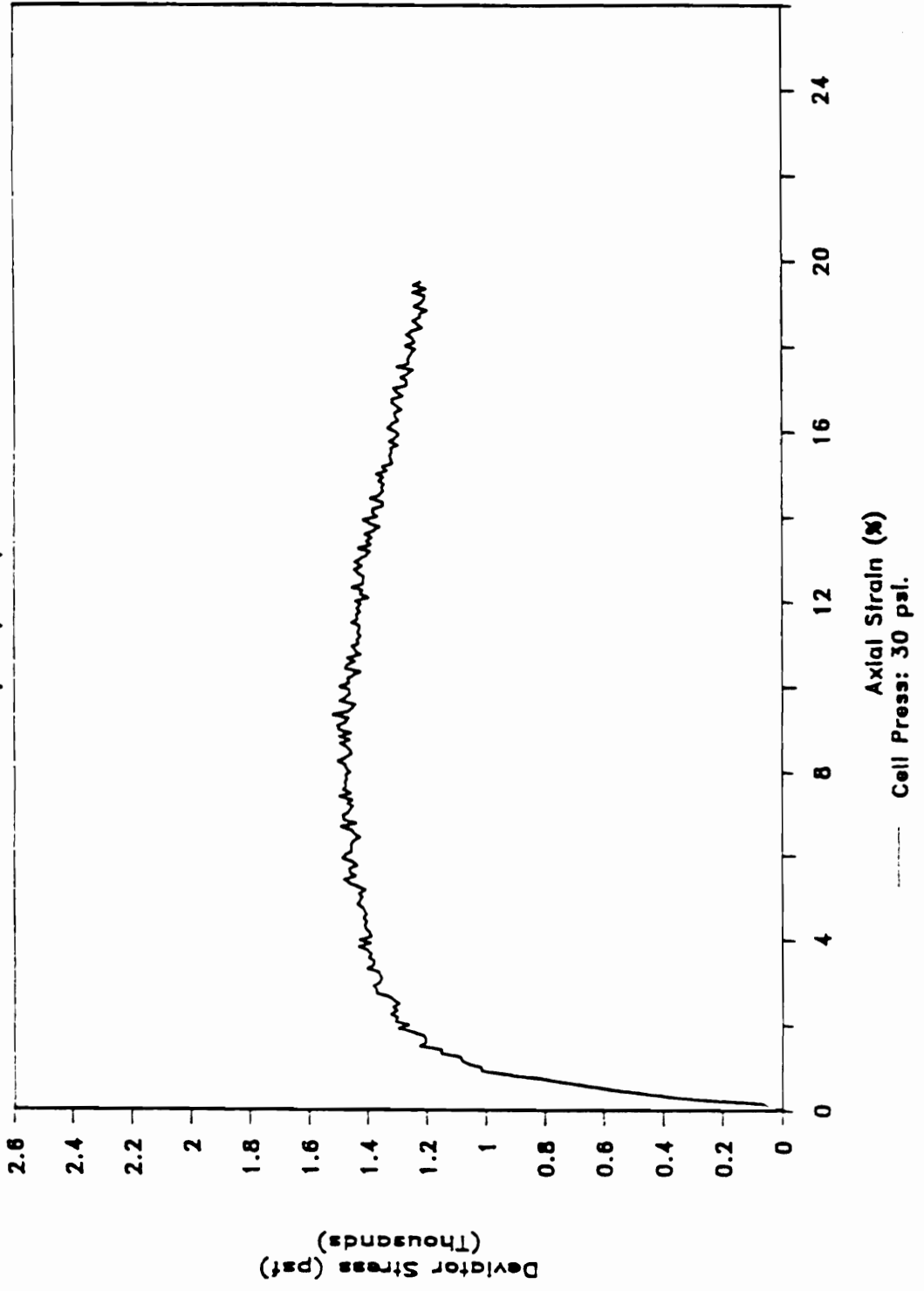
Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 27 ft.



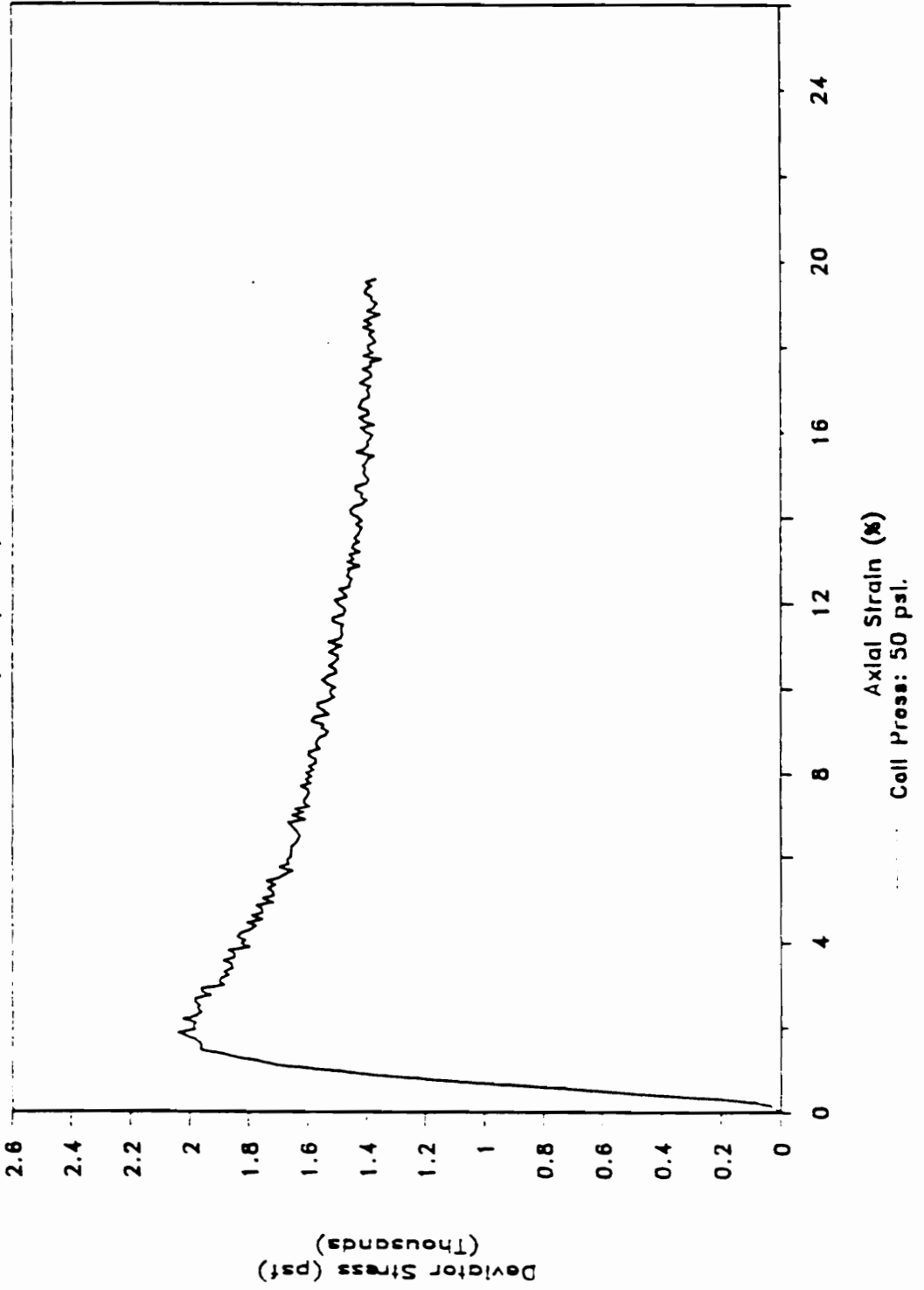
Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 27.5 ft.



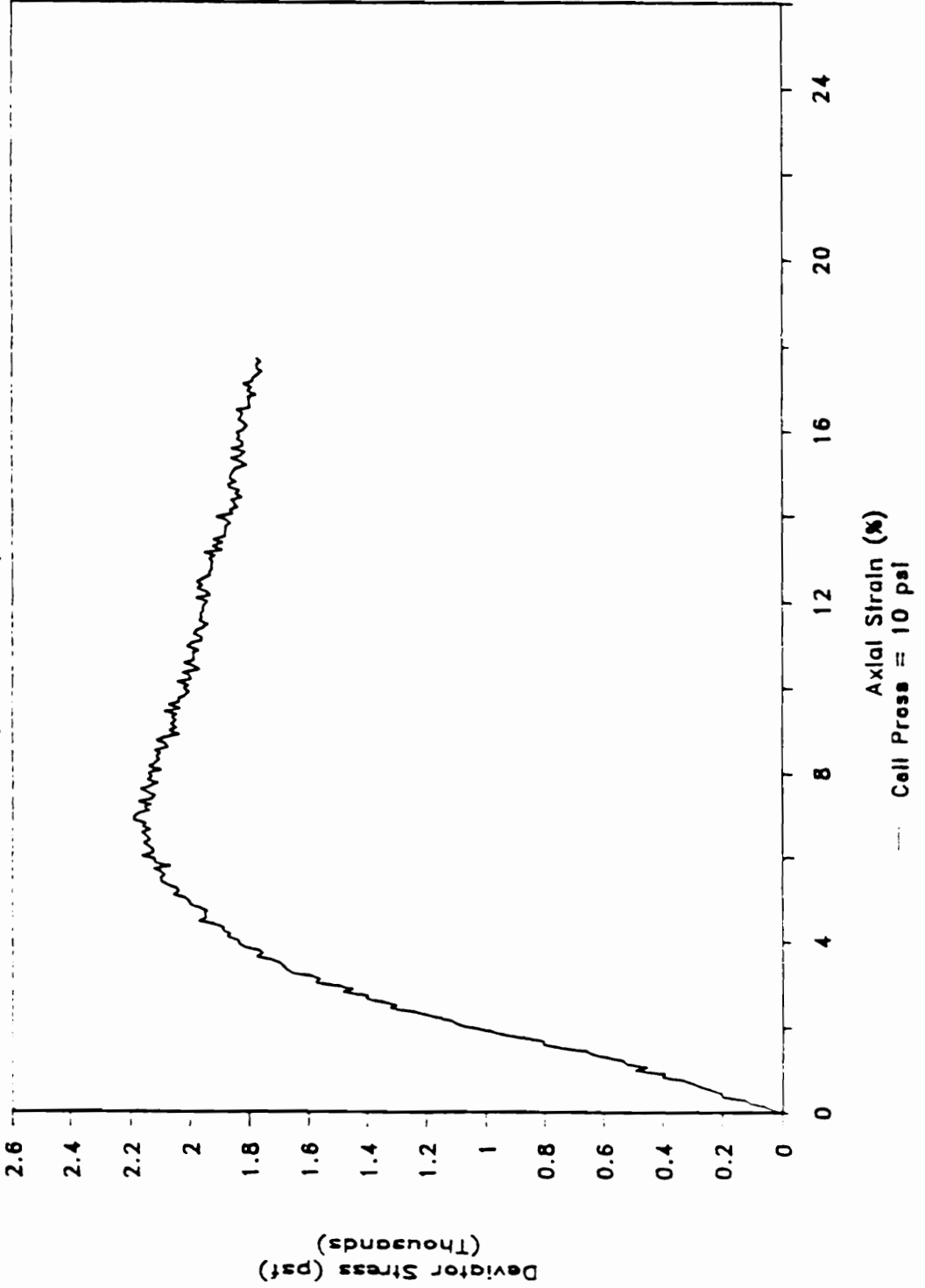
Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 28 ft.



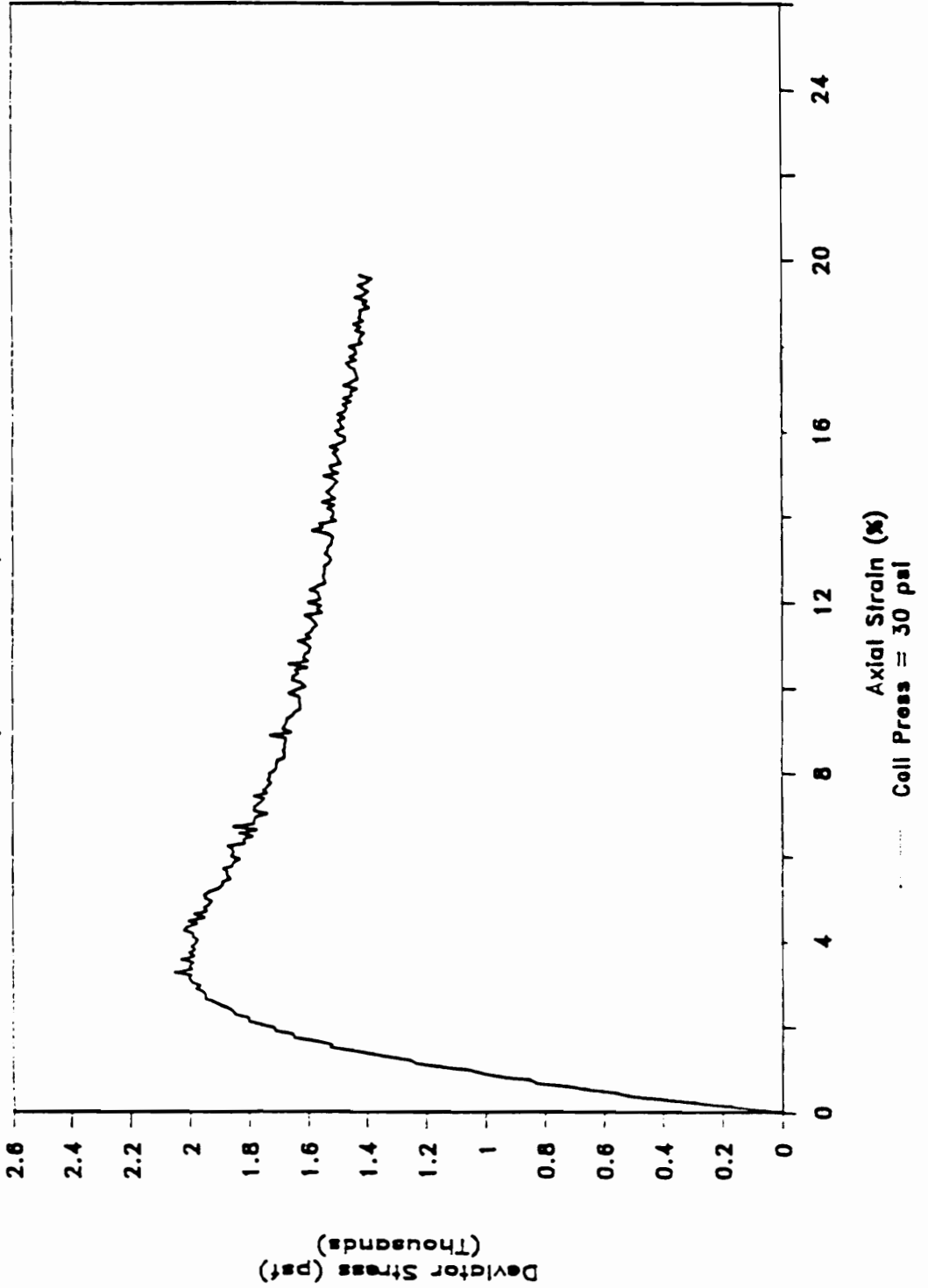
Location: Mt. Pleasant Pits (MPPITS)

Cooper Marl - Depth 30.5 ft.



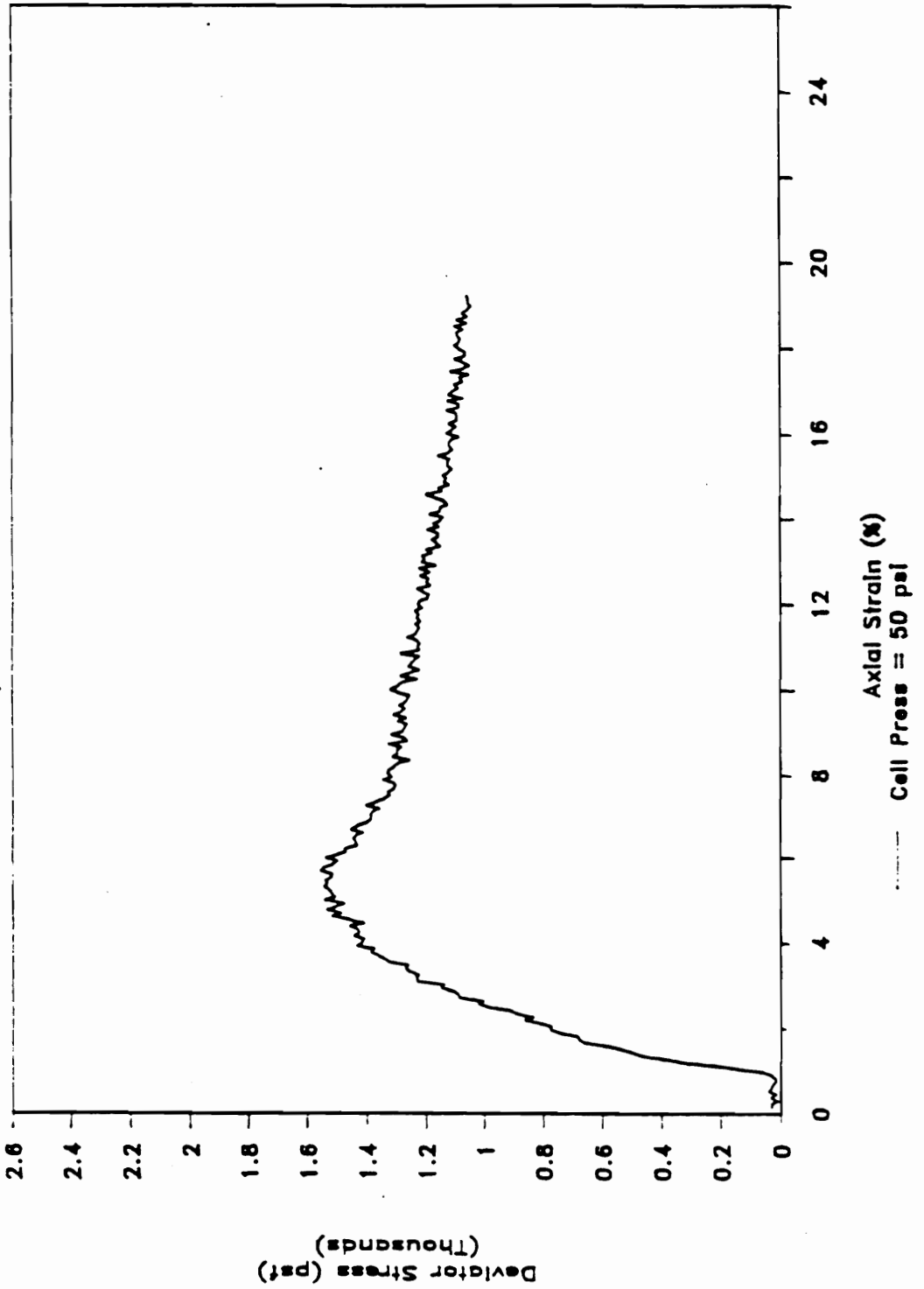
Location: Mt. Pleasant Pits (MPPITS)

Cooper Marl - Depth 31.0 ft.



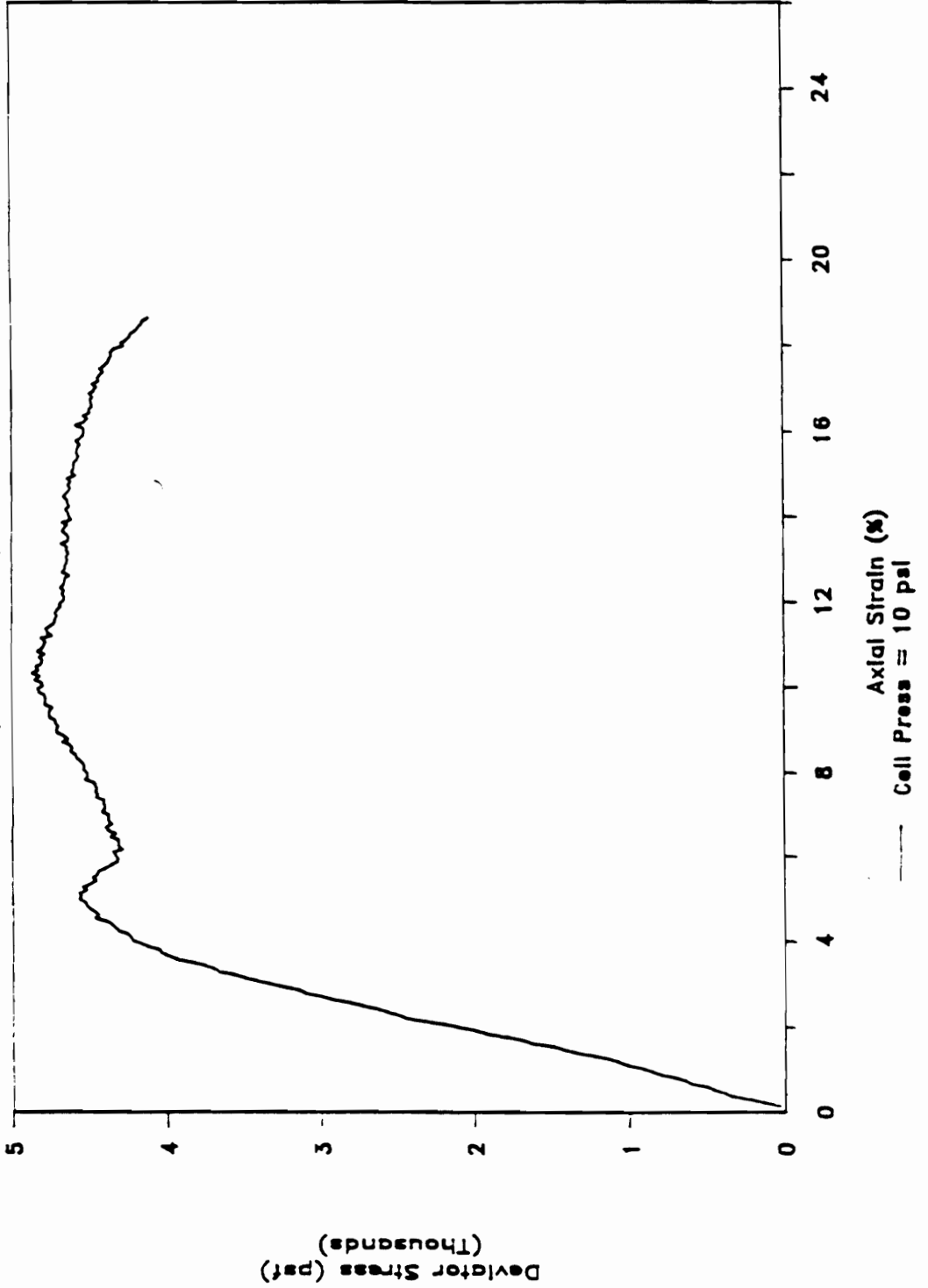
Location: Mt. Pleasant Pits (MPPITS)

Cooper Marl - 31.5 ft.



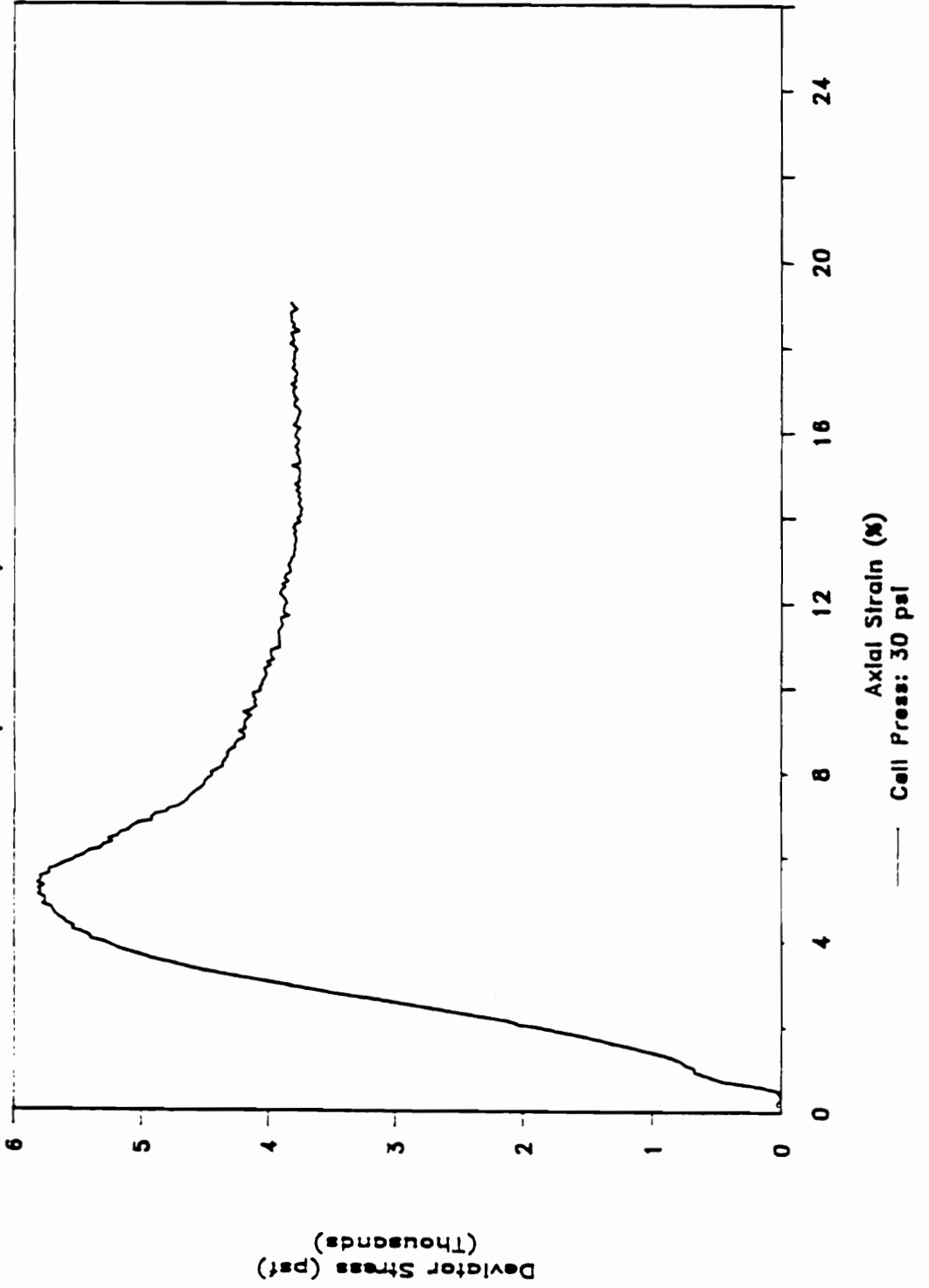
Location: St. James-Santee School (SJSS)

Cooper Marl -- Depth: 41 ft.



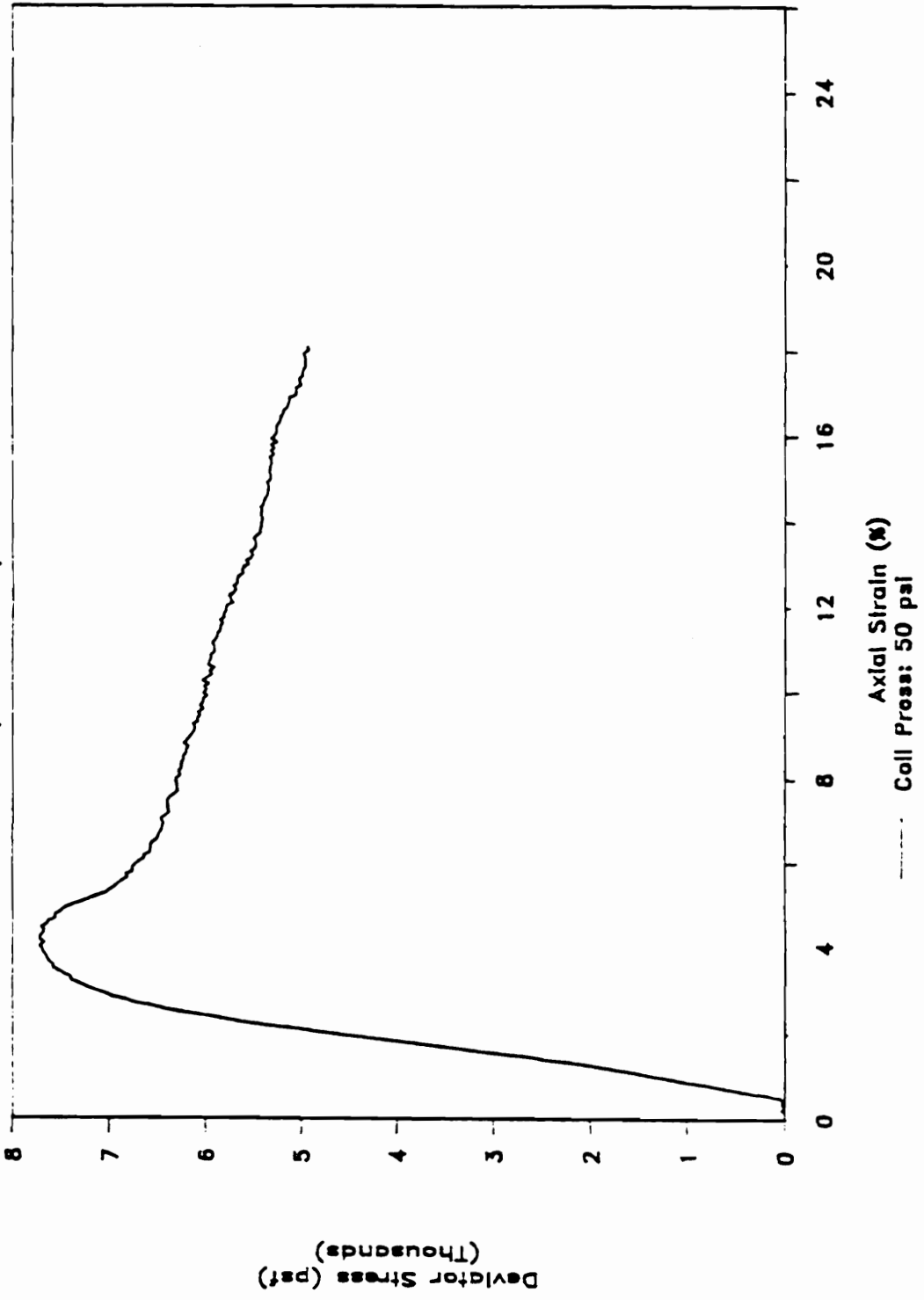
Location: St. James - Santee School (SJSS)

Cooper Marl - Depth: 41.5 ft.



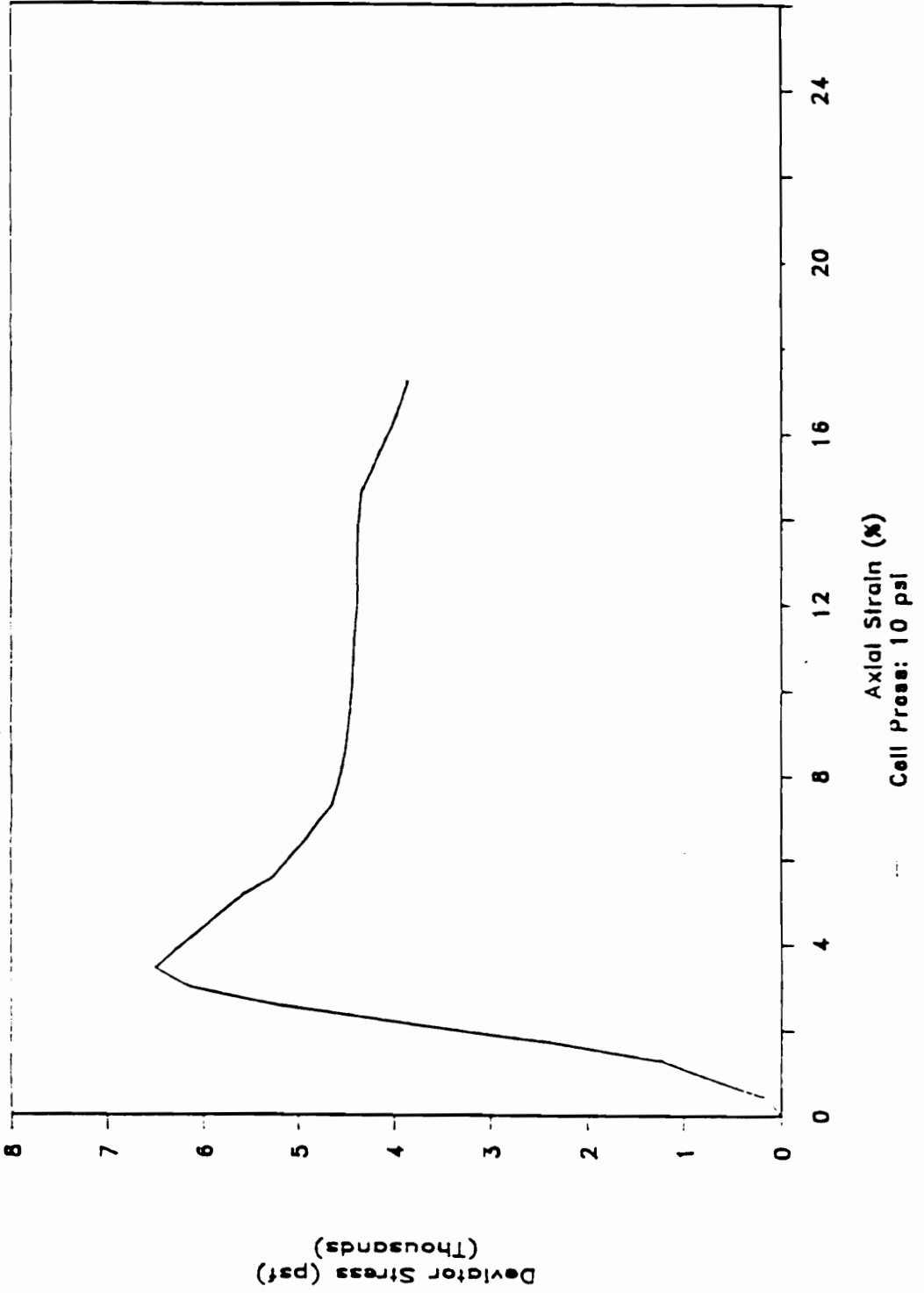
Location: St. James-Santee School (SJSS)

Cooper Marl - Depth: 42 Ft.



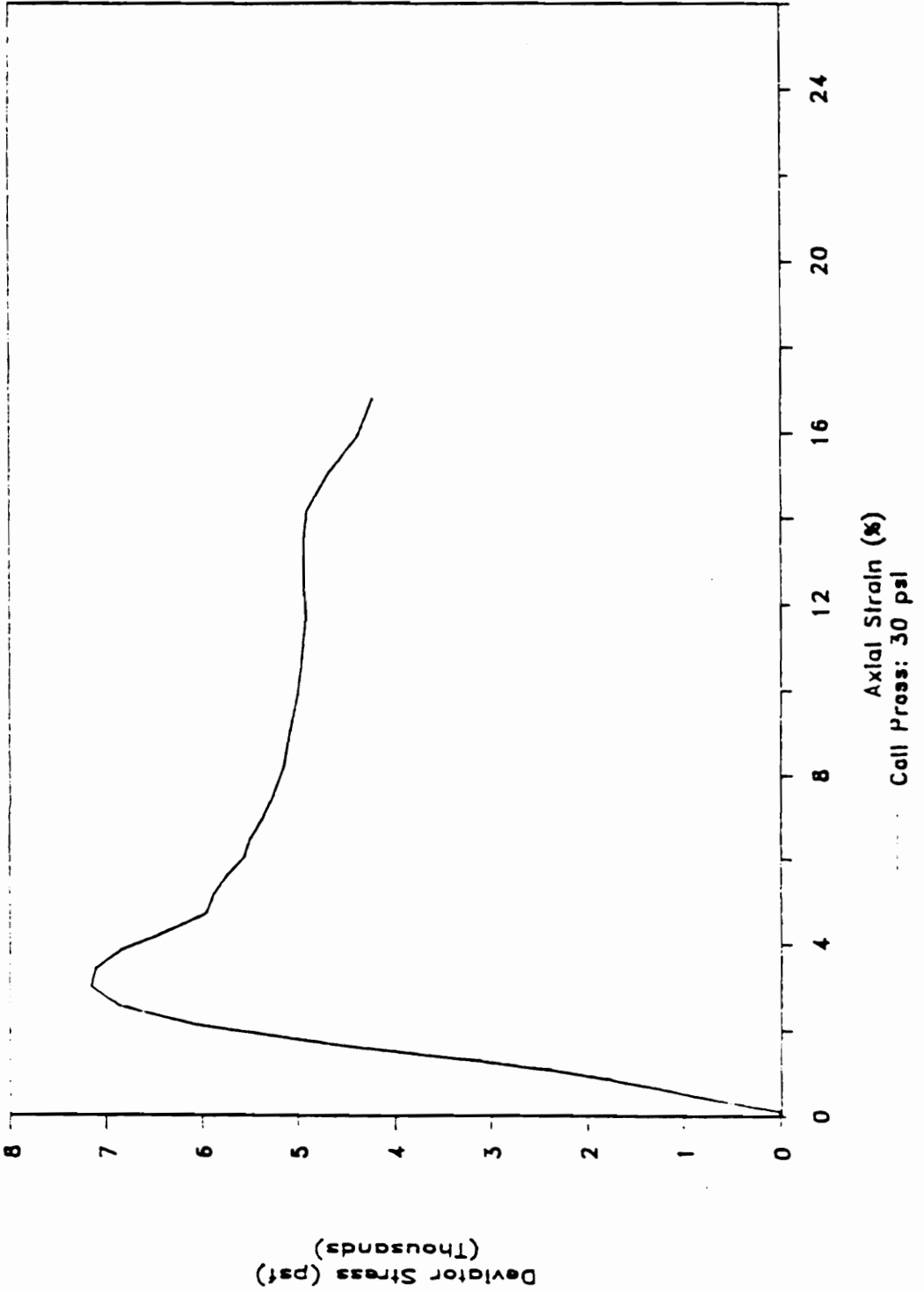
Location: St. James-Santee School (SJSS)

Cooper Marl- Depth: 42.5 ft.



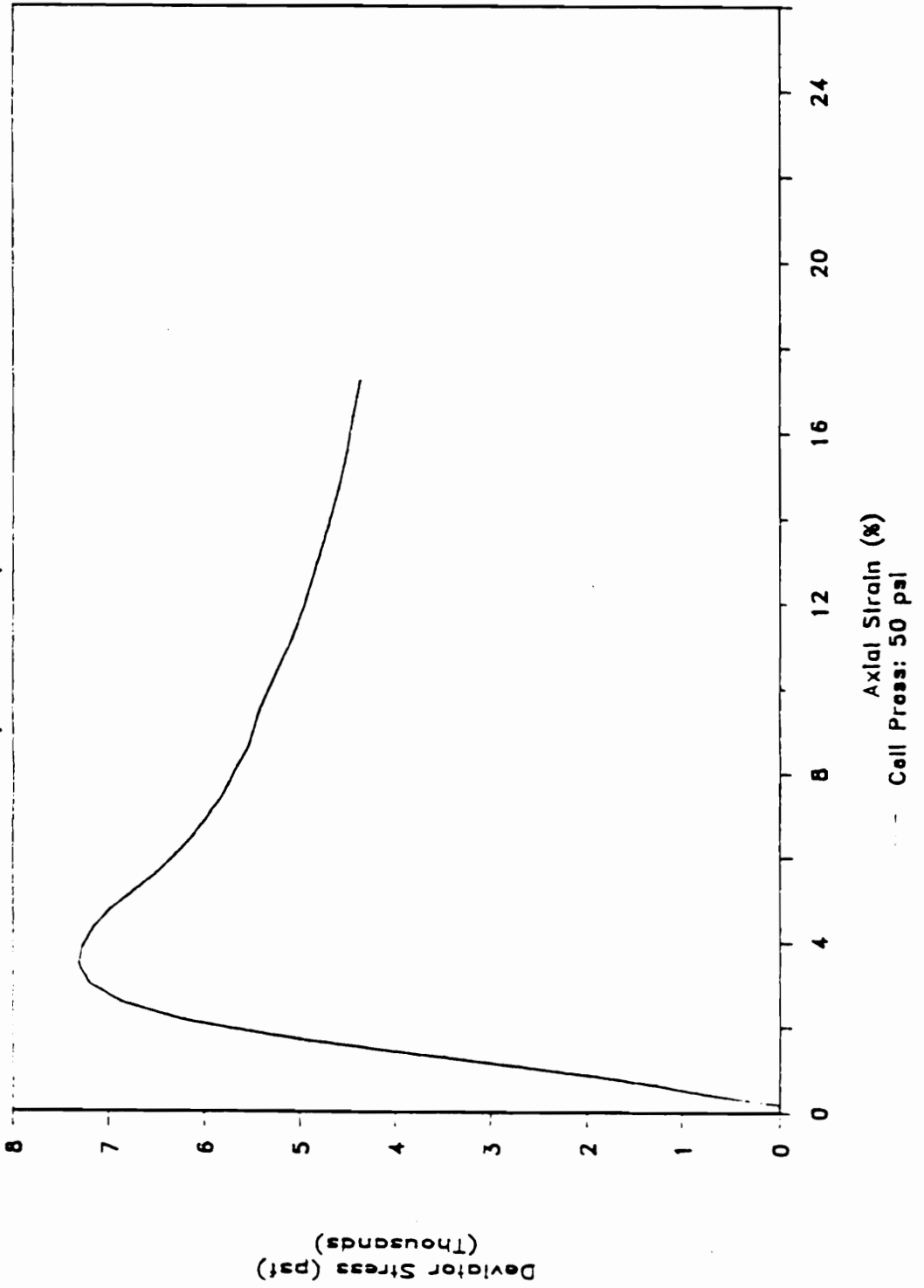
Location: St. James--Santee School (SJSS)

Cooper Marl-- Depth: 43.0 ft



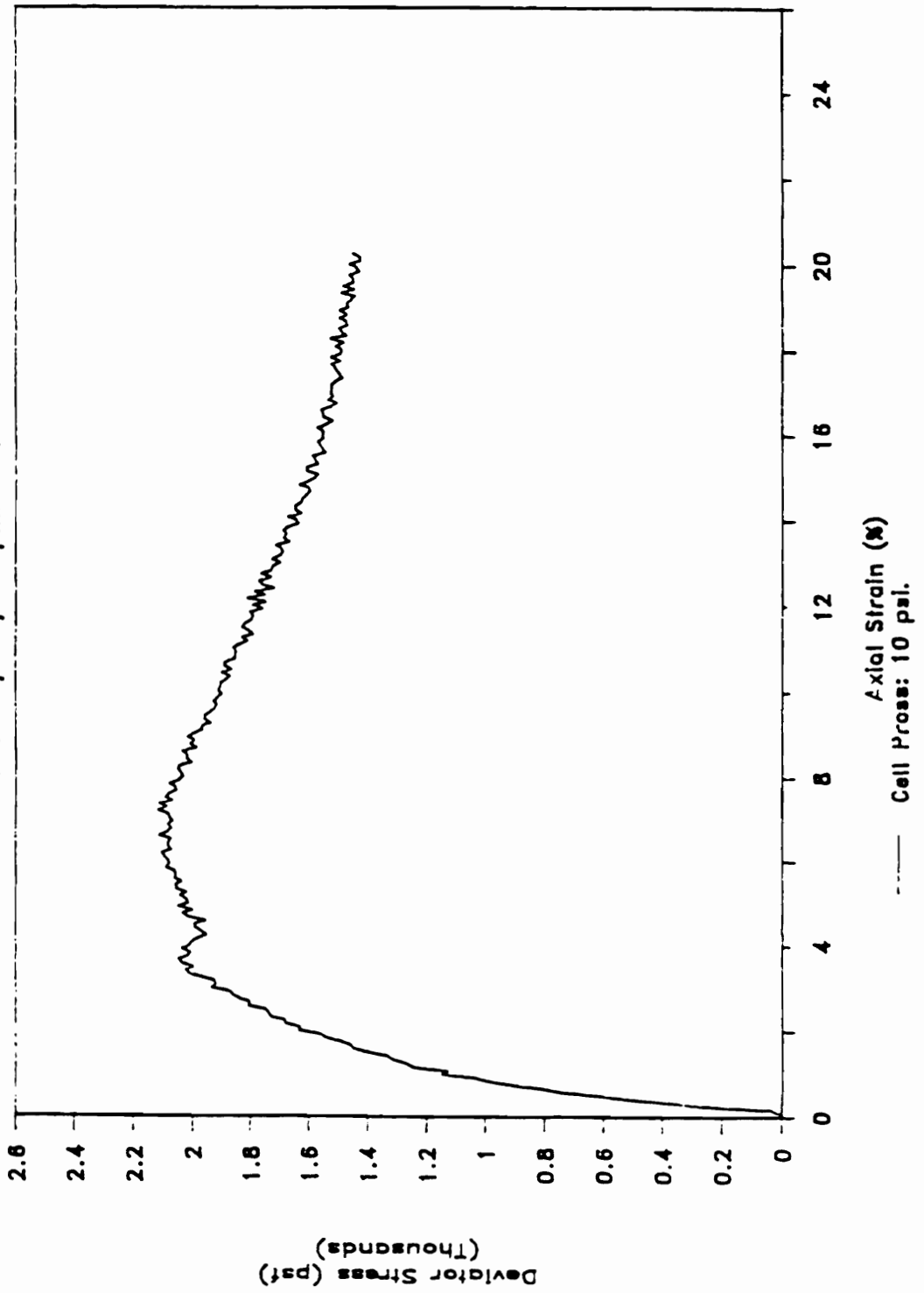
Location: St. James-Santee School (SJSS)

Cooper Marl - Depth: 43.5 ft.



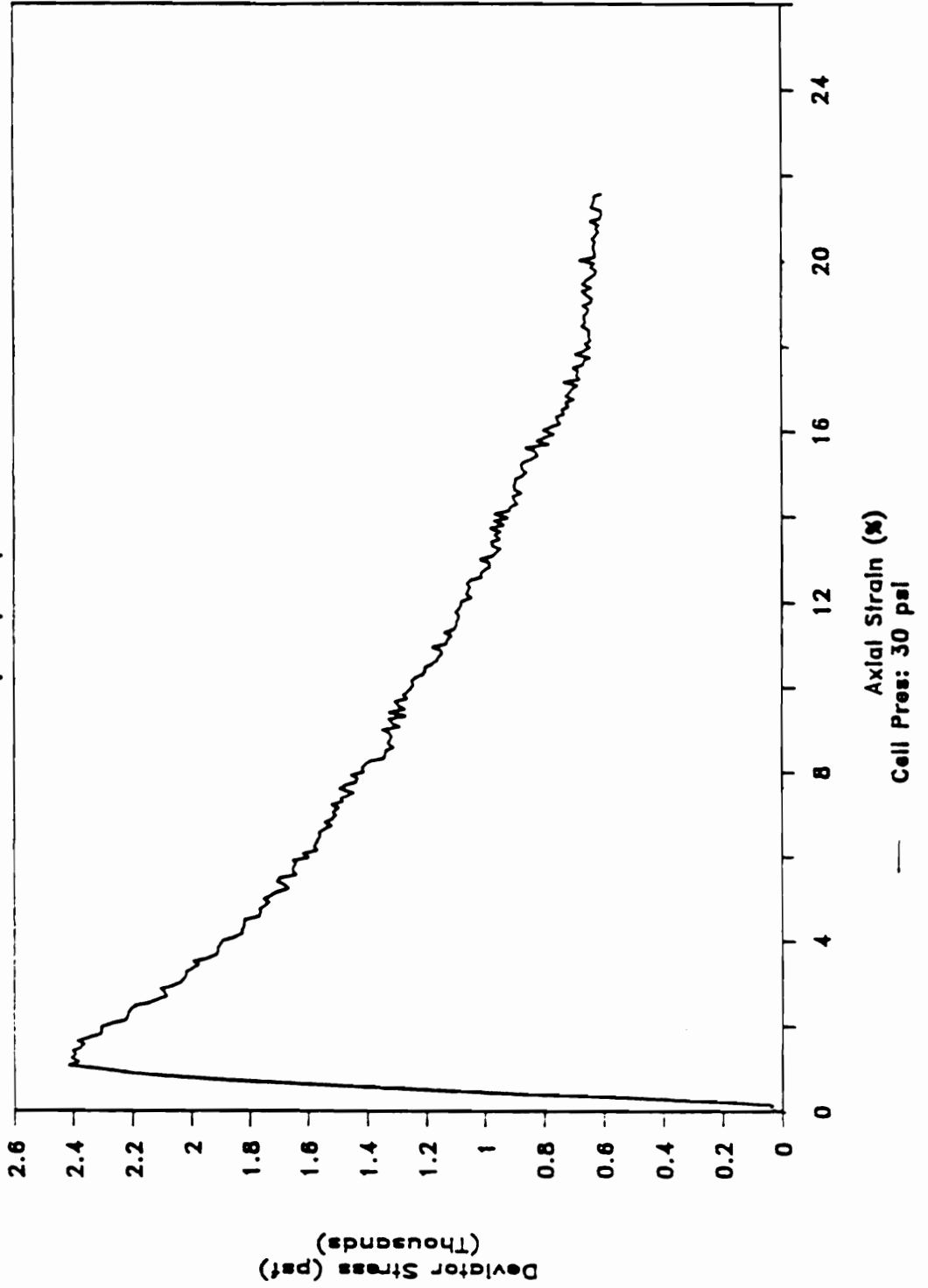
Location: Edisto Island (SSP)

Soft Gray Clay - Depth: 37 ft.



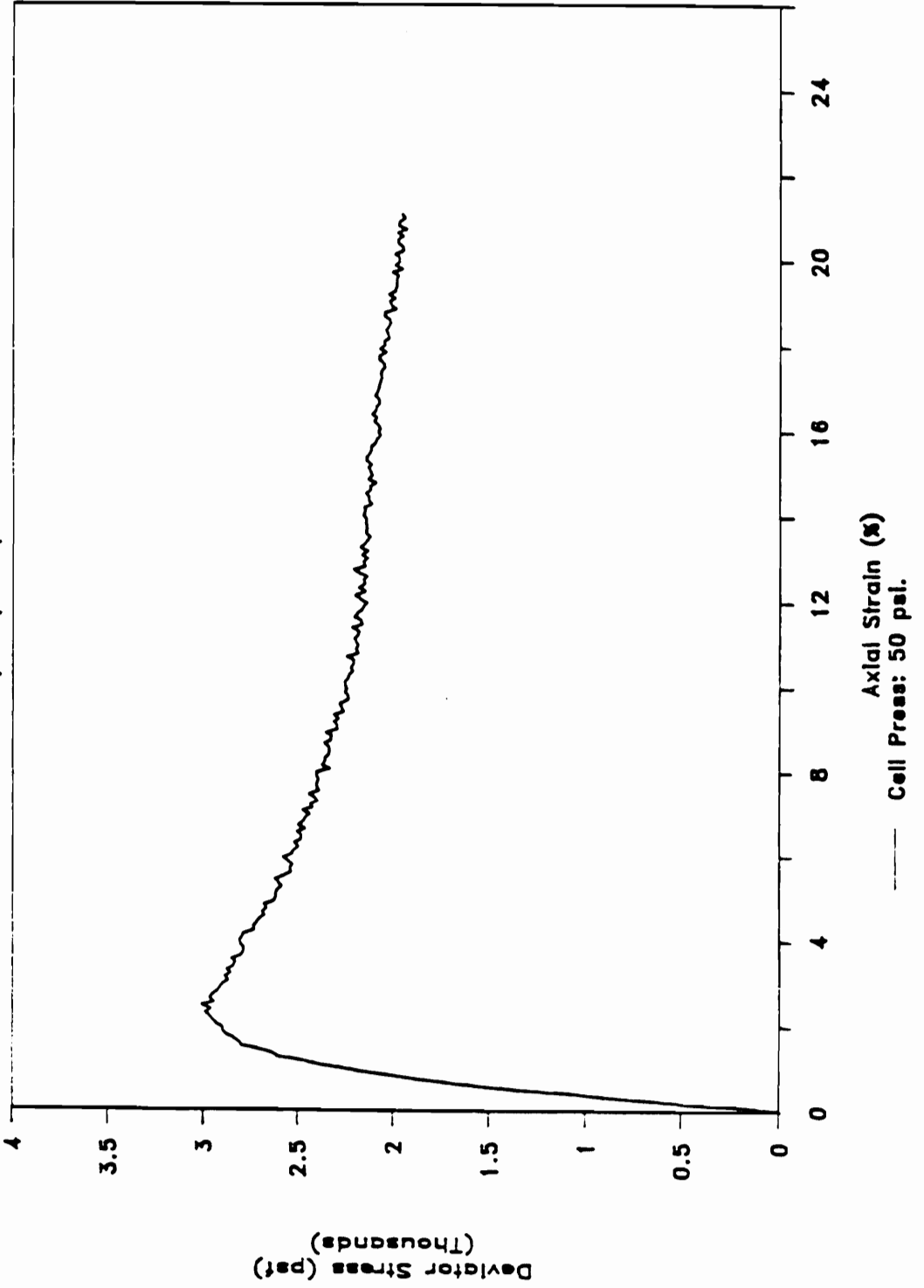
Location: Edisto Beach (SSP)

Soft Gray Clay: Depth 37.5 ft.



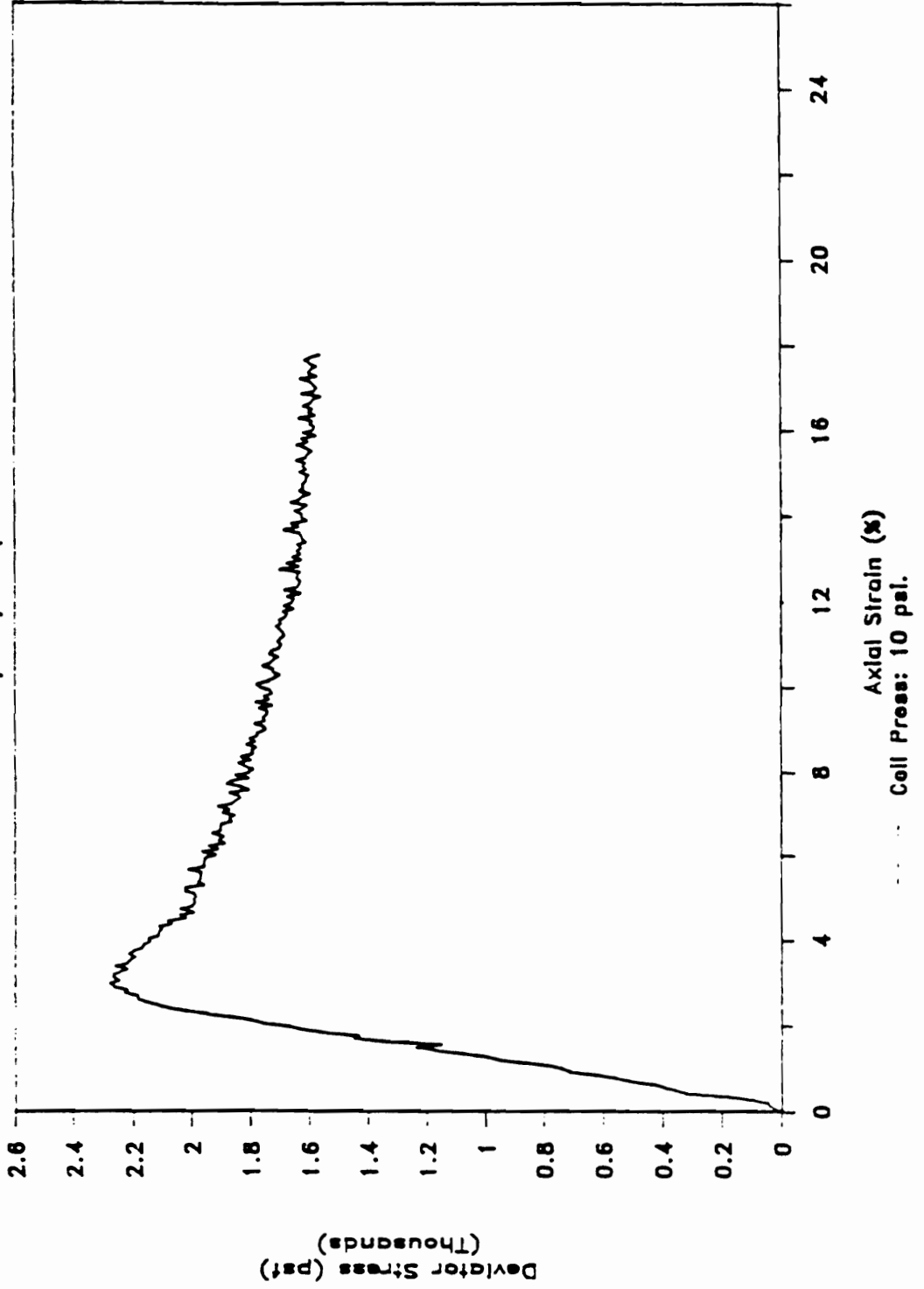
Location: Edisto Island - (SSP)

Soft Gray Clay - Depth: 36 ft.



Location: Mt. Pleasant Pits (MPPITS)

Soft Gray Clay - Depth: 24 ft.



APPENDIX D

METHODOLOGY USED IN COMPUTING THRESHOLD ACCELERATIONS

This appendix outlines the procedure used for determining the threshold accelerations required for ground disruptions at a site. The described procedure is introduced mainly for cases where the threshold acceleration level is thought to lie between those estimated from Seed's simplified procedure and those estimated from Ishihara's criteria. That is, in some cases, it is judged that liquefaction over the entire lateral and vertical liquefiable layer is not necessary to produce liquefaction features. Inasmuch as Ishihara's relationships are developed for cases where presumably the entire liquefiable layer is liquefied, an alternative procedure is presented here which uses both Ishihara's and Seed's criteria to make a refined estimate of the threshold accelerations. In the explanation of this procedure, the Hollywood site is used as an example.

The plot in Fig. D-1.1 shows what percentage of the samples (SPT and equivalent CPT resistance values) obtained from the liquefiable layer at the Hollywood site (cross-hatched zone) would be susceptible to liquefaction at different acceleration levels. The plot is developed for both magnitude 6 and 7.5 events. Examining this plot, one question that arises is: What is the implication of a certain percentage of the samples liquefying upon the development of liquefaction the site? It is reasonably clear that liquefaction of a high percentage of the samples from the liquefiable layer would indicate widespread liquefaction throughout the layer and result in ground damage at the surface. In other cases however, where only a low to moderate number of samples are shown to liquefy, it is unclear as to the actual pattern of liquefaction that would result in the layer. In the analyses in this report, the assumption is made that the percentage of samples that are shown to liquefy within a particular layer, corresponds to the percentage of that layer which is actually liquefied. This assumption is not entirely correct because of variability in the soil conditions from location to location and with depth. Thus, there is the possibility that liquefaction of a moderate percentage of the samples from within the liquefiable layer could occur in an aggregate sense, but remain confined to isolated pockets within the soil profile

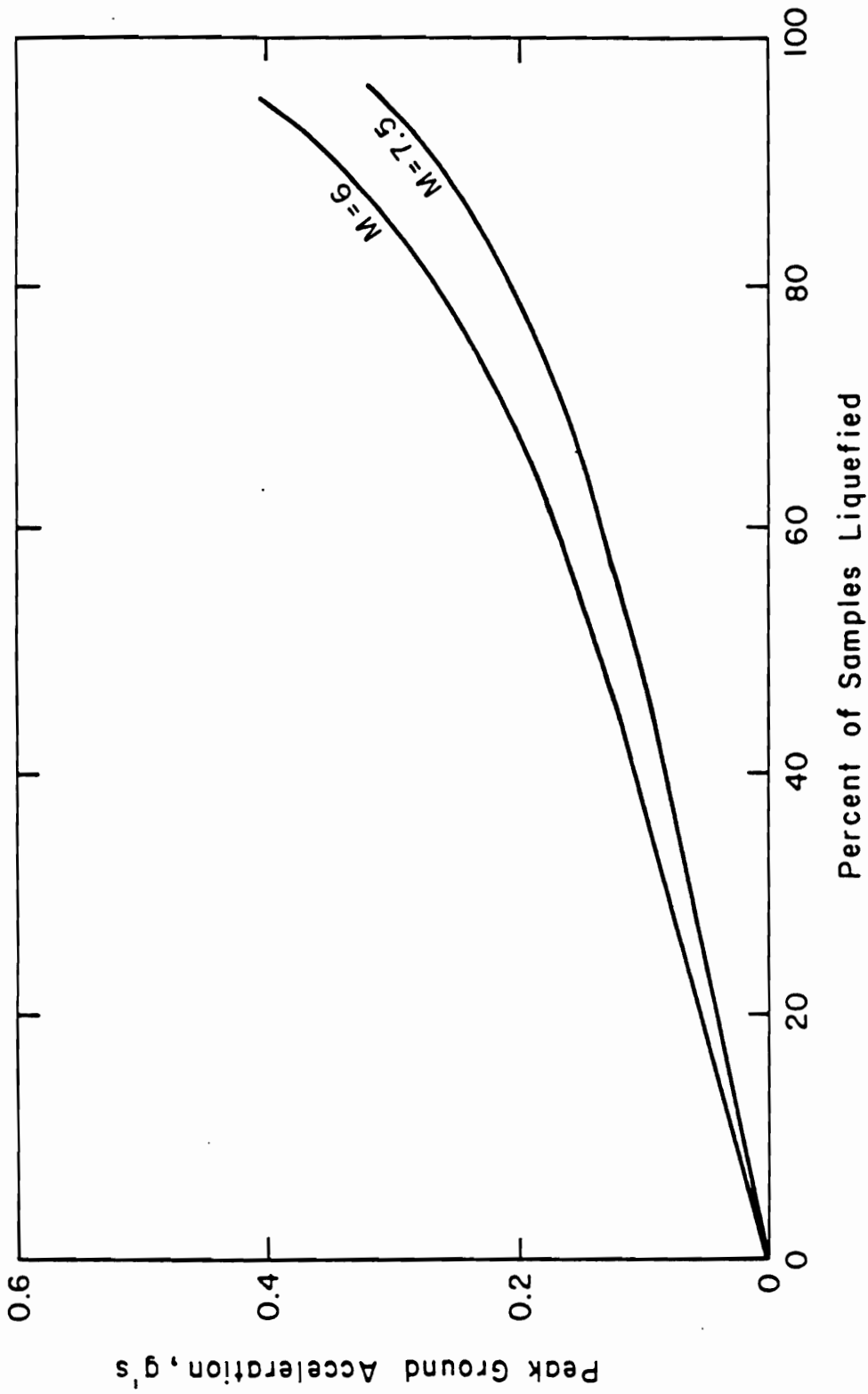


Figure D-1.1 Plot Showing Percent of Liquefaction-Prone CPT and SPT Resistance Values From Liquefiable Layer at Hollywood site for Various Acceleration Levels for $M = 6$ and 7.5 Earthquakes. Data Includes 43 CPT and SPT Samples with $Q_c/N = 4.5$.

or at selective locations across the site. In this case, it is more likely that no evidence for liquefaction would be observed at the ground surface. The higher the percentage of the samples within the liquefiable layer liquefied, the more likely liquefaction evidence would occur because the liquefaction would be more continuous across the site and throughout the profile. This is obviously also dependent upon how well the penetration tests define the soil conditions at the site, and how much variability is present within the soil profile. One way to assess these issues is to examine earthquake case histories where knowledge of soil conditions and field behavior are both well-known. One such case history is that of Clough and Chameau (1983) in their study of the liquefaction potential of the sand fills along the San Francisco waterfront. Their work involved an area about the size of the Hollywood site and with about the same variability in soil conditions. Examining penetration data from their field tests, it is seen that prior to the 1989 Loma Prieta earthquake ($M = 7$), only about 30% of the penetration samples obtained from the fills were susceptible to liquefaction, and these samples were generally obtained from near the bottom of the fills (which exaggerates the Ishihara layering effect). The performance during the 1989 Loma Prieta earthquake however, revealed that liquefaction did occur in these fills at an acceleration level of about

0.3g, and settlements and limited sand boils were observed. Thus it is possible that even for cases where a relatively low percentage of samples within the liquefiable layer are liquefied, liquefaction evidence can still occur at the ground surface. This is probably even more the case for environments such as the Charleston beach deposits where variability within the soil profile is generally low.

Using the Hollywood site as an example (see Fig. D-1.2), it can be shown from the Ishihara charts that minor ground disruptions should begin to develop when only about 5.5 ft. (or approximately 80%) of the cross-hatched layer (7.0 ft. thick) is liquefied. This was estimated using the Ishihara curves (Fig. D-1.3) with the thickness of the non-liquefiable overburden fixed at 7.0 ft. (the average overburden thickness across the site), and by varying the thickness of the underlying liquefied layer in 20-percent increments from 0 ft. to the full 7.0 ft. The threshold accelerations were computed for cases where the liquefied layer was: 0 ft. thick (0%); 1.4 ft. thick (20%); 2.8 ft. thick (40%); 4.1 ft. thick (60%); 5.6 ft. thick (80%); and, 7.0 ft. thick (100%). Each 20-percent increase in the liquefied layer thickness corresponds to a different (lower) threshold acceleration level required to produce disruption at the ground surface. The combination of these percentages and accelerations were superimposed onto Fig. D-1.1 and

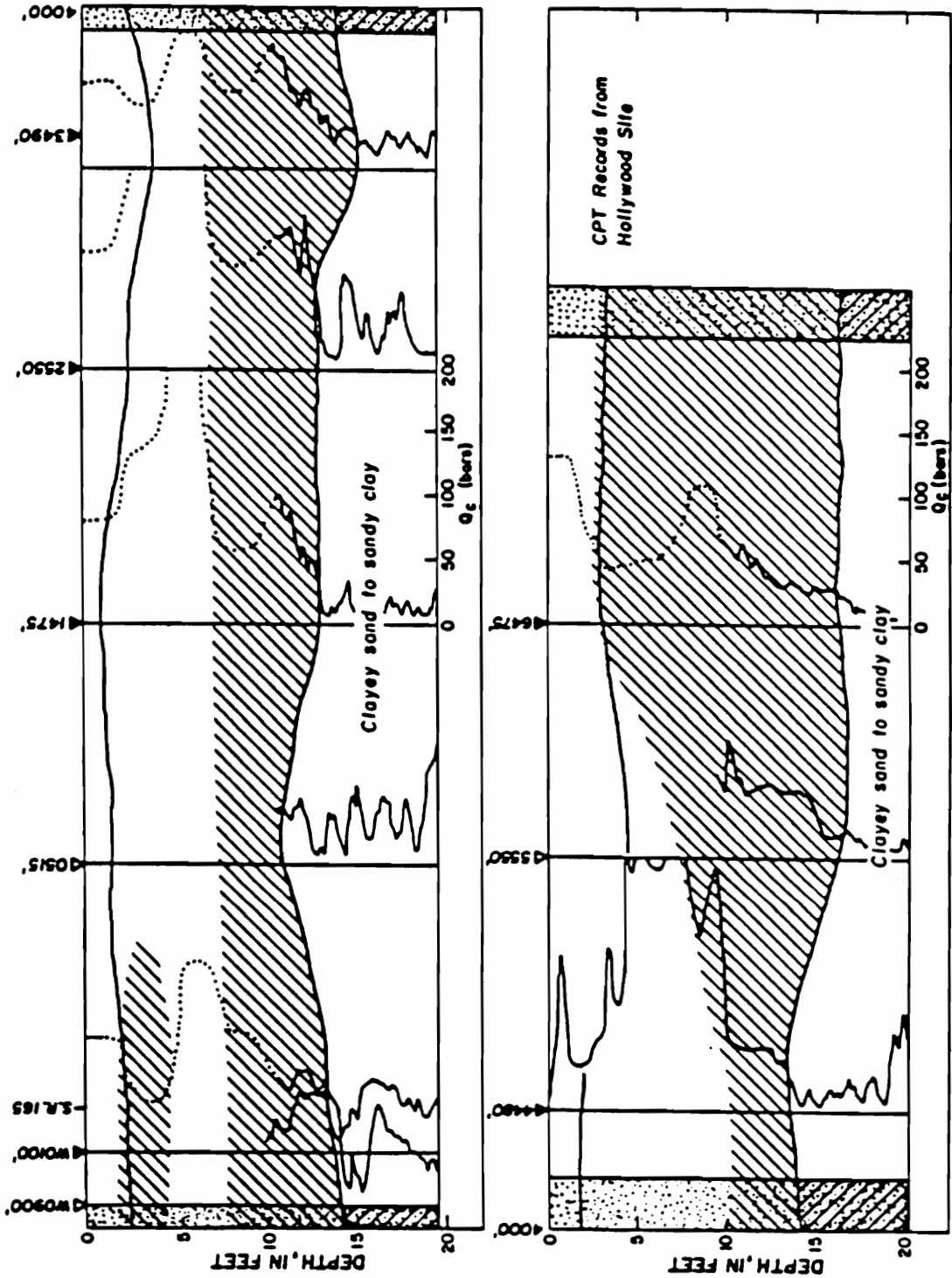


Figure D-1.2 Location of Sand Layers Susceptible to Liquefaction Under Seismic Loadings Estimated For the 1886 Earthquake.

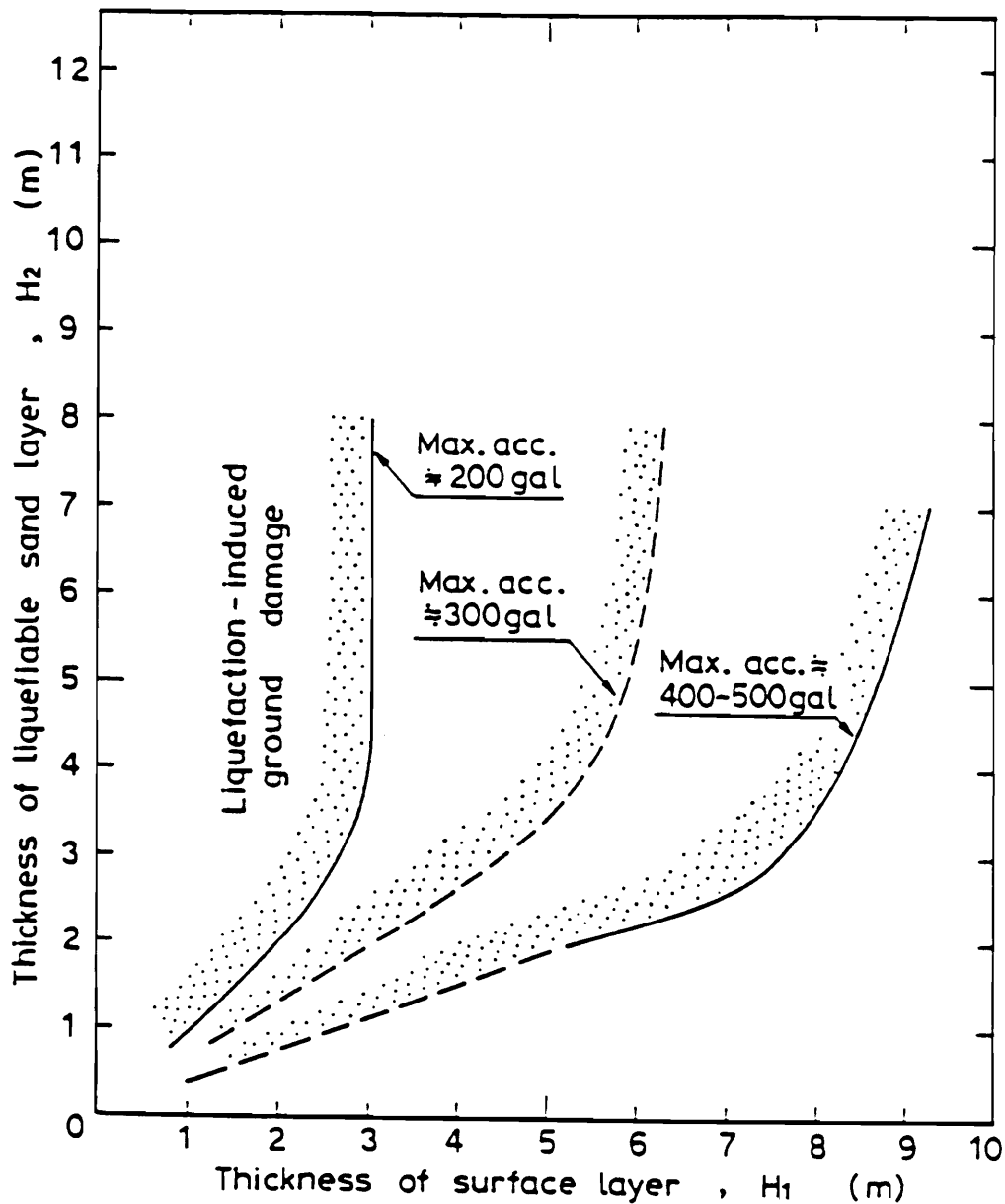


Figure D-1.3 Boundary Curves for Site Identification of Liquefaction-Induced Ground Damage (Ishihara, 1985).

shown as the "layer effect" curve in Fig. D-1.4. Note that the abscissa of Fig D-1.1 is labelled as the "percentage of samples liquefied", while the percentages from the Ishihara analyses are in terms of the percentage of the layer liquefied. It is here that it was assumed that the percentage of liquefied samples from within a layer is roughly equivalent to the percentage of that layer that is liquefied. At the Hollywood site for instance, if say 60% of the samples from within the liquefiable layer are liquefied, it is assumed this would mean that about 4.2 ft. (60% of 7.0 ft.) of the 7 ft. layer would be liquefied. This is reflected by the combined labelling of the abscissa (samples/layer) in Fig. D-1.4. Examining Fig. D-1.4, it can be seen that the layer effect curve trends opposite to the $M = 6$ and $M = 7.5$ acceleration curves, and the crossover point where the curves intersect (with respect to the ordinate) is presumably the threshold acceleration values above which ground disruption would occur. The figure shows that using this technique, ground disruption is predicted to occur at the Hollywood site at 0.24g for an $M = 7.5$ event and at 0.28g for an $M = 6$ event. At the 0.24g acceleration, about 85% of the samples would be liquefied, and at 0.28g, approximately 80% of the samples would be liquefied. Because the cross-over points correspond to situations where large percentages (say > 50%) of the samples are liquefied,

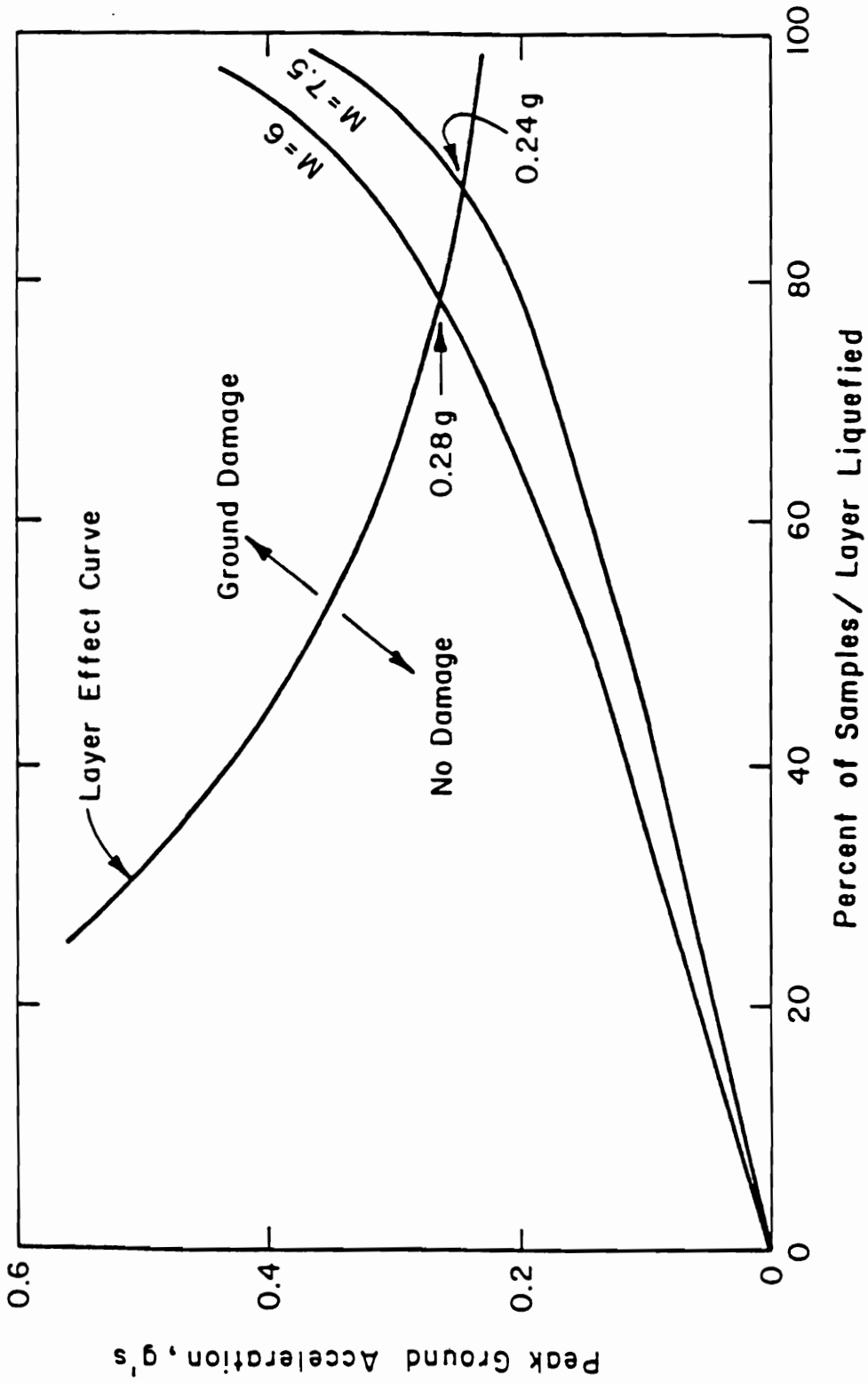


Figure D-1.4 Refined Estimate of Threshold Accelerations Required to Cause Ground Disruption at Hollywood Site Based on Percent of Samples Liquefied and Relative Thicknesses of Liquefiable and Non-Liquefiable Layers (43 CPT & SPT Samples).

it is judged that these accelerations would in fact cause the liquefaction to be detected at the ground surface. The steps involved in using this procedure are summarized:

- 1) The accelerations based on Seed's criteria for liquefaction within the liquefiable layer are determined;
- 2) The accelerations required to produce significant ground disruption at the surface are determined from Ishihara's criteria. In most cases, the accelerations obtained from the Ishihara criteria are higher than those necessary to cause significant liquefaction within the layer.
- 3) At some sites, where the evidence is clear that only minimal liquefaction occurred there in 1886, the actual 1886 acceleration level is thought to lie between those indicated by the two approaches. In these cases, the estimated accelerations for the 1886 event are refined by using plots such as that in Fig. D-1.4. The threshold acceleration is estimated as the point where the layer-effect curve intersects the $M = 7.5$ and $M = 6$ acceleration curves. The threshold accelerations obtained using this procedure are considered less credible in cases where less than 50% of the soil profile is liquefied at the cross-over point. In this case, the assumption that the percentage of the samples liquefied equals the percentage of the layer liquefied

may be misleading in terms of predicting whether liquefaction will occur at the ground surface. This factor was considered in estimating the probable levels of 1886 ground accelerations such as those listed in Table 6.2 and those used in the construction of the attenuation curves in Figs. 6.22 and 6.23.

The threshold acceleration values obtained using the procedures described thus far are further tempered by the following considerations:

- 1) At most sites, the accelerations are estimated based on the average soil conditions across the site, and the actual field behavior at specific locations may be different from that indicated by the average conditions, especially for cases where a low percentage of the samples from within a layer is liquefied.
- 2) Also, the estimated threshold accelerations vary depending upon whether it is assumed that liquefaction within the cross-hatched zone is initiated near the top of the zone or toward the bottom. This is because for cases where less than 100% of the liquefiable zone is liquefied, the thickness of the non-liquefiable overburden would be different in these two extremes. For example, at the Hollywood site, if it is assumed that the liquefaction is initiated near the bottom of the cross-hatched region, the non-liquefiable overburden

thickness would be greater than if it were assumed the liquefaction occurred in the upper part of the profile. Assuming liquefaction is concentrated at the bottom of the layer, the thickness of the non-liquefiable layer is equal to the average non-liquefiable overburden thickness plus the thickness of the overlying portion of the cross-hatched layer that did not liquefy. This would mean that in this situation, the accelerations predicted to cause surficial ground disruptions would be greater than those predicted for the case where the liquefied portion of the layer is closer to the ground surface. The liquefaction analyses performed in this report are for the case where it is assumed that the liquefied zone develops immediately below the non-liquefiable overburden layer, near the top of the profile. This is based on the fact that the paleoseismic evidence supports the idea that liquefaction in the Charleston region in 1886 was generally initiated at shallow depths within the profile.

- 3) The Ishihara relations (as well as those proposed by Seed) are not exact, and some subjectivity is required in their interpretation and use.

VITA

James R. Martin, II was born on July 27, 1963, in Spartanburg, South Carolina. He grew up in Cross Keys, South Carolina, a small farming community situated in the foothills of the Appalachian Mountains. There he enjoyed a fruitful childhood while developing a strong love and appreciation for land and nature. After graduating from Union High School in 1981, he attended The Citadel, The Military College of South Carolina, where he received his Bachelor's of Science degree in Civil Engineering in 1985. He then began graduate studies at Virginia Tech, where, in September of 1987, he received his Master's of Science degree in Civil Engineering. He completed requirements for the Doctor of Philosophy degree in Civil Engineering in November of 1990 at which time he was appointed to the rank of Assistant Professor of Civil Engineering at Virginia Tech, working in the Geotechnical Engineering Division. At present, he resides at an undisclosed location in the mountains of Pembroke, Virginia, where, with casual elegance, he perfects the art of gentleman farming.

