DESIGN OF CEMENT STABILIZED SOIL RETAINING WALLS WITH CONCRETE PANEL FACING

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	2.54	centimetres	cm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	centimetres squared	cm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

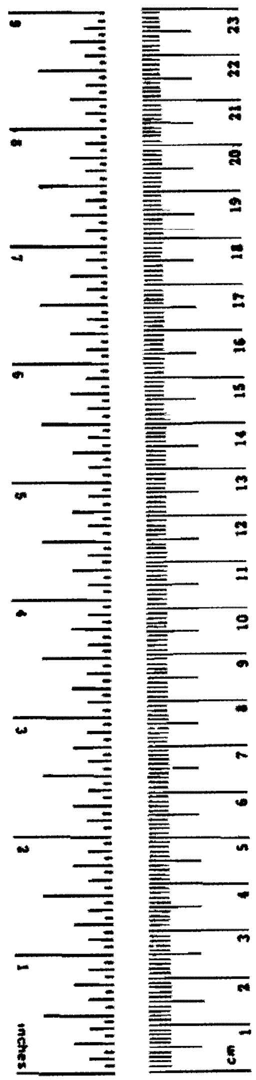
VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac

MASS (weight)

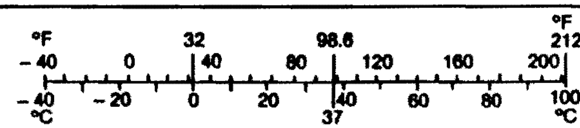
Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

ABSTRACT

In order to minimize the cost of earth retaining structures, particularly for the low to medium height retaining walls required by highway departments, a new design of retaining wall was investigated. This utilized facing panel units anchored into a cement stabilized compacted fill. Only short anchors are required to retain the facing panels, as mechanical stabilization of the cross-section is achieved by the addition of about 7% cement, rather than by reinforcing strips.

The feasibility and applicability of this design has now been demonstrated, on the basis of this study which included laboratory testing, numerical and physical modeling, and full scale construction and instrumentation of two experimental retaining walls. Specifications have been prepared and these have been refined on the basis of full-scale experience.

This design (which is non-proprietary) may be used in situations where proprietary designs are used nowadays, although there are significant differences in construction procedure from other mechanically stabilized earth designs. It is suitable for general non-specialized use, without requiring specialist subcontractors, and there should be some cost advantage, particularly for locations where cement is inexpensive. Unit cost was around $20 per square foot for the experimental wall, and would be reduced with wider acceptance. The main disadvantage appears to be that large differential settlements from compressible foundation soils cannot easily be accommodated.

Key words: Retaining Walls, Soil Cement, Earth Retaining Structures, Tiebacks, Retained Earth, Reinforced Earth

SUMMARY STATEMENT ON RESEARCH IMPLEMENTATION

In order to minimize the cost of soil retaining structures to highway departments, a new design proposed by the Texas Department of Transportation, was investigated and found to be viable.

Verification of the design concepts and field performance, will enable the use of this retaining wall design as a proven low-cost alternative to proprietary and non-proprietary designs currently being specified by the department.

The Texas Transportation Institute is coordinating with department personnel in implementation of the research results, to ensure that these will be relevant to highway department practice. The results of this study have shown that suitable allowance should be made in using this design on very compressible foundation soils. The long-term performance of this design is still being monitored, to address, among other things, long-term corrosion resistance.

Satisfactory utilization of this method of retaining wall construction, may enable major economies to be realized whenever earth retaining structures are required. The design is expected to be particularly inexpensive in situations where stabilizing material (such as cement) is cheaply available. Furthermore, any contractor can bid on construction of such a wall without payment of royalties or engaging specialist subcontractors for proprietary work.

DISCLAIMER

SYNOPSIS

To investigate the feasibility of a new design of retaining wall, a comprehensive study was undertaken by the Texas Transportation Institute in conjunction with the Texas Department of Transportation. The concept was to use precast concrete facing units fastened to a cement stabilized soil cross-section with short anchors. Rather than using reinforcing strips or bars or fabric in the backfill, the strength of the soil is improved only by the addition of sufficient cement. This is designed to result in a conventional mass gravity structure, of substantially lower cost than a mass concrete wall or even a reinforced earth wall. Other potential advantages in construction are that large lifts of fill can easily be compacted to the required densities, the likelihood of soil wash-out between units is reduced, and trafficability of the compacted fill is improved.

An initial information review produced significant and encouraging data on the properties of stabilized soil, including difficult materials, marginal soils, mixing procedures, curing requirements, additives, strength and failure characteristics, etc. A privately constructed case history, utilizing similar design principles,was also identified in California, constructed to 32 ft. at a 1 H to 8 V batter. An interlocking concrete block facing was used, mechanically linked (without anchors) into a 5% cement stabilized sand backfill. Unit cost in 1985 was $17 per square foot of wall.

To obtain further information on the three dimensional strength properties of stabilized fill, a program of laboratory strength testing was carried out on cement stabilized soil, using both conventional sand and lightweight materials. In addition to conventional cylinder testing, Texas triaxial testing was utilized to provide stress strain characteristics at different confining pressures, as well as values of cohesion and internal friction. The results indicated general suitability, with unconfined compressive strengths in the range of 150 to 500 psi (1 to 4 MPa) for 3.5 to 7% cement by weight (1 to 2 sacks per cubic yard) for typical degrees of compaction.

The capacity of anchors in stabilized fill was evaluated by conducting pull-out tests in accordance with ASTM standard C900-82 for concrete anchors. The capacity in stabilized soil turned out to be comparable to the capacity of similar anchors in weak concrete, once due allowance was made for the lower strength, so it appears that similar design formulae can be applied to stabilized soil.

Before construction of a full-scale experimental section, finite element analyses of the wall cross-section were performed for several geometries and load cases, using the program ABAQUS. Non-linear stress-strain relations and three-dimensional failure criteria were incorporated. Critical points in the cross-section were found to be yielding of soil beneath the toe of the wall. However, analysis indicated that behavior would be acceptable if wall height was limited to keep self-weight wall stresses well below compressive strengths, and if bearing pressures on foundation soil was adequately addressed. Large stress concentrations were observed (of the order of 2.5 to 3) within the outer toe of all cross-sections, so a minimum factor of safety of 5 was recommended against crushing with respect to overburden stress within the wall.

At the same time, some physical modelling was also carried out by conducting some centrifuge model tests at 1/60th scale, using model wall units and miniature instrumentation. In three cases failure of the cross-section was observed by crushing (two tests) and by toppling (one test), but this only occurred for extreme load conditions. For typical wall geometries and properties, measured movements were small (seldom exceeding a scaled value of a few inches), and earth pressures were as indicated by standard theory. Forces in the anchors were also small, typically of the order of 200 lbs (1 kN) full scale equivalent. These experiments tested mainly the wall cross-section, as the model subsoil was prepared cohesionless soil, and under these conditions, the observed behavior was generally stable.

A field site for a full-scale experimental section was identified as the Cypress/Fairbanks bypass of Highway 290, north-west of Houston. In conjunction with District 12, two of the 10 retaining walls at this site (walls 7 and 9 of heights 16 and 23 feet respectively) were designed as experimental sections, together with draft material specifications for the stabilized fill. The construction contract was let in 1989 for just under 2.3 million dollars for the 102, 173 square feet of total retaining wall, with T.T.I. to supply the instrumentation at the experimental sections.

Construction proceeded initially at several locations, including wall 7, until on October 31, 1989 a major rainstorm brought work to a halt on the entire site. Subsequently significant distress was noted at a number of locations, but particularily at the experimental wall, where some facing units displayed significant separation from the stabilized backfill. Substantial settlement of the wall levelling pad was noticed. The stabilized fill close to wall face appeared to be very weak, and this was later confirmed by laboratory testing that indicated cement content was slightly low, and the compressive strengths were much lower than specified.

The prime reason for the distress, appeared to be massive settlement of a foot or more, of backfill over an underground storm drainage culvert. At several locations, this was located in close proximity to the toe of the retaining walls (including the conventional VSL Retained Earth Walls), resulting in a noticeable outward lean of several sections. The true sub-surface conditions were then investigated as accurately as possible, with a special program of cone penetrometer testing in order to allow continuous measurement of soil strengths very close to the wall face. This confirmed very weak localized soil (shear strengths 600 psf (30 kPa.) or less). Subsequently, all the wall units assembled at that time (both conventional and the one experimental wall) were dismantled, in preparation for foundation improvement.

Reworking of the foundation included widening of the sand and stabilized sand layer under the walls, and localized remedial piling under wall 2, 4, 5, 7 and 8 under District 12 supervision in 1990. Re-construction of the retaining walls at the Cypress-Fairbanks site, took place during late 1990 & 1991, and field instrumentation was installed at the two experimental sections, wall 7 and 9. Wall 9 had not been built prior to the earlier field problems, and (after appropriate modifications to the foundation design but without any remedial piling) was successfully constructed to the design height of 23 feet, incorporating vertical and horizontal inclinometers, force gauges on some anchors, and earth pressure cells.

The results of instrumentation readings indicate some continuing settlement, but this has so far been primarily uniform. Differential settlements as measured by horizontal inclinometers have been no greater than 0.1 inches (2-5 mm) and overall absolute settlements from survey data have so far been of the order of 1 inch (25 mm.). Soil stresses and anchor forces are largely as expected, at least in the absence of traffic loading. The fill in the body of the material is settling more than at the face of the wall, as was originally indicated by analysis, but this is not causing any problems so far. At present, the visual performance of both the experimental sections, and of the re-constructed conventional sections, has been acceptable.

Long-term data will be available from a separate monitoring study of the site.

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1. INTRODUCTION

The need for economical methods of retaining earth fill has resulted in many alternative designs of retaining structure, even of the traditional gravity type walls. Mass concrete walls have long been superseded by designs that make more efficient use of the retained earth fill, to assist in providing overall stability.

A common commercial example uses galvanized steel straps and select backfill to form the retaining wall mass behind a precast concrete facing. The overall strength of the fill is provided by friction between the soil and the reinforcement.

A new and potentially even more economical design has been proposed by the TxDOT using facing panel units anchored into a cement stabilized backfill of sufficient strength to avoid the need for any soil reinforcement at all.

Overall stability is provided by the self-weight of the stabilized soil, artificially given an intact strength by the relatively inexpensive addition of between 4 to 8% cement by mass. Moreover the design is non-proprietary, not subject to any licensing restrictions.

This report describes the studies that were done, in order to investigate the technical feasibility of the design.

2. INITIAL REVIEW

2.1 Properties of Cement Stabilized Soil

The material, mix proportions, construction methods, and environmental conditions all have an influence on the properties of cement treated soil. These factors can be divided into the nature of materials and the proportions of the mix; mixing and compaction; and conditions of curing.

Nature of Materials and Proportions of Mix:

As a result of several studies the following conclusions can be drawn:-

The properties of compacted and hydrated soil-cement mixtures are dependent to a great extent upon the types of soils involved. The influence of the nature of soil is also indicated by the ranges of cement requirements to produce soil-cement for the various AASHTO and Unified Soil System soil groups. Figure 2.1 shows the variation in strength for 4 different types of soils at various cement contents and age.

Compressive strength of soil-cement is not appreciably affected by the material retained on a No. 4 sieve, unless the proportion is greater than 50 percent by weight of the total material.

Reinhold (HRB Bull. 108,) concluded that as the clay content increases, modulus of elasticity in compression decreases. Another study in Iowa (Handy et al., HRB Bull. 108) showed that increasing the clay content increased the percentage of cement required to produce soil-cement. Properties of cement-treated soil are affected by the plasticity of the soil. However, there are no well-defined relationships between plasticity and the nature of soil-cement for soils of A-2 and A-3 groups. For soils of the A-4 group, there is a noticeable trend of increasing cement requirement with increases in the liquid limit (Catton, HRB Bull. 20). The trend is more marked for soils of A-6 and A-7 groups.

The constituents of soils include substances that react with cement to varying degrees. Normally reacting soils may differ in degree of reaction depending on the nature of cations associated with clay-sized minerals. Soils that do not react normally

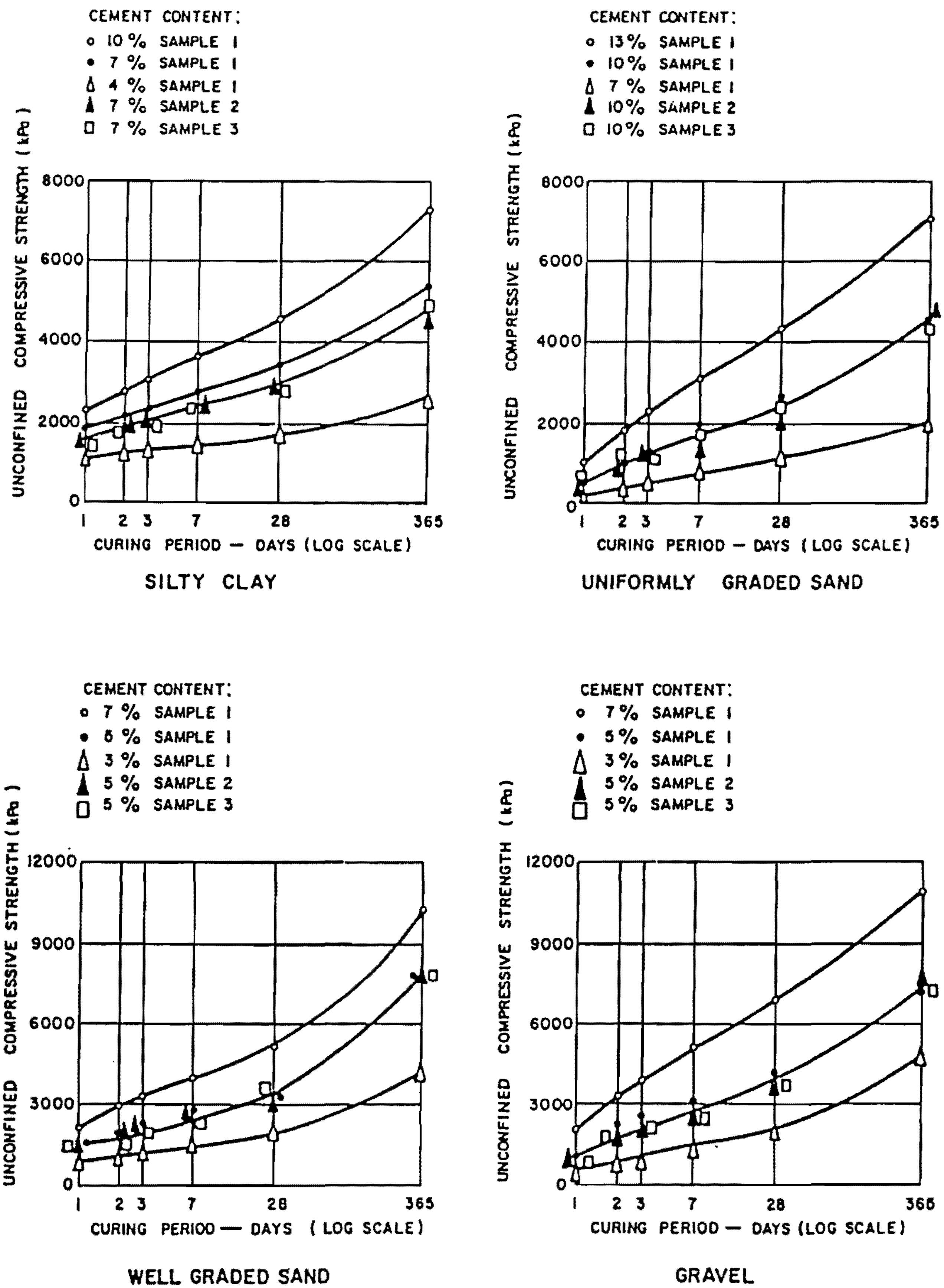


Figure 2.1 Strength variation with soil type

with cement may owe that property to the presence of organic matter that causes delayed setting or to the presence of sulfates that cause swelling and/or reduction in strength in the presence of water.

Catton (HRB Bull. 20) also showed that the pH of the soil did not influence the cement content of the soil, i.e. a soil can be either acid, neutral, or alkaline. It will respond in equal degree to cement when the pH value is the only variable involved. Robins and Mueller (1960) tried to develop correlations between pH, organic content and 7-day compressive strength, but a definite relationship was not obtained. The results of the correlation showed a trend of decreasing strengths with increasing organic content, although some very low strengths were obtained on soils having organic content as low as 2000 ppm.

Robins and Mueller (HRB Bull. 267) developed a calcium absorption test to identify poorly reacting sand. During this study, it was found that organic content and pH do not in themselves constitute an indication of poorly reacting sand. However, a sandy soil with a organic content greater than 20,000 ppm or having a pH lower than 5.3 would in all probability not react normally with cement. The calcium absorption test provides a method of quickly determining either in the laboratory or in the field whether or not a sandy soil will react normally with cement in soil-cement construction. Accordingly, a soil having a calcium adsorption factor less than 11 ml may be considered normal; those with factors greater than 11 ml, as reacting poorly.

The moisture content of a soil-cement mixture and the density to which the material is compacted have a great influence upon the quality of the product after cement hydration. Felt (HRB Bull. 108) studied the effect of variations in the moisture content and density on compressive strength and on losses in wet-dry and freeze-thaw tests. His results are summarized as follows:

> Compressive strength increases to a maximum at slightly less than optimum moisture for the sandy soil and the silty soil and at greater than optimum for the clay soil.

Results from wet-dry and freeze-thaw tests showed that clayey soil had less resistance at moisture contents below optimum and the silty soil had less resistance in the freeze-thaw test at moisture contents less than optimum. Moisture contents were not so critical in either tests for the sandy soil.

The difference between optimum moisture contents for both maximum density and maximum strength may be attributed to the surface area of the soil particles. Sands have low surface area, hence most of the water added is available for the hydration of cement. Since this water is more than that required for the hydration, the strength is therefore reduced. In clays, it is just the opposite. Clays have small particle sizes and also have high affinity for water. Therefore, water available for hydration of cement may be less than that which is required with the result that maximum strength occurs on the wet side of optimum.

The amount of moisture needed for maximum strength also depends on the clay minerals present. For the sand-kaolinite clay mixtures, maximum strength is always on the dry side of optimum. For sand-illite and sand-montmorillonite clay mixtures, the optimum moisture content for maximum strength shifts to the wet side of optimum moisture for maximum density (Davidson, 1962). As the cement content of a given sand-clay mixture increases, the optimum moisture for maximum density and the optimum moisture for maximum strength does not change much. This suggests that water necessary for hydration of cement may be small in comparison with the amount needed to obtain maximum density.

For a given soil that reacts normally with cement, the cement content determines the nature of cement-treated soil. The compressive strength and resistance to wetting and drying and freezing and thawing increases as cement content is increased (Felt, HRB Bull. No. 108).

Soil-cement mixtures containing air entraining cement behave approximately the same as those containing type I cement. Moisture density relations, compressive strengths, wet-dry and freeze-thaw test results are sufficiently similar to show that the cements may be used interchangeably in soil-cement construction.

The moisture content at the time of performing a strength test has a great influence on the compressive strength of cement-treated soil. One source of data shows that for a nonplastic sandy loam soil (A-1-b) containing 3 and 10 percent cement, the dry strength averaged 180 percent of the strength of the moist specimens; for an A-6 sandy clay soil (PI-15) dry compressive strengths averaged 245 percent of that of the moist specimens (Felt and Abrams, 1957).

A second source of data on four soils shows that strengths of dry specimens ranged from 190 to 290 percent of the strength of moist specimens (Chadda, 1956). This shows that in the dry state, the strength of soil-cement is made up of the cohesive effect of soil and the cementing action of cement.

Mixing and Compaction

The efficiency of mixing and compacting equipment and the time required, influences both the strength and durability of cement-treated soil.

In a study conducted at Louisiana State University (Arman and Saifan, 1965) regarding the effects of delayed compaction on stabilized soil-cement, the following conclusions were drawn:

1. The durability, compressive strength, and density of soil-cement mixes compacted at, below, or above their optimum cement and moisture contents decreases considerably and uniformly after a delay of two hours or more in compaction of the soil-cement after mixing.

2. The loss of compressive strength, durability and density due to delayed compaction can become so great in intensity that any physical improvements to be derived from the addition of Portland cement to the mix are nullified.

3. An increase in cement content above the optimum does not necessarily improve the quality of soil-cement.

4. The effect of delayed compaction is directly dependent on the final setting time of the cement used. Compaction in soil-cement mixes should not be delayed

beyond the initial setting time of the cement gel. It was also recommended that a factor of 0.80 be used as a multiplier of the initial setting time to determine the maximum allowable period of delay in compaction.

Numerous tests have shown that field strengths of soil-cement seldom match laboratory strengths (Robinson, 1952). This may be due to insufficient mixing. Robinson used the ratio of field strength to laboratory strength as a measure of field mixing efficiency. He found varying value of mixing efficiency depending on the mixer type and soil type, but a value of 60 percent is typical. He further noted that if the mixing efficiency could be increased to 80 percent, 30 percent less cement might be used for the same strength.

Baker (HRB Bull. 98) studied the effect of degree of mixing on the strength of soil-cement and concluded that the strength of soil-cement varies as the log of mixing uniformity. An increase in the uniformity of the mix results in an increase in strength. Further, the rate at which the strength of the soil-cement increases with increasing uniformity of the mixture is greater for a soil mixed at slightly above the optimum. However, he also realized that to get a small increase in uniformity, the amount of energy required was greater than the increase in strength obtained. Therefore, he recommended development of more efficient mixing equipment.

Elrawi (1968) studied the effect of method of compaction on soil-cement mixtures and concluded that:

1. The method of compaction influences the cohesion and therefore the strength of soil-cement mixtures.

2. For granular soil-cement mixtures, the specimens molded by impact compaction give higher cohesion than the corresponding specimens molded by kneading compaction.

3. For silt soil-cement mixtures, specimens molded at optimum moisture content by kneading compaction give higher cohesion than the corresponding specimens molded by impact compaction at 7-days. Wet soil-cement at optimum kneading compaction gave lower cohesion than impact compaction.

4. For all practical purposes, friction angle values were not influenced by the method of compaction, cement content or age.

Curing Conditions and Age

Data on moisture retention by soil-cement under bituminous seals (HRB Bull. 292) indicates their success in retaining the required moisture content. Further data from experiments in Virginia (Maner, HRB Bull. 31) yielded results that permit comparison of the effectiveness of moist soil, waterproof paper, calcium chloride, RC-2 asphalt, tar and asphalt emulsion as curing materials for soil-cement. The three types of bituminous cover, the moist soil cover and the waterproof paper were the most effective aids to retention of moisture in the soil-cement.

British researchers bring out the following facts on the influence of temperature on the strength of soil-cement mixtures (HRB Bull. No. 292).

1. The 7-day compressive strength increases with increasing temperature by 2 to 2-1/2 percent per degree centigrade when temperature is in the vicinity of 25°C.

2. Soil-cement will harden in cold weather provided the temperature is over 0°C.

3. If the compressive strength is taken as the sole criterion for the quality of soil-cement, less cement is needed in warm weather than in cold weather.

4. Because of the ambient temperature differences, soil-cement constructed during warm weather should be 50 to 100 percent stronger than similar construction made during cool weather, at least during the first three months of life of the construction.

Just like concrete, the strength of a soil-cement mixture also increases with age. Abboud (1973) indicates that the 28-day strength may be taken as 1.4 times the 7-day strength. Other values are shown in Figure 2.2.

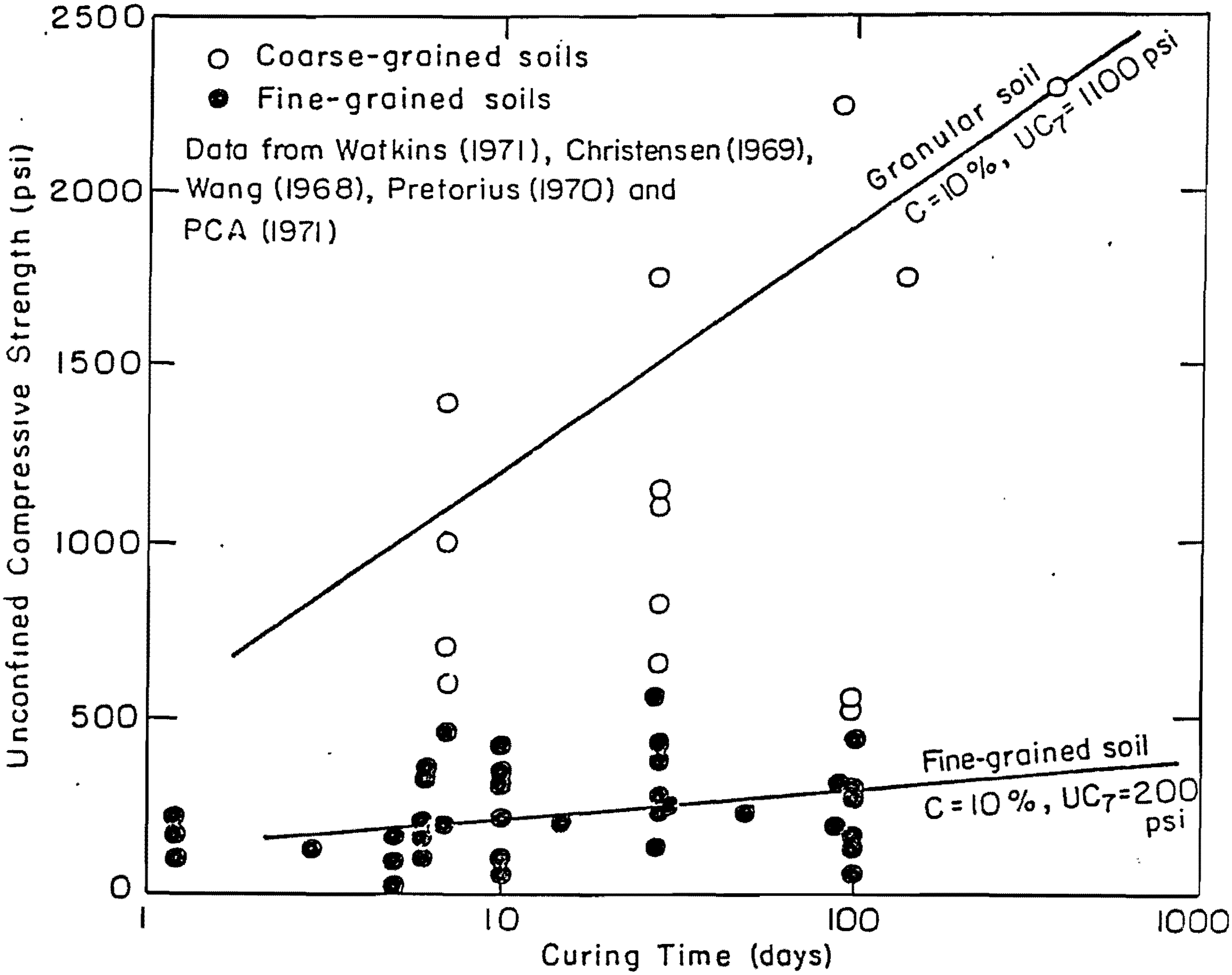


Figure 2.2 Strength variation with curing time

2.2 Full Scale Information

Following a fact-finding meeting with Mr. Ken Jackura of CALTRANS (Office of Transportation Laboratory, 5900 Fulsome Blvd., Sacramento, California, 95819-19128, (916) 739-2364) a case history was identified of a cement stabilized wall in California.

This was constructed in Laguna Beach, California for a private developer (Hoffmann Brothers) in 1985 for a cost of $\$17/ft^2$. This compares with costs of \$25 to $\$30/ft^2$ of "soft wall", (e.g. Reinforced Earth) in California and $\$27/ft^2$ in Texas. Photographs of the wall, which had a maximum height of 32 ft. are shown in Figures 2.3 to 2.6. The design used a proprietary interlocking concrete block facing, in this particular case a German product "Earthstone", without any separate tie backs. Cementitious bond between the facing blocks and the stabilized backfill (which was carefully compacted manually in this region) was sufficient to retain the wall facing. In addition, the wall was constructed with a slight backward slope of 1 horizontal to 8 vertical (7°). Some terraces were incorporated into the side sections, but not in the main body of the wall.

The engineer was E. James Miller of 2R Engineering, Inc., 244 Pico Avenue, San Marcos, California 92069, (619) 744-4440 who has subsequently been responsible for the design and construction of some smaller similar retaining walls (constructed vertically in some instances), although the wall described above remains the tallest example. His practice is to use stabilized backfill with a minimum cement content of 4% by weight and at least 200 psi (1.4 Mpa) cylinder crushing strength. Sandy soil is preferred (which often gives 500 to 600 psi strength (4 MPa), but some fines content has also been permitted. Their preference is to limit the overburden stresses in the wall to 1/6 of the crushing strength, treating the toe as the critical point, and allowing for any eccentricity of the wall center-of-gravity by simple elastic theory. In other respects, the design is like a conventional gravity wall.

As well as sloping the front face backward, they also recommend a rearward slope on the rear face of the wall (typically 1 horizontal to 4 vertical or 15°), which can often be incorporated into construction against original earth, by using the cut slope as a rear form and compacting stabilized fill directly against it. This has the advantage of moving the wall center of gravity rearwards, so that the resulting eccentricity of the self-weight can to a large extent neutralize the overturning moments on the wall due to soil pres-

Figure 2.3 Compaction of cement stabilized fill

Figure 2.4 Initial construction

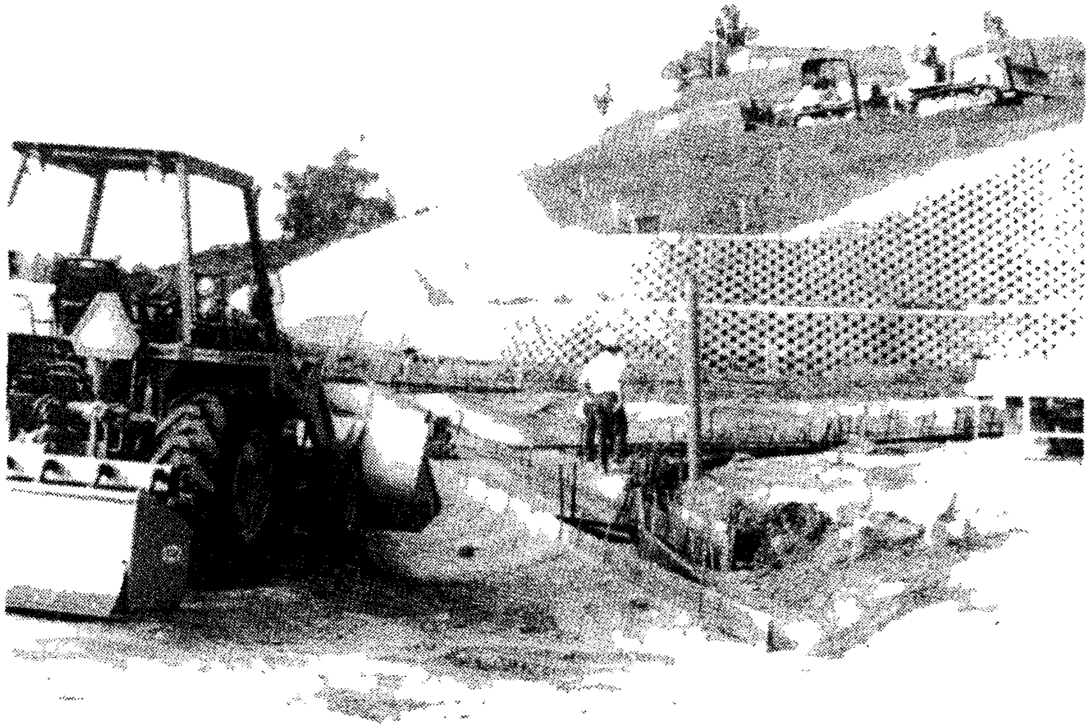


Figure 2.5 Terracing of earthstone block

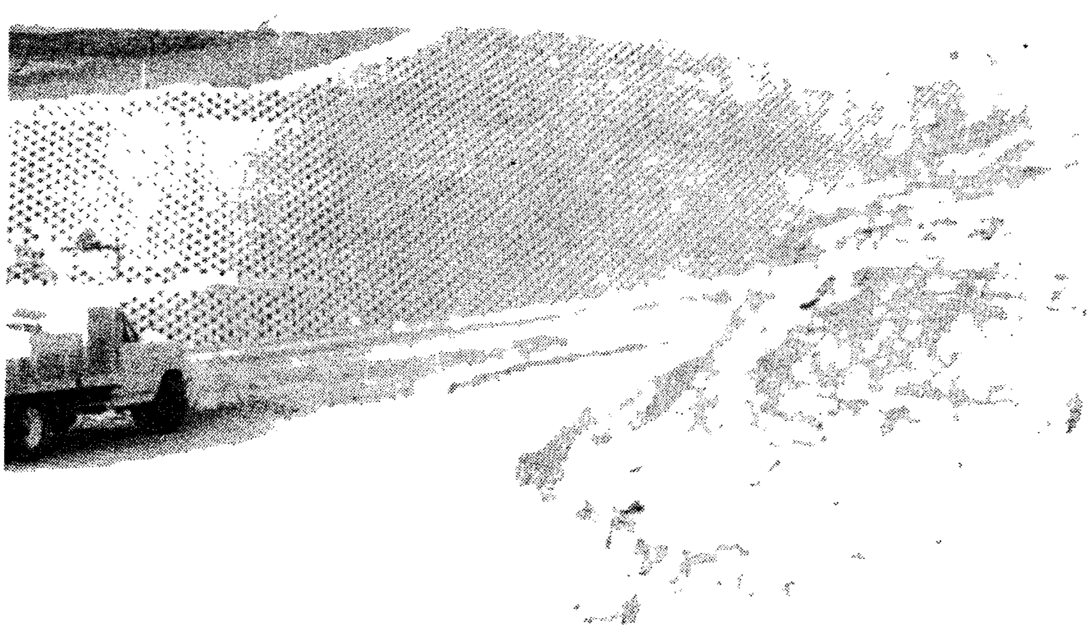


Figure 2.6 Fully completed 32 foot high wall

sures, and relieve high stresses at the toe of the wall. It is also possible to terrace the rear face of the wall, thus reducing the thickness of the stabilized fill towards the top, and using a larger proportion of natural earth for self-weight.

A further advantage of this method of construction, with an interlocking block front wall, is that no special provision need be made to allow safe drainage of the wall. Groundwater pressures in the backfill can freely dissipate through the permeable wall, but the geometry of the interlocking blocks is such that washing out of fine grained soil particles will be inhibited after initial construction, as the distribution of fines stabilizes.

In this way, 2R Engineering claim that it is possible (with non-union labor) to build walls of typically 10,000 ft^2 frontal area typically 15 ft high, for \$12 to \$14/ft^2, with local cement cost of \$65 to \$70/ton, compared with \$60/ton in Texas. There seems no reason to suppose that such a unit price of wall could not be achieved here.

3. LABORATORY TESTING

3.1 Strength of Cement Stabilized Soil

In order to evaluate the specific properties of the material for this project, a specific laboratory program of material testing was carried out. These utilized Texas triaxial tests to provide a basis for prediction of stress strain characteristics at different confining pressures. This also provided information about cohesion and internal friction.

The material chosen for the study was supplied by District 12, and was a poorly graded sand from Houston used by contractors as a backfill for reinforced earth retaining walls. Table 3.1 summarizes its properties. Material passing #4 sieve was used for molding specimens. Type I Portland cement was used as a stabilizer.

TABLE 3.1 - Properties of sand tested

% Passing sieve no. 4	98
% Passing sieve no. 40	87
% Passing sieve no. 200	4
Plasticity index	NP
AASHTO Classification	A-3
UNIFIED Classification	SP
Optimum Moisture Content (%) (for sand + 7% cement)	13.5
Maximum Dry Density (pcf) (for sand + 7% cement)	109

The Impact method of compaction was adopted for all specimens, which were compacted at Texas energy. All the specimens were compacted in molds 8 in. high and 4 in. in diameter.

Seven percent cement content by weight of dry soil was initially chosen. For sand this cement content is in the range specified by PCA, for an A-3 soil. All specimens were molded at optimum moisture contents. Additional samples were later molded for sand treated with 5% cement.

Two different humidities were chosen for cement treated sand: 95% and 50%. They were tested at an age of 75 days. Additional samples cured at 95% humidity were also tested at an age of 14 days. Samples containing 5% cement were tested at ages of 7, 21 and 28 days. They were cured at 95% humidity.

Just before testing, the cylindrical samples were taken out of the curing room and were placed in a Texas triaxial cell. Porous stones were placed at top and bottom. The whole assembly was placed between the loading plates of the testing machine, and was centered. An air line was connected to the triaxial cell and lateral pressure applied. A dial gauge was placed to measure vertical deformation. Load was applied at a loading rate of 0.05 in/min. until failure. The results are presented in Tables 3.2 and 3.3.

TABLE 3.2 - Results for sand + 7% cement

Sample No.	Curing Humidity (%)	Age (Days)	Confining Pressure (psi)	Failure Stress (psi)	Strain to Failure
S1	95	75	0	318	0.0037
S2	95	75	5	337	0.0046
S3	95	75	15	399	0.0066
S4	95	75	30	461	0.0138
S8	50	75	0	515	0.0053
S10	50	75	5	578	0.0080
S9	50	75	15	649	0.0074
S5	50	75	20	664	0.0082
S6	50	75	30	670	0.0080
S11	95	14	0	222	0.0042
S13	95	14	5	199	0.0057
S12	95	14	10	273	0.0060
S14	95	14	20	318	0.0075

TABLE 3.3 - Results for sand + 5% cement

Sample No.	Curing Humidity (%)	Age (Days)	Confining Pressure (psi)	Failure Stress (psi)	Strain to Failure
S15	95	7	0	113.	0.0046
S16	95	7	5	130	0.0055
S17	95	7	10	165	0.0073
S18	95	7	20	199	0.0180
S19	95	21	0	159	0.0044
S20	95	21	5	164	0.0062
S21	95	21	10	181	0.0082
S22	95	21	20	227	0.0094
S23	95	28	0	170	0.0048

Summary of Results

Typical stress strain curves for the specimens are shown in Figures 3.1 to 3.3 for various confining pressures and curing conditions. Increasing confining pressure required increased applied stresses to fail (Balmer G.G., 1958). It is noticed that failure strains also increase as confining pressure is increased. It appears that as confining pressure is increased the specimens require more energy to fail. Stress strain curves are linear up to about one-third of the ultimate stress. It is noticed that the stress strain curve for sample "S4" which was subjected to a confining pressure of 30 psi, is flat as compared to other stress strain curves, for samples having the same curing conditions and age. This is probably due to the fact that the triaxial cell membrane used for this sample did not adequately handle confining pressures greater than 20 psi. Applying 30 psi confining pressure caused ballooning of the rubber membrane, and the deformations measured included the compression of the rubber membrane.

It is interesting to note that specimens cured at 50% humidity had higher strengths than those cured at 95% humidity. This is because of the difference in moisture contents at the time of testing. Specimens cured at 50% humidity had a moisture content of about 8%, whereas those cured at 95% humidity had a moisture content of 12%. Curing humidity did not seem to have much effect on modulus of elasticity. It was also noticed

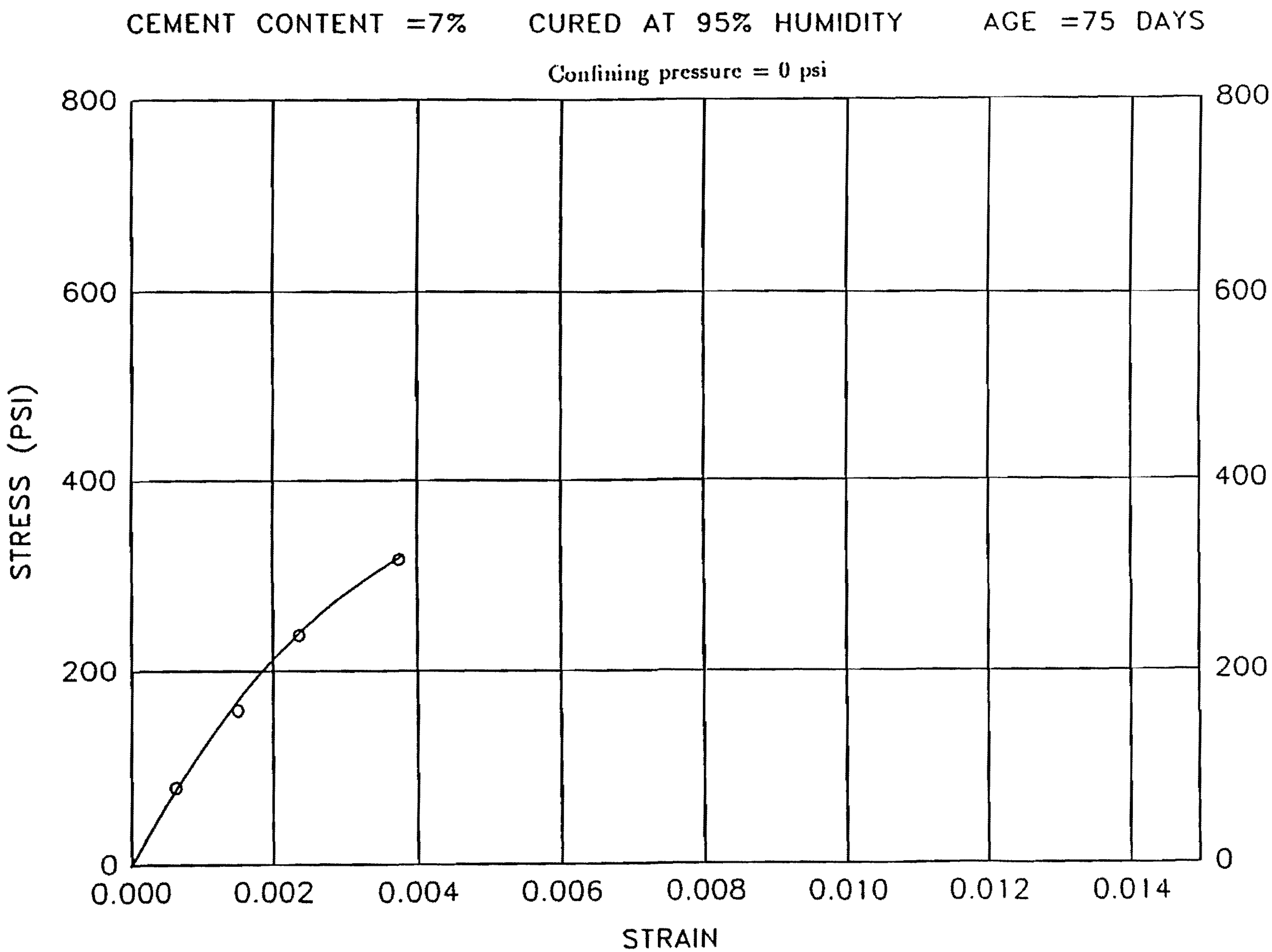


Figure 3.1 Stress-strain curve - zero confining pressure

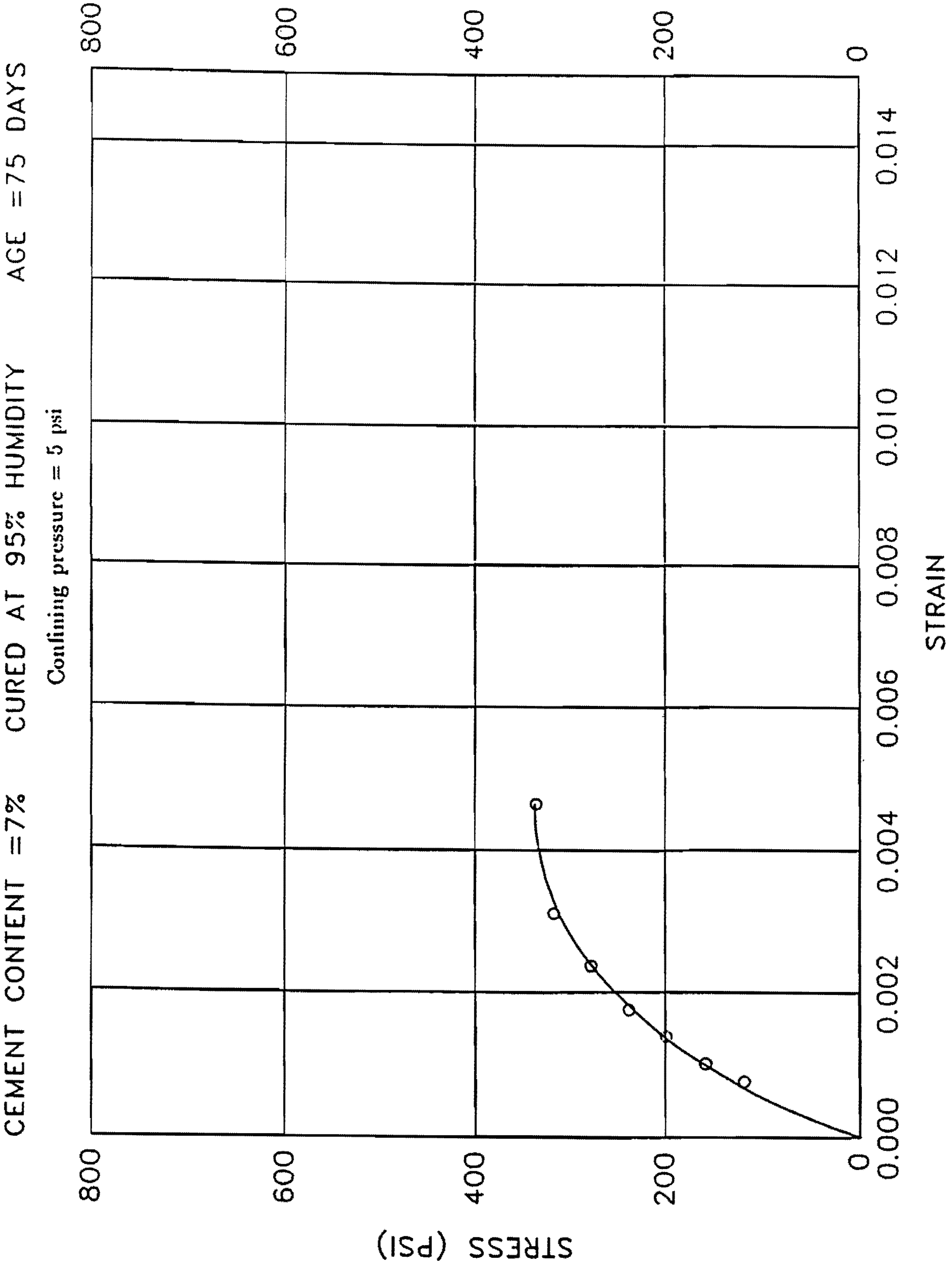


Figure 3.2 Stress-strain curve - 5 psi confining pressure

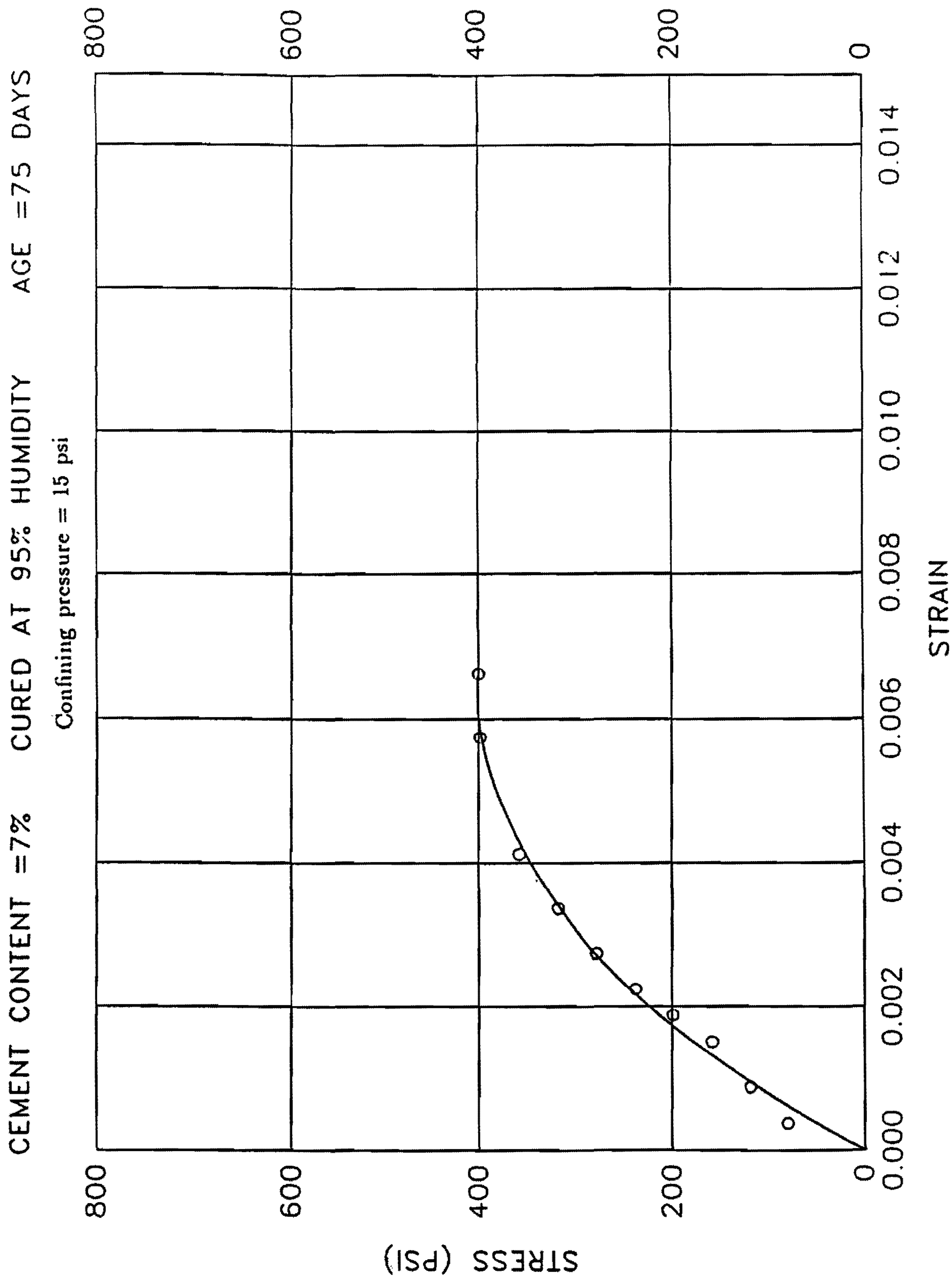


Figure 3.3 Stress-strain curve - 15 psi confining pressure

that samples cured at 50% humidity had higher failure strain. From typical Mohr-Coulomb envelopes shown in Figures 3.4 to 3.6, it is clear that curing humidity affects cohesion as well as angle of internal friction. The lower the humidity the higher the values of these parameters.

As expected, strength increased with age as summarized in Figure 3.7. The same is true for cohesion intercept as well as modulus of elasticity. The value of angle of internal friction at 14 days is higher than the corresponding value at 75 days. This is probably due to the fact that at 14 days the strength of the sample tests at a confining pressure of 5 psi had a lower than expected strength which changed the slope of the Mohr-Coulomb envelope.

Table 3.4 summarizes the mechanical properties of the stabilized soil tested. It is clear that as the cement content is decreased the strength decreases as well as the modulus of elasticity, cohesion, and angle of internal friction.

TABLE 3.4 - Summary of Cement Treated Properties

Cement Content (%)	Age (Days)	Curing Humidity (%)	E (psi)	C (psi)	ϕ (Deg)	fc (psi)
7	14	95	106000	41	44	230
7	75	95	127000	71	42	320
7	75	50	127000	107	45	320
5	7	95	38000	26	39	120
5	21	95	64000	39	35	160
5	28	95	100000			170

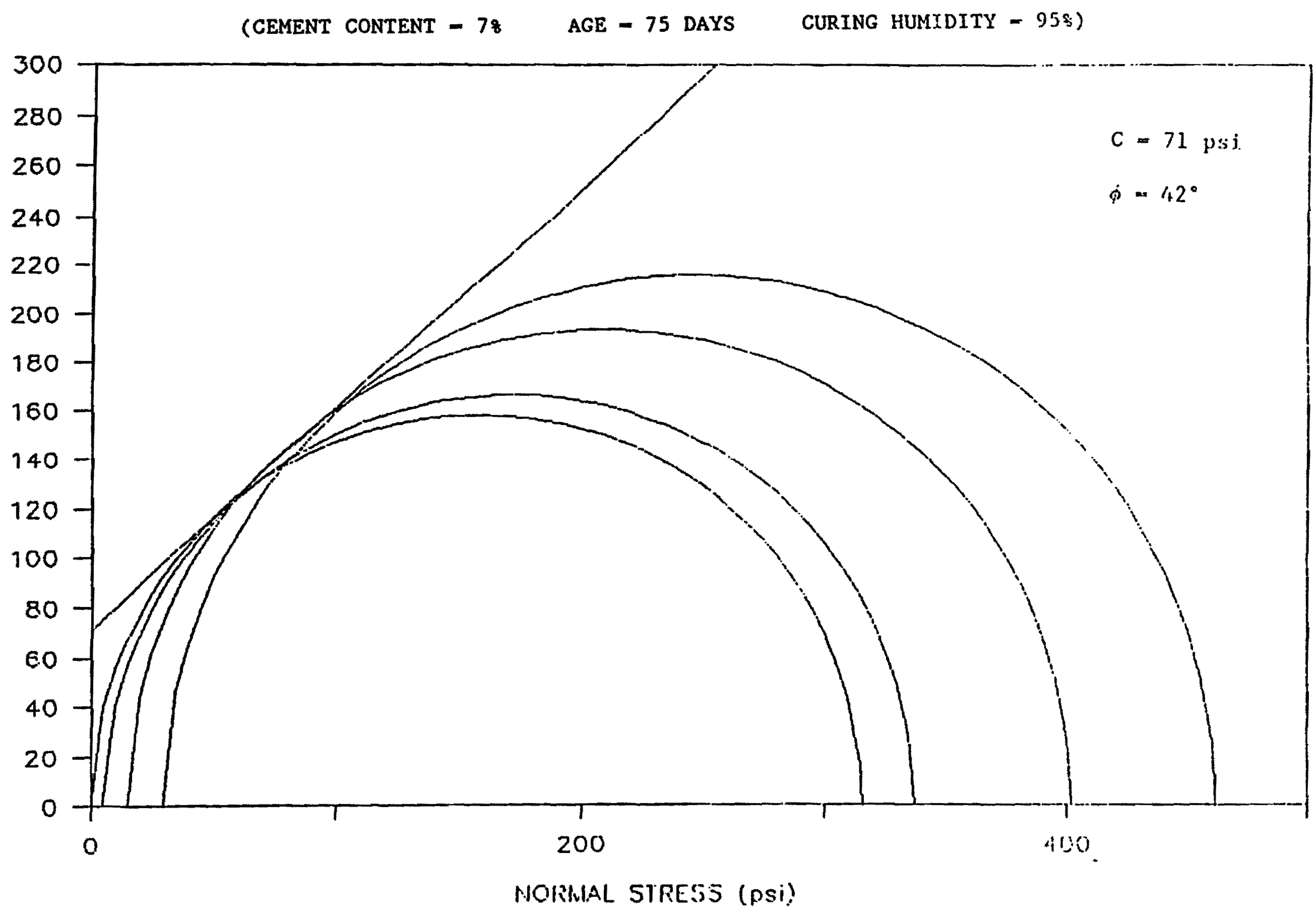


Figure 3.4 Mohr envelope, 95% humidity, 75 day cure

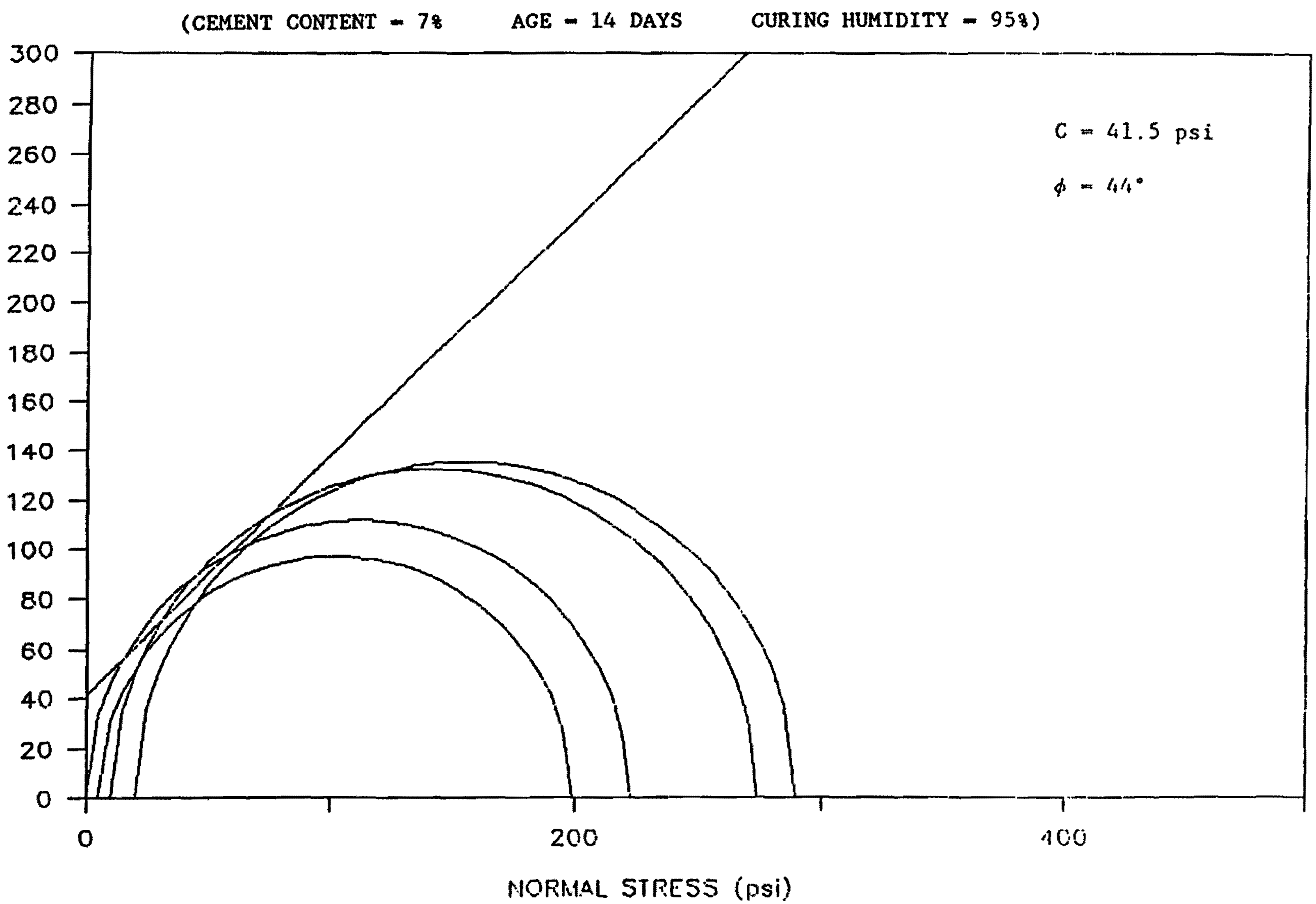


Figure 3.5 Mohr envelope, 95% humidity, 14 day cure

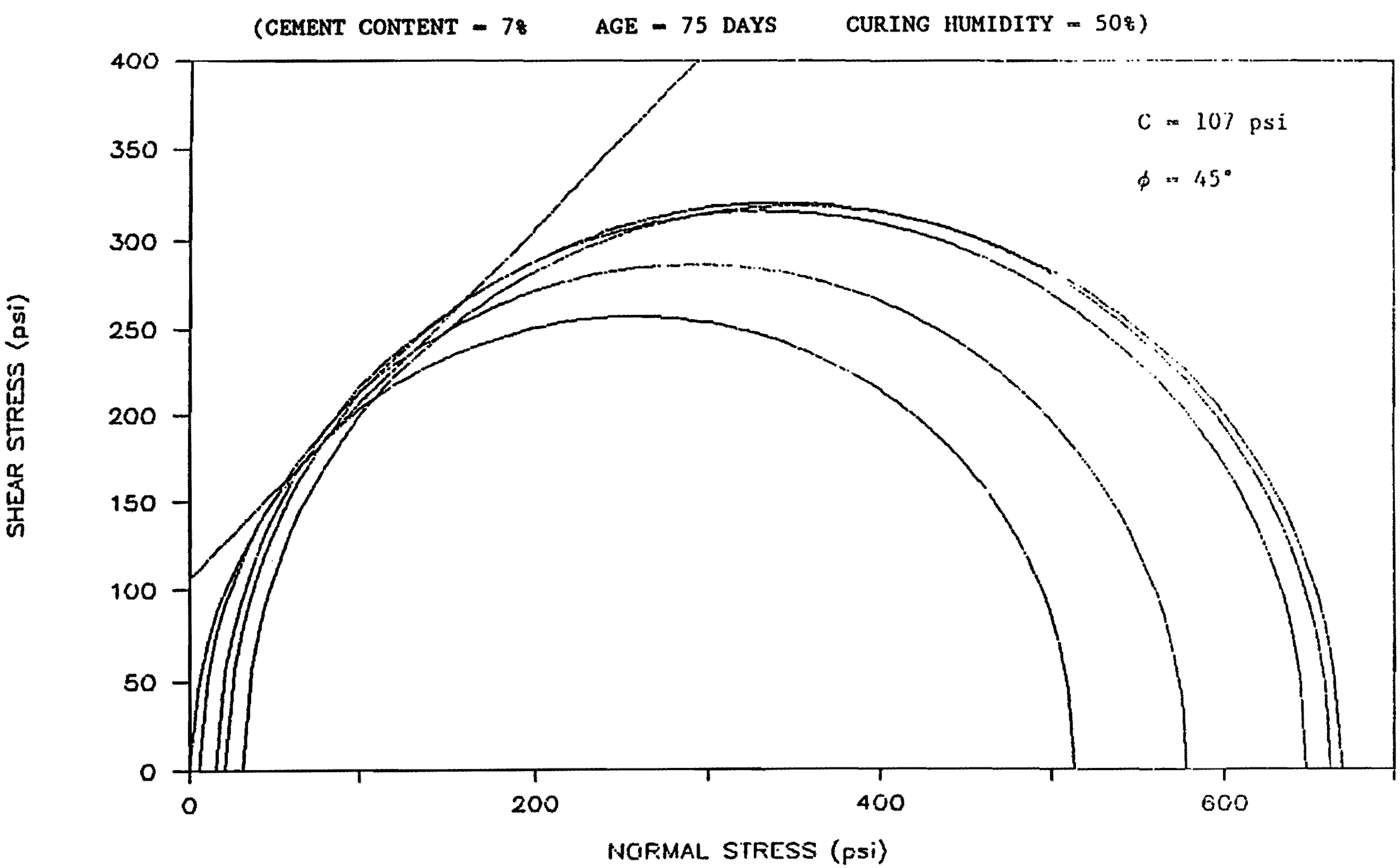


Figure 3.6 Mohr envelope, 50% humidity, 75 day cure

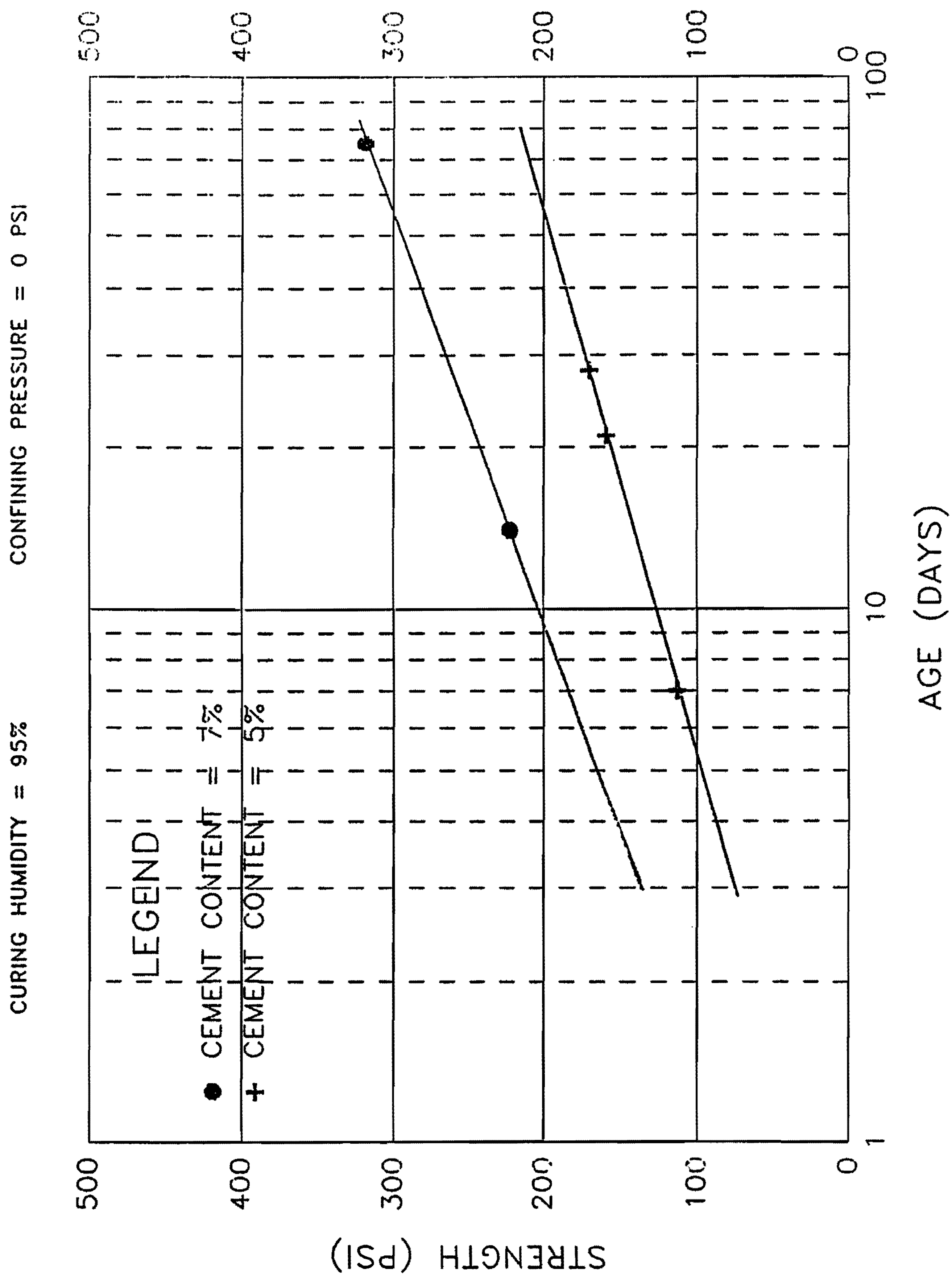


Figure 3.7 Variation of strength with age

3.2 Anchor Pull-out Testing

In this design the facing panels are tied into the stabilized soil by means of short anchors whose main function is to retain the individual facing units. They are not required to carry primary loads or to prevent structural failure. The main design criteria is that the pull-out resistance should be sufficient to counteract the outward forces that act on facing units. These forces are quite small in nature, and probably derive primarily from incidental wind loading and from eccentricities between the self-weight of a panel and its foundation.

Some time was spent on considering the design of anchors used to secure the facing units. In particular it was felt important to be sure that the pull-out capacity of anchors in cement stabilized soil could be adequately predicted (even if conservatively). The possibility of pull-out can always be reduced by making them longer, but excessive length is not desirable or economical. It is preferable to use only a short length of anchor, as it is not required to prevent structural failure of the entire wall, but merely to retain individual units. Such a design also minimizes any possibility of conflict with proprietary designs (like "Reinforced Earth"), since a short anchor is not being used to "reinforce" the soil in any way.

To establish the performance of anchors in stabilized soil, tests were conducted on 1/4 in. (6 mm) diameter bolts embedded to a length of 1-1/8 in. (30mm) and 1-3/4 in. (45 mm) in sand stabilized with 5% cement. These were performed in accordance with ASTM standard C900 for pull-out of anchor bolts in concrete, so that a direct comparison could be made between behavior of anchors in stabilized soil and behavior in weak concrete. Pullout was performed on the bolts with hydraulic ram after curing times of 7, 14, and 28 days, and could be compared to the known strength properties of the cement stabilized soil mix under these conditions.

Typical results of the pullout test for the 1-3/4 in. (45 mm) embedment length are summarized in Figure 3.8. Resistance to pullout was observed to be significant even for small anchors. The head of the anchors used in the pullout test was 0.72 in. (18 mm) in diameter, and after 7 days of curing, the force required to pull out the anchors was more than 250 lbs (1 kN). Furthermore, the resistance to pullout increased significantly in the following 21 days. Comparison of the pull-out capacity of the anchors in cement stabilized soil with standard predictive formulae using the ACI concrete code

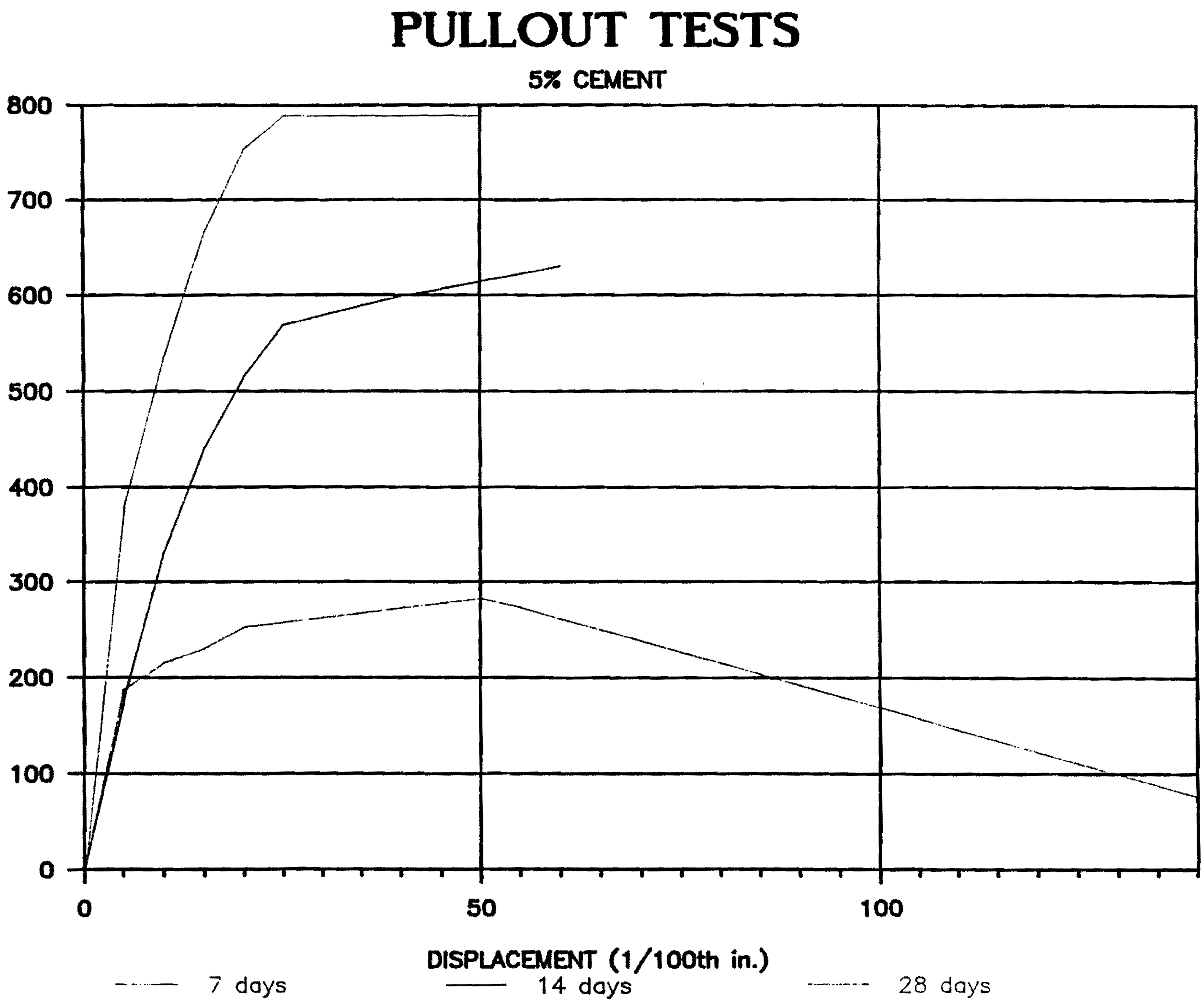


Figure 3.8 Anchor pull-out curves

(and incorporating the known compressive strength characteristics of the stabilized material) has indicated that measured capacities exceed theoretical predictions, once due allowance is made for the actual strength. In reality major reductions in stabilized soil strengths resulted in correspondingly minor reductions in pullout values - certainly the capacities were not reduced proportionately.

As a result it appears that there will be no difficulty in supporting anchor loads with steel bars or metal strips in cement stabilized soil, and that standard pullout formulae from concrete design can be conservatively extrapolated to this material.

In reality, results from centrifuge testing (see next section) indicate that field loads on the anchors from the facing units will be small (of the order of 200 lbs full scale equivalent), so that actual anchor design is probably best based on ease of installation. It would also be desirable to be able to mobilize full capacity with the soil with a minimal amount of wall movement, possibly by incorporating a threaded section that would allow for a pretensioning load to be applied to the anchor. However, field experience so far does not indicate that this is really necessary.

Anchor Design Procedure

Consequently, following ACI practice, the following design procedure was recommended for facing unit anchors in cement stabilized soil. This is analogous to pullout formulae for concrete (although the coefficients are different) and assumes that the capacity is the sum of shear and frictional components.

Pullout Capacity of a rough anchor can be predicted in lbs. as the sum of shear and frictional components (shear usually dominates)

$$P = A_s(2d\sqrt{f_c} + \gamma H f)$$

where: - A_S is surface area of anchor in in.^{2}, ignoring outermost 12 in.

(= π dL for a round section, e.g. rebar) d is diameter in inches (or equivalent circular diameter in the case of non-circular sections)

f_c is compressive strength of cement stabilized soil in psi.- preferably 7 day value (typically 100 to 300 psi.)

γ is unit weight of cement stabilized soil in lbs/in^3 (typically 0.07 lbs/in^3 i.e. 121 lbs/ft^3)

H is overburden depth of anchor under cement stabilized fill in <u>inches</u>

f is soil friction coefficient, either Reinforced Earth specs. which vary from 1.5 at the surface to tan $\varnothing$ at depth, or can be assumed as tan $\varnothing$ everywhere (0.7 for $\varnothing \simeq 35°$)

Alternatively, instead of cylinder compression testing to obtain f_c, if Texas Triaxial testing is preferred, the expression $2d\sqrt{f_c}$ may be replaced by the cohesion intercept C in psi from test envelope. 7 day values should be used, or else values reduced to 7 days, no more.

Using an A.C.I. type hook on anchor end will be beneficial (e.g. if standard rebar is used) and can be expected to contribute additional pull-out capacity of

$$70d^2\sqrt{f_c}\text{lbs.}$$

Stresses within the Anchor (i.e. just dividing the design anchor load by the cross-sectional area in each case) should also be checked to ensure they not exceed permissible working values, but these will normally be nominal.

Horizontal Design Loads to be applied to facing units (and suitably distributed between horizontal anchors if more than one) are suggested as the greater of :-

(a) 10 lbs/ft^2 possible wind suction applied to entire facing area

(typical code design values for localities like Houston are 3 to 4 times this, but in reality exposures are unlikely to be severe, and suction pressures will only act on a portion of the overall unit area)

(b) 10% of weight of facing unit

(to allow for possible eccentricities of support conditions. Will be less than (a) unless facing is thicker than the equivalent of 10 inches of concrete.)

(c) 200 lbs. (minimum nominal force from external sources)

Factor of Safety against pullout is recommended as :-

2.5 if a single anchor is provided per facing unit
2.0 if two anchors are provided per facing unit
1.5 if three or more anchors are provided per facing unit
(these values should normally be easily exceeded)

It is worth noting that this last condition is similar to the Reinforced Earth factor of safety, except that they apply this only for the "resistive zone", and that they also specify a design alternative of testing to 1.33 times the working load.

4. NUMERICAL MODELLING

4.1 Introduction

A numerical analysis of the cement-stabilized soil retaining wall system was performed using a commercially available finite element computer code capable of modelling geotechnical materials, "ABAQUS", available under academic license from Hibbitt, Karlsson, & Sorensen, Inc., Providence, R.I.

The objective of the numerical analyses was to predict the response of the proposed wall system under different geometric and loading conditions. Emphasis was placed on investigating the distributions of displacement, stresses, plastic deformation (or failure mode), and ultimate load bearing capacity of the system. The system was modelled using 2-dimensional finite elements (plain strain conditions) and nonlinear plastic material characteristics. The concrete-panel facing and steel straps were not incorporated in the analysis because of their minor influence on the overall stability of the wall system and also because of the additional complications involved in their modelling.

4.2 Details of Computer Code Employed

ABAQUS is a comprehensive finite element analysis program with high analysis capabilities, and is capable of performing linear as well as nonlinear static and dynamic analysis. The nonlinearities may be due to geometry or material properties or combinations. A primary merit of the code lies in its extensive capabilities of modelling different types of materials including metal, soils, rock, and concrete. ABAQUS also contains commendable pre- and post-processing utilities.

Cement-stabilized soil was assumed to behave in a similar fashion to plain concrete. The concrete model in ABAQUS is designed to provide a general capability for modelling plain and reinforced concrete in different types of structures. The model is designed for applications in which the concrete is subjected to essentially monotonic loading at low confining pressures. When the principal stress components are dominantly compressive, the response of the concrete is modelled by an elastic-plastic theory, using the three-dimensional yield surface written in terms of hydrostatic pressure (p; measure of normal stress) and equivalent deviatoric stress (q; measure of shear stress).

Associated flow rule and isotropic hardening laws are used (Chen and Saleeb, 1982). A "crushing" type of failure occurs when the state of stress is of compressive type, and once crushing occurs, the material element is assumed to lose its strength completely. In tension, a "cracking" type of failure occurs, and when cracking is defined to occur by the tension failure surface, the material loses its tensile strength normal to the crack direction but retains its strength parallel to that direction.

The cracking and compressive responses of concrete that are incorporated in the model are illustrated by the uniaxial response of a specimen shown in Figure 4.1. When concrete is loaded in compression it initially exhibits elastic response. As the stress is increased some plastic straining occurs, and the response of the material softens (Owens and Hinton, 1980). An ultimate stress is reached, after which the material loses its strength until it can no longer carry any stress. When the specimen is loaded in tension it responds elastically until cracks form abruptly at a stress that is typically 7%-10% (9% used for the current analysis) of the unconfined compressive strength. In multiaxial stress state these observations may be generalized through the concept of a failure surface in multidimensional stress space (Desai and Siriwardane, 1984). Typical failure surfaces in two and three dimensional stress spaces are given qualitatively in Figures 4.2 and 4.3.

The behavior of granular materials is complex, but under essentially monotonic loading conditions rather simple constitutive models provide useful design information. The extended Drucker-Prager plasticity model is used in ABAQUS in modelling granular materials. This model uses a smoothed Mohr-Colulomb failure surface and non-associated plastic flow rule (associated flow can also be chosen, and was used for the current analysis). The restrictions associated with the conventional Mohr-Coulomb criterion, such as neglected effects of intermediate principal stress on the failure criterion and computational difficulties associated with the sharp corner in the failure curve, can be overcome by adopting the Drucker-Prager model (Chen and Baladi, 1985). The Drucker-Prager failure criterion in comparison with the Mohr-Coulomb criterion is shown in a principal stress space and on a deviatoric plane in Figures 4.4 and 4.5, respectively. It is indicated that the shear resistance of soil increases approximately linearly, as the hydrostatic pressure to which the soil is subjected increases. As is conventional, the angle of internal friction (phi) and cohesion (c) are the two most important soil parameters in determining soil shear strength.

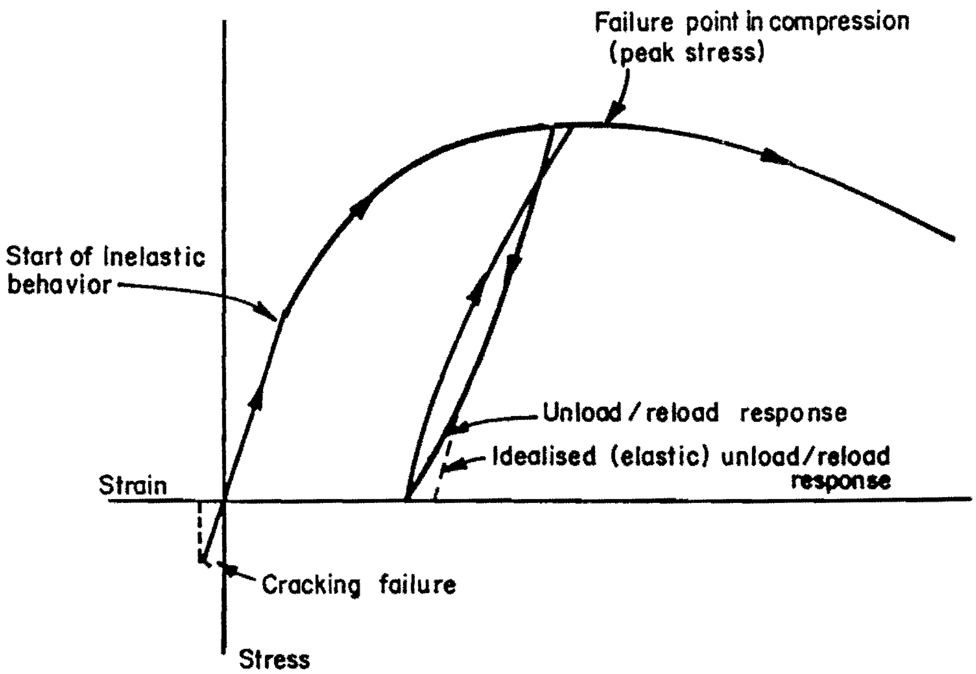


Figure 4.1 Uniaxial behavior of plain concrete

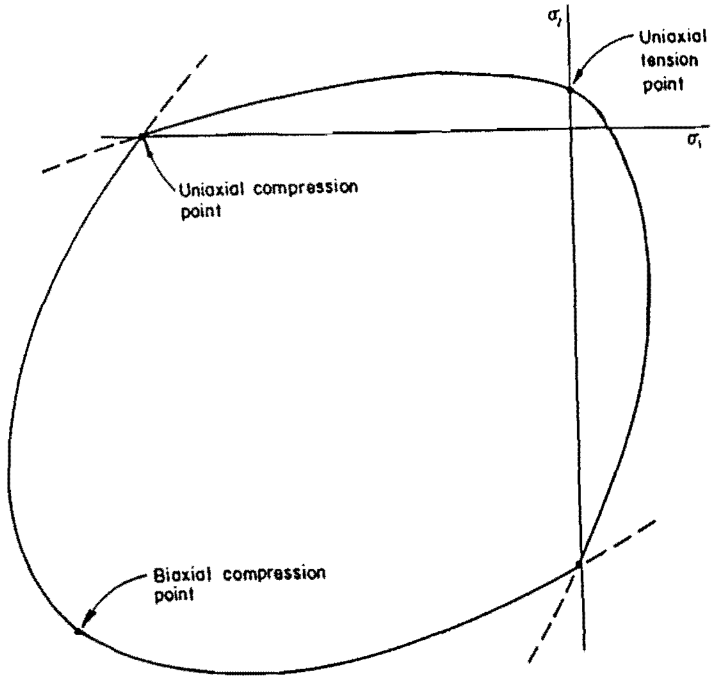


Figure 4.2 Concrete failure curve in 2-d stress

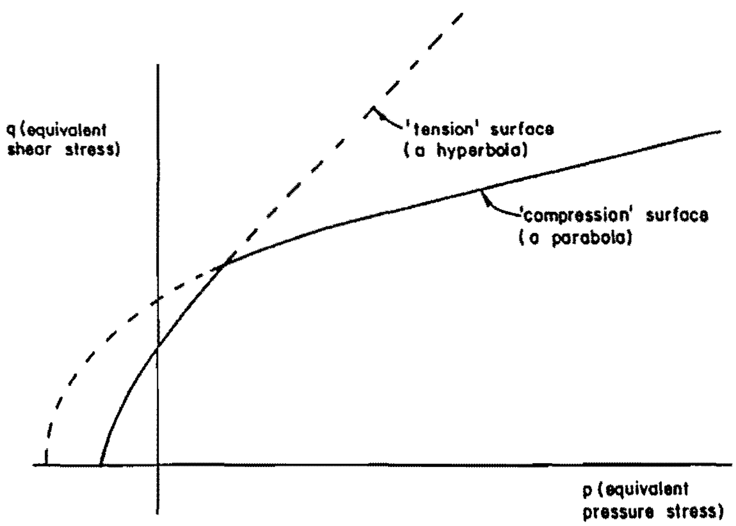


Figure 4.3 Concrete failure curve in 3-d stress

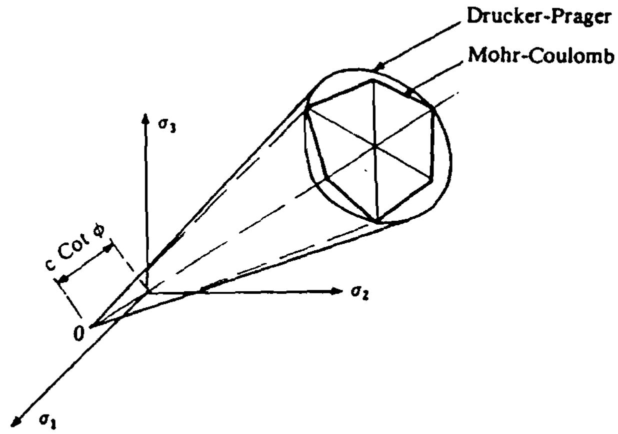


Figure 4.4 Yield surfaces in principal stress space

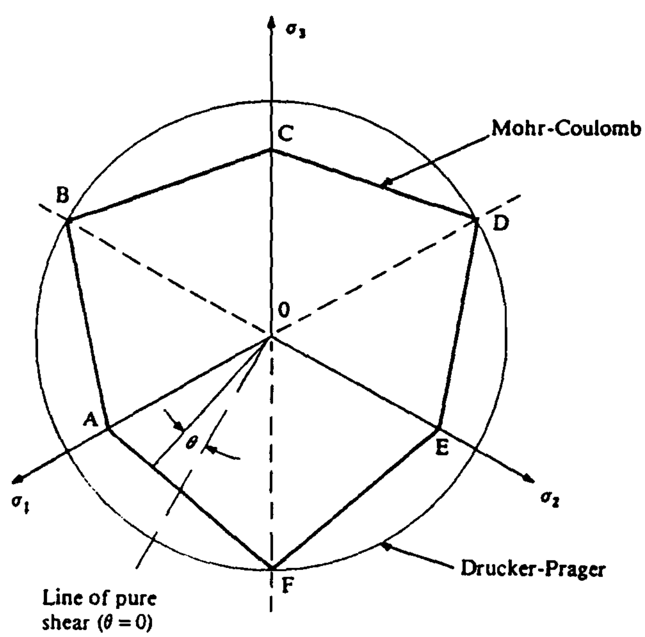


Figure 4.5 Yield surfaces in deviatoric plane

4.3 Input Data

The finite element mesh for a typical model (with 10 m x 8 m cement-stabilized wall) used for the analysis is shown in Figure 4.6. The model consists of two-dimensional quadrilateral/triangular solid elements of plain strain condition. The analysis model is assumed to be laid on a fictitious "rigid" foundation, and the vertical boundaries at the left- and right-hand sides are assumed to be on rollers to allow downward movement due to construction of the embankment (Chang, 1977). The size effects from restricting the dimensions proved to be negligible. The reference model consists of 142 nodel points and 150 elements.

Material properties of the cement-stabilized soil obtained through laboratory testing were incorporated in the analysis, as follows. Otherwise typical values introduced in published literature were adopted and utilized in the analysis, as follows.

TABLE 4.1 - Stabilized fill properties used in analysis

Unconfined Compressive Strengths 320 psi (2.21 MPa; test data)
Uniaxial Tensile Strengths 29 psi (0.20 MPa; 9% of Unconfined Compressive Strength was adopted as recommended in ABAQUS manual)
Initial Young's Modulus = 120 ksi (830 MPa; test data)
Poissons' Ratio = 0.14 (from literature)
Total Unit Weight = 125 pcf; (20 kN/M^3; test data)
Uniaxial Stress-Strain Relation (from 9% cement content, 75 day aging, 95% relative humidity test data) is presented in Figure 4.7.

Material property values of typical sandy soil introduced in the literature were incorporated in the analysis of the backfill and foundation. Relatively conservative values were adopted. The following are material input data associated with the granular backfill and foundation soil:

TABLE 4.2 - Soil properties used in analysis

Internal Friction Angle:	35 degrees
Cohesion:	104 psf (5 kPa)
Initial Young's Modulus:	15 ksi (100 MPa)
Poisson's Ratio:	0.3
Total Unit Weight:	110 pcf (17.5 kN/M3)

Gravity was used as the primary source of loading on the system. The cement-stabilized wall and soil backfill were treated as if they were constructed on top of the existing grade. No excavation situation was considered. Therefore, the applied load consisted only of the self-weight of the embankment (cement-soil wall and granular backfill). The typical loading arrangement is shown in Figure 4.6. Ultimate load bearing capacity of each model was assessed in terms of the multiple of gravity each model could withstand. Traffic surcharge loading was also considered on some models.

4.4 Results for 80% Aspect Ratio Wall

a. Model Description

The finite element mesh arrangement and loading conditions for the reference model have already been shown. Gravity of the cement-stabilized wall and soil backfill constituted applied loading. Dimensions of 10 m height and 8 m width were arbitrarily but carefully chosen for typical wall cross sections, and will be used as a reference section for subsequent analyses of different models.

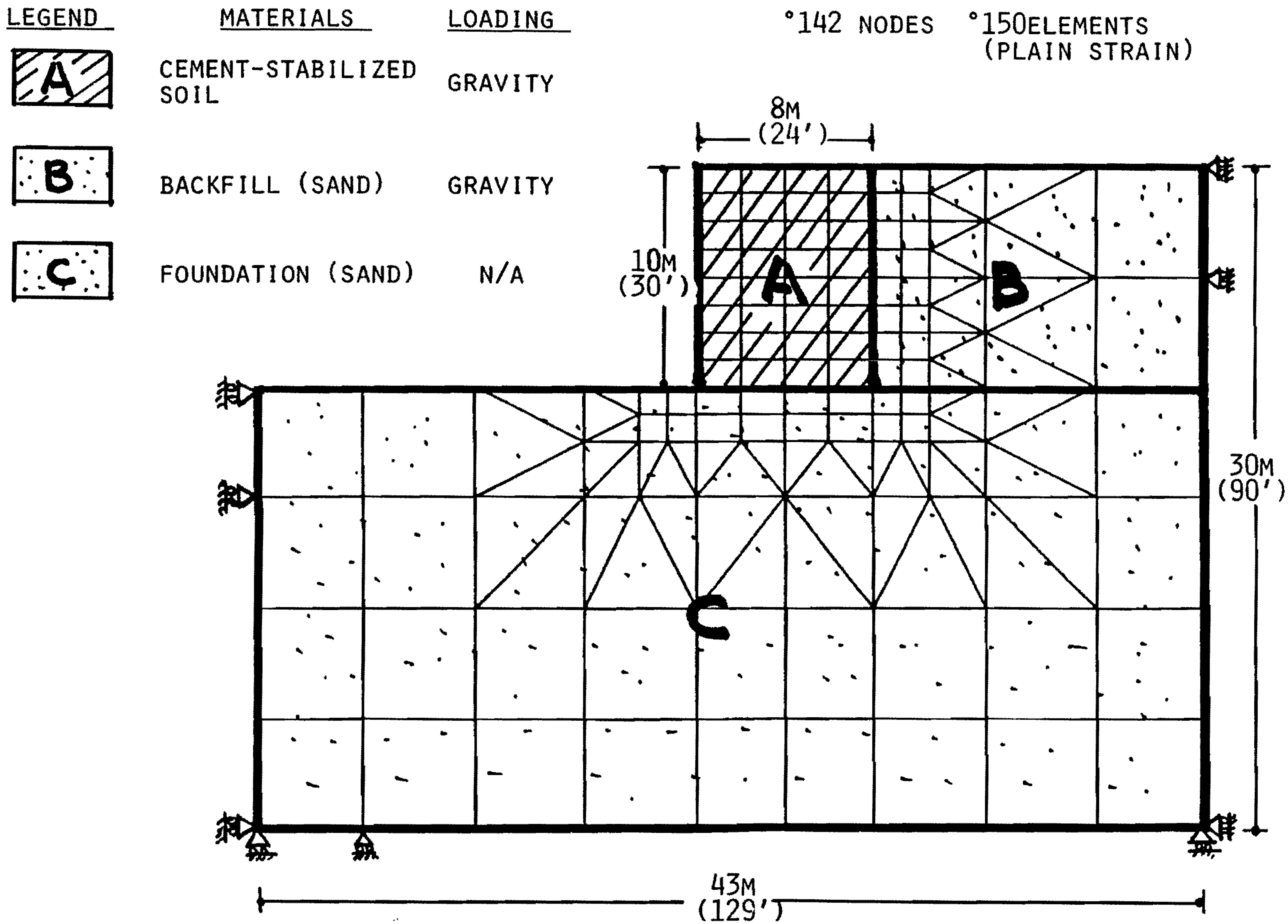


Figure 4.6 Mesh for 80% aspect ratio wall

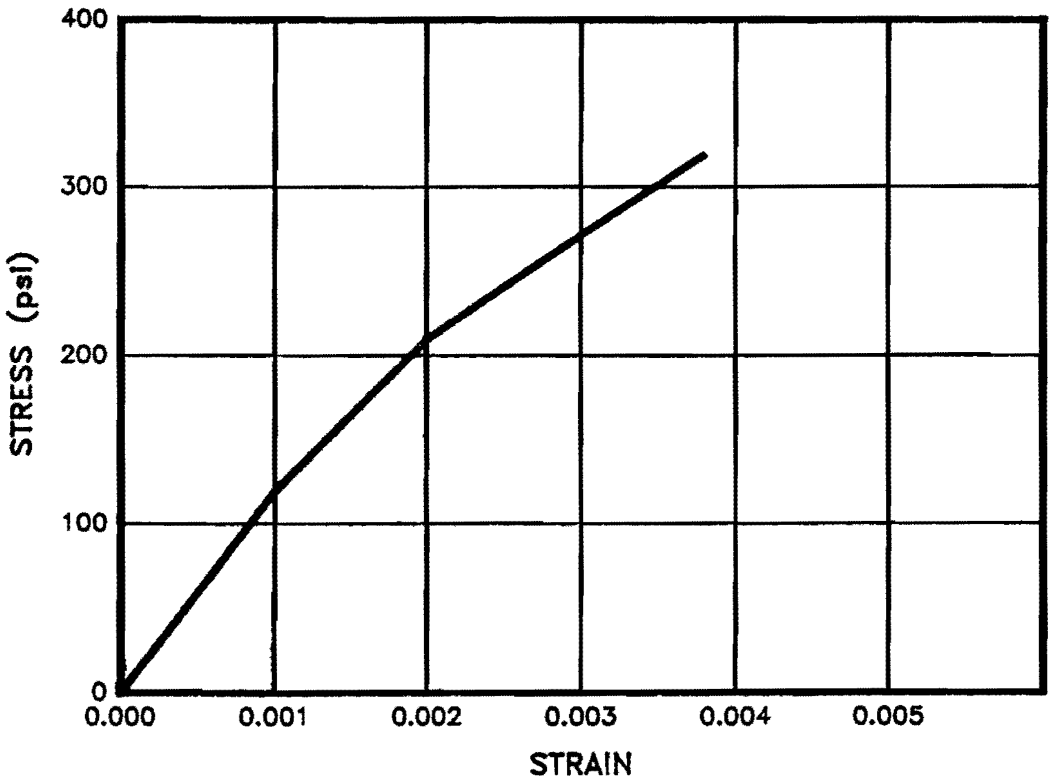


Figure 4.7 Uniaxial stress-strain relation used

b. Displacements

The displacement field of the model is shown in Figure 4.8. In general, the embankment settles downward due to its gravity and also rotates around the toe of wall. The foundation soils below the embankment are compressed due to the weight of the embankment, with some corresponding heave at the other side of the existing ground. Significant distortion was predicted of the soil elements around the toe of the wall. The wall rotates toward the backfill instead of overturning toward the face of the wall.

This was later observed also in the field, and would be expected to be true for walls of thick aspect ratio, which are not close to failure. For thin walls however, of narrow aspect ratio that are much closer to failing by overturning, then one might expect some outward movements to take place as bearing capacity failure starts to take place under the front toe. In this case however, the program failed to converge (indicating the onset of failure) before such an effect was observed.

c. Minor Principal Stresses (Maximum Compression Stresses)

The distribution of maximum compression stresses is shown in Figure 4.9. Since ABAQUS uses the sign convention generally used in continuum mechanics (i.e. tension is positive), the minor principal stresses computed correspond to maximum compression stresses. A significant stress concentration was observed around the toe of the wall. The maximum compressive stress at the toe was about 1.5 times the overburden pressure, and this value represents approximately 13% of the unconfined compressive strength of the cement-stabilized soil.

d. Major Principal Stresses (Maximum Tension Stresses)

The distribution of maximum tension or minimum compression stresses is shown in Figure 4.10. Tensile stresses are observed near the toe and top of the cement-stabilized wall. The maximum tensile stress observed within the wall represents approximately 3% of the uniaxial tensile strength of the cement-stabilized soil.

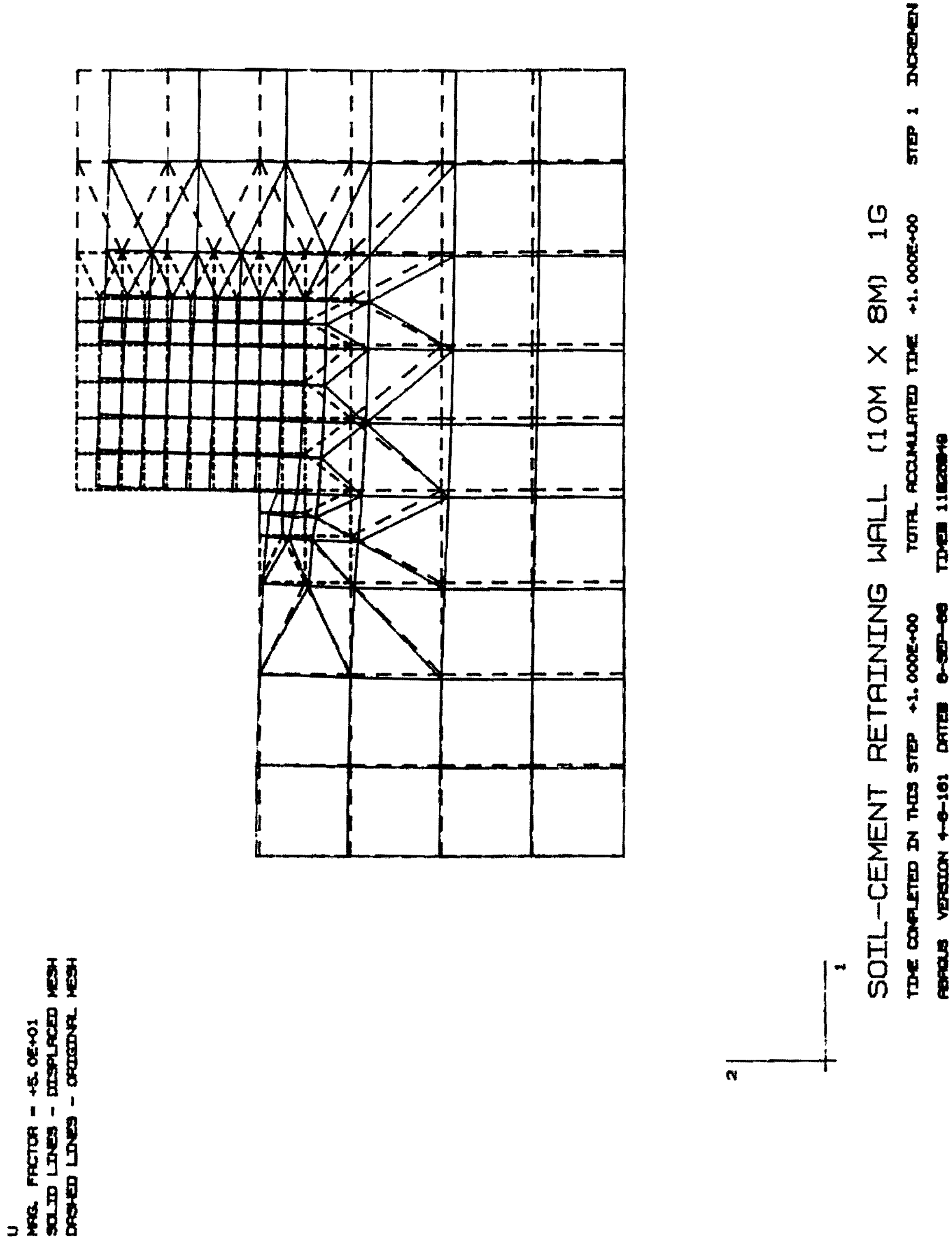


Figure 4.8 Displacements for 80% aspect ratio wall, 10 m. high

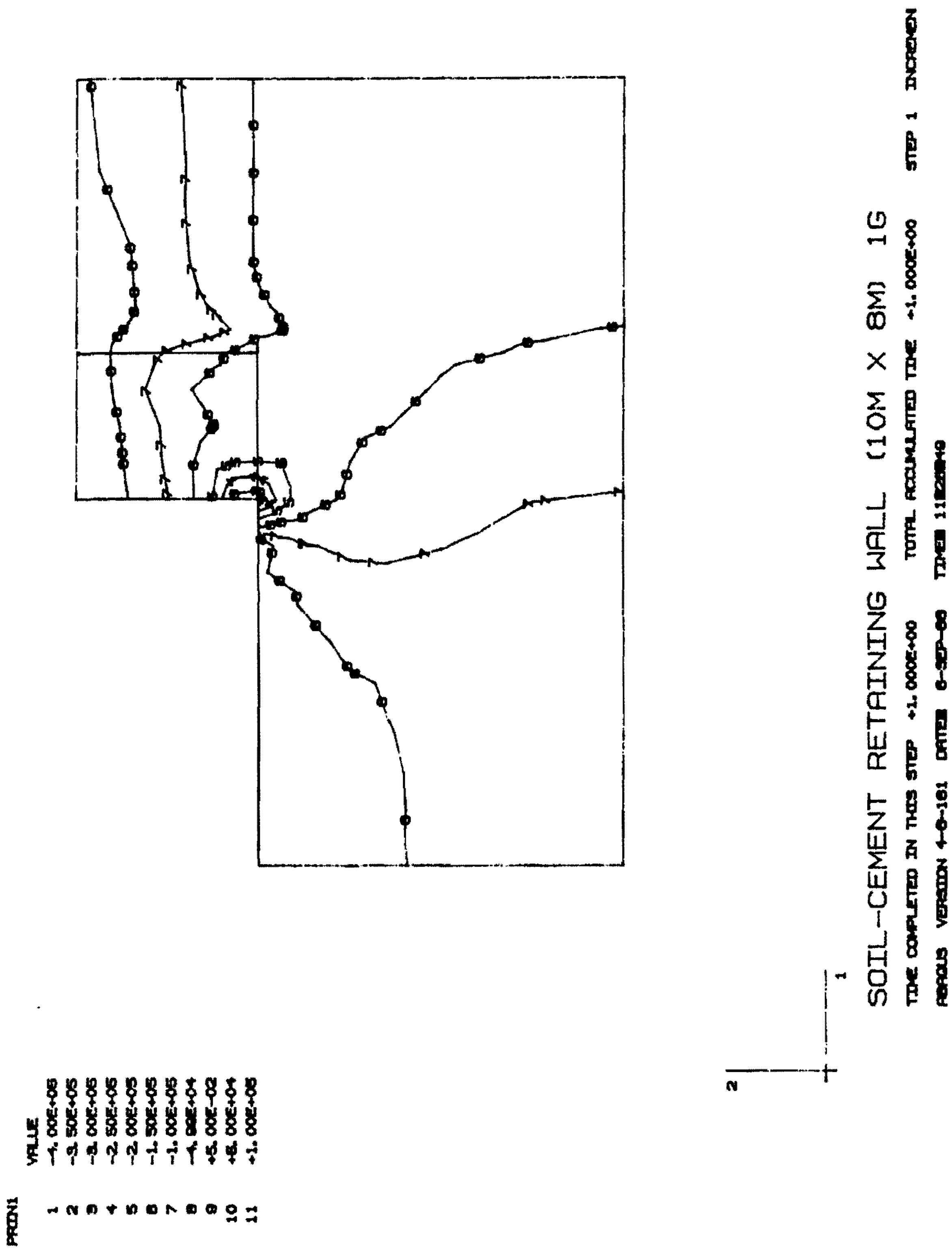


Figure 4.9 Maximum compressive stresses

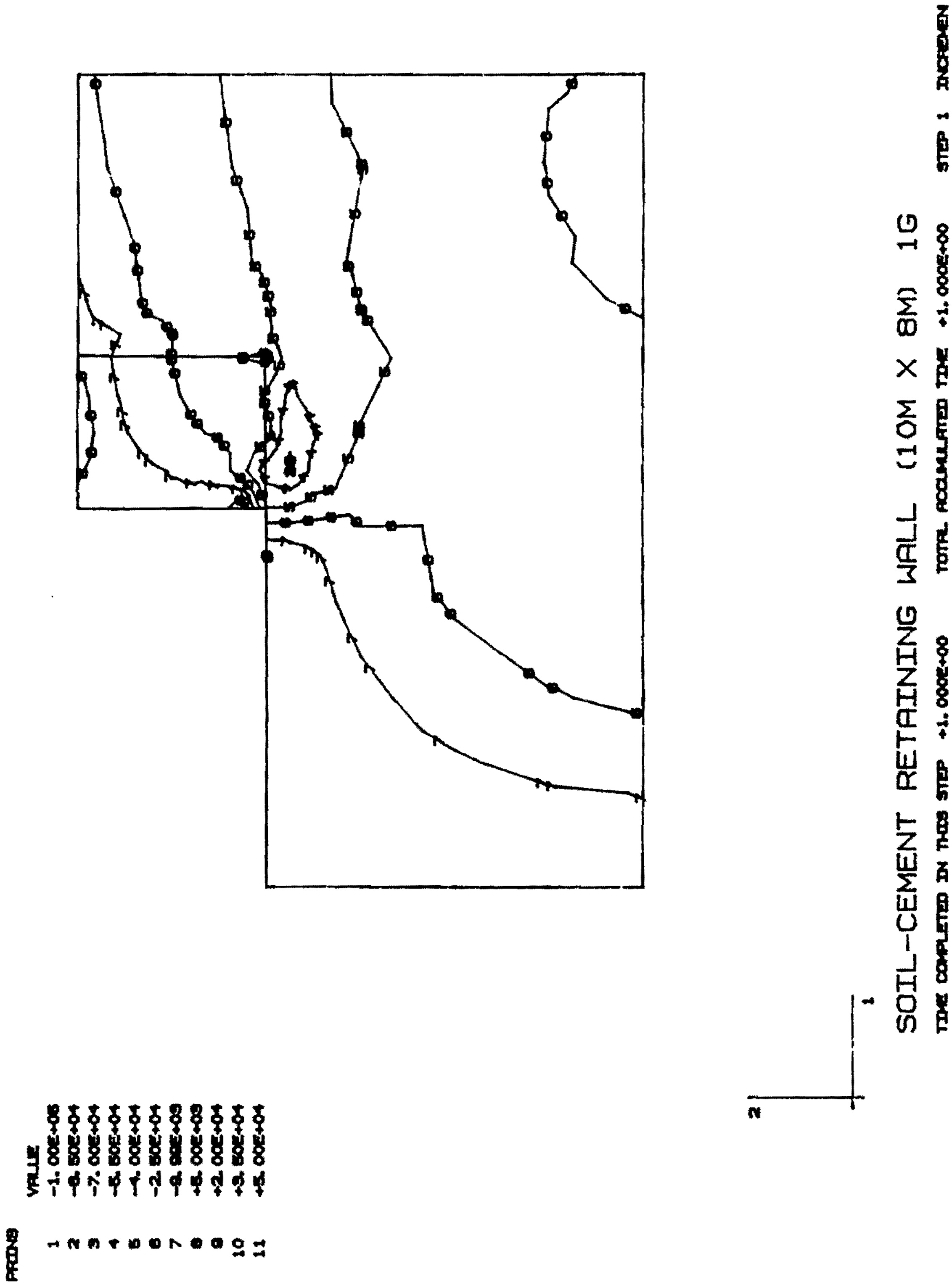


Figure 4.10 Maximum tension stresses

e. Maximum Shear Stresses

The maximum shear stress is measured in terms of equivalent deviatoric stress. The equivalent deviatoric stress actually represents two times the maximum shear stress. Figure 4.11 shows the distribution of maximum shear stresses within the model. The maximum value was observed at the toe of the wall. The maximum shear stress is compared with the maximum shear strength available at that point to determine possible failure condition. The shear strength available at a point is dependent upon the hydrostatic pressure at that point. The contours also indicate that shear stresses drop sharply along the boundary between the wall and soil backfill, which may be due to low interface friction between the two different materials.

f. Plastic Deformation and Failure Mode

The program enables the extent and magnitude of plastic deformations of the constituting materials to be represented in contours. They indicate that there will be a localized plastic zone within the soil beneath the toe of the wall, which suggests a local bearing capacity failure. Possible ultimate failure modes may be established by carefully inspecting the contours of plastic deformations computed under the ultimate loading conditions. Figure 4.12 shows the contours of plastic deformations under the ultimate loading of 7.8 g (which corresponds theoretically to a 255 ft. tall wall (78-meter). The figure suggests that the wall would fail by rotating around its toe, due to bearing capacity failure of the foundation soils.

4.5 Results for 40% Aspect Ratio Wall

The displacement response of this model is generally similar to that of the reference model (10 m x 8 m). However, distributions and magnitudes of different stresses are somewhat different from those of the reference model. The maximum compression stress occurring at the toe was about 1.8 times the overburden pressure, and the magnitude and areal extent of plastic deformations are slightly increased. A small zone of tensile failure was detected at the top of the wall/backfill boundary under 1 g loading. The model withstood approximately 5.8 g gravity loading which is about 74% of what the reference model carried, and corresponds to a maximum height of 190 ft. (58 m).

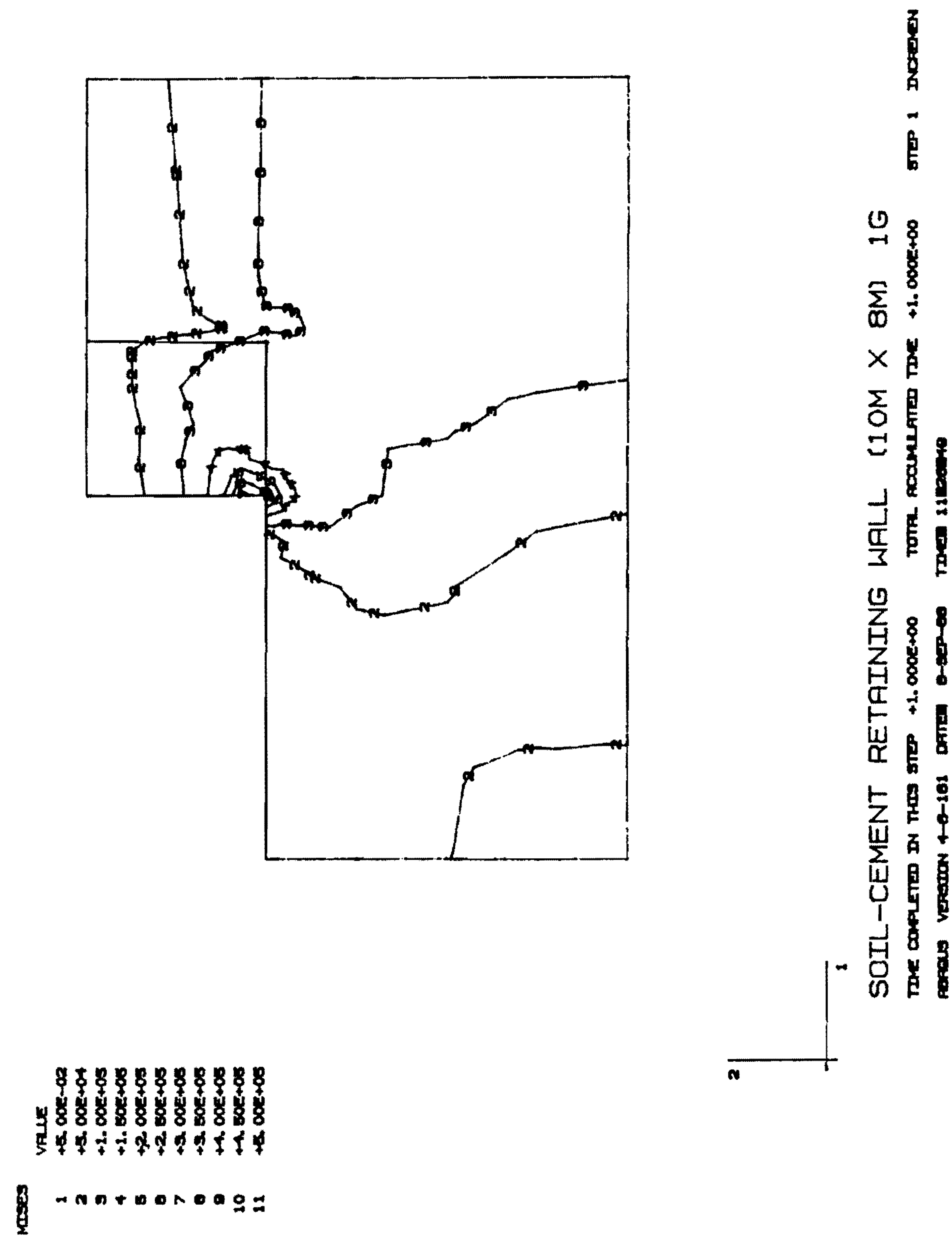


Figure 4.11 Maximum shear stresses

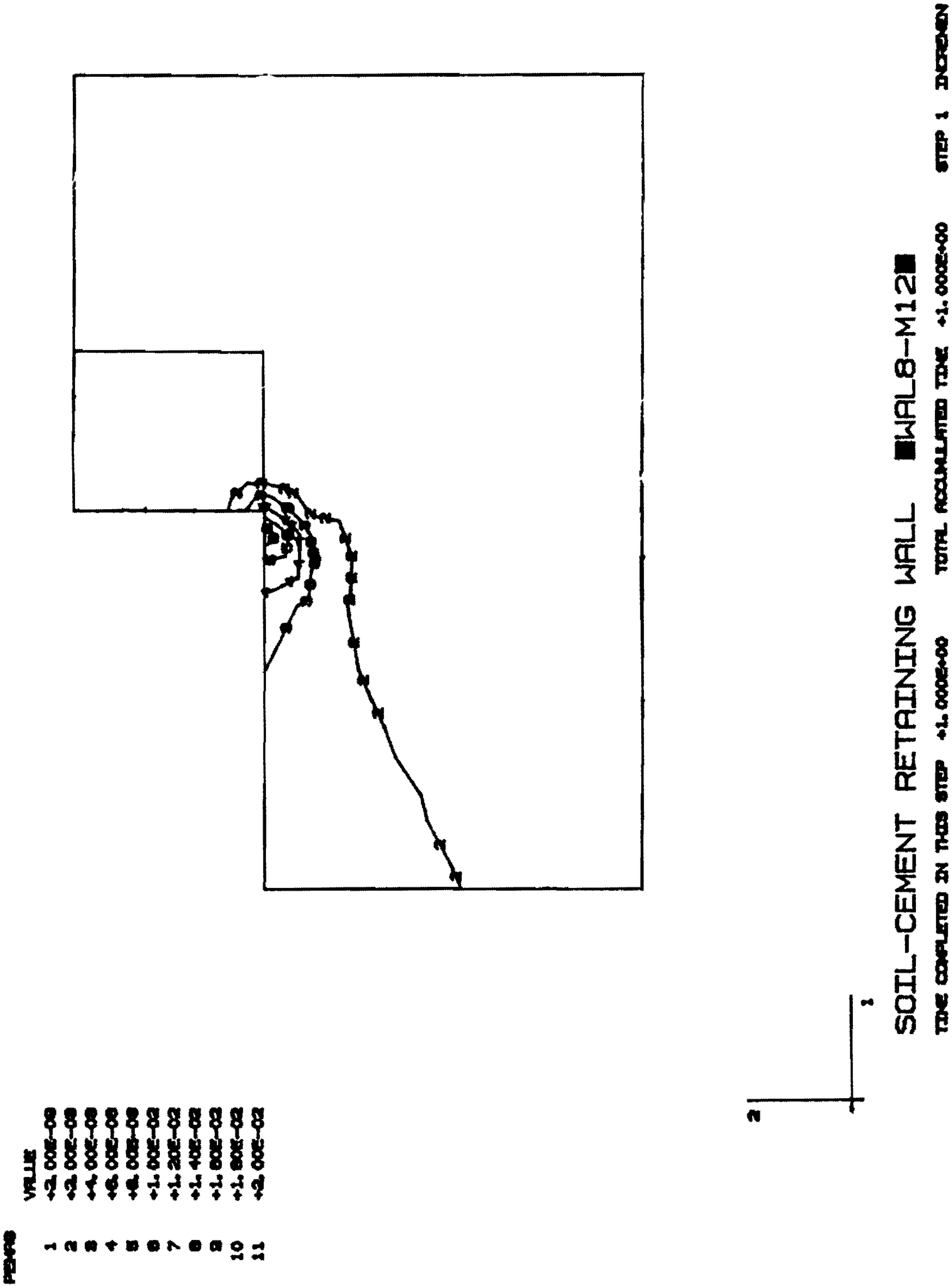


Figure 4.12 Plastic deformations

Extensive tensile failures were predicted along the upper wall/backfill boundary. Figures 4.13 through 4.15 show various responses of the model under 1 g loading, and Figure 4.16 under the ultimate loading of 5.8 g.

4.6 Other Cross Sections

a. Lower Triangular Section

The "lower triangular" model is shown in Figure 4.17. The performance of this model appears to be as good as that of the reference model. An ultimate gravity loading of 7.7 g (in comparison with 7.8 g for the reference model) was carried by this model. Therefore, substantial savings in material cost may be realized by adopting this model without significantly sacrificing the performance of the system. It would also be possible to use weaker material towards the top of the wall, thereby effecting some economies, without significant loss of safety.

b. Upper Triangular Section

The "upper triangular" model is shown in Figure 4.18. The performance of this model is quite poor compared to the lower triangular model. An ultimate loading capacity of 5.0 g was predicted, (i.e. a full scale height of 164 ft. or 50 m).

4.7 Surcharge Loading

Traffic load on the embankment was simulated by applying a roughly estimated surcharge loading to the reference model. A load intensity of 345 psf (16.5 kPa; about 3 ft of soil overburden) was arbitrarily used at this stage. The results indicate that the influence of surcharge loading on the performance of the wall is negligible. However, dynamic effects of the traffic loading have not been considered, and were not attempted in the current study because of the prohibitive scope of work involved.

4.8 Conclusions

The results of the numerical analyses of the proposed cement-stabilized soil retaining wall system indicated that the system should perform adequately under the assumed geometric and material conditions. This analysis assumed that the subsoil could be idealized as a primarily frictional material. The wall would behave as an integral mass

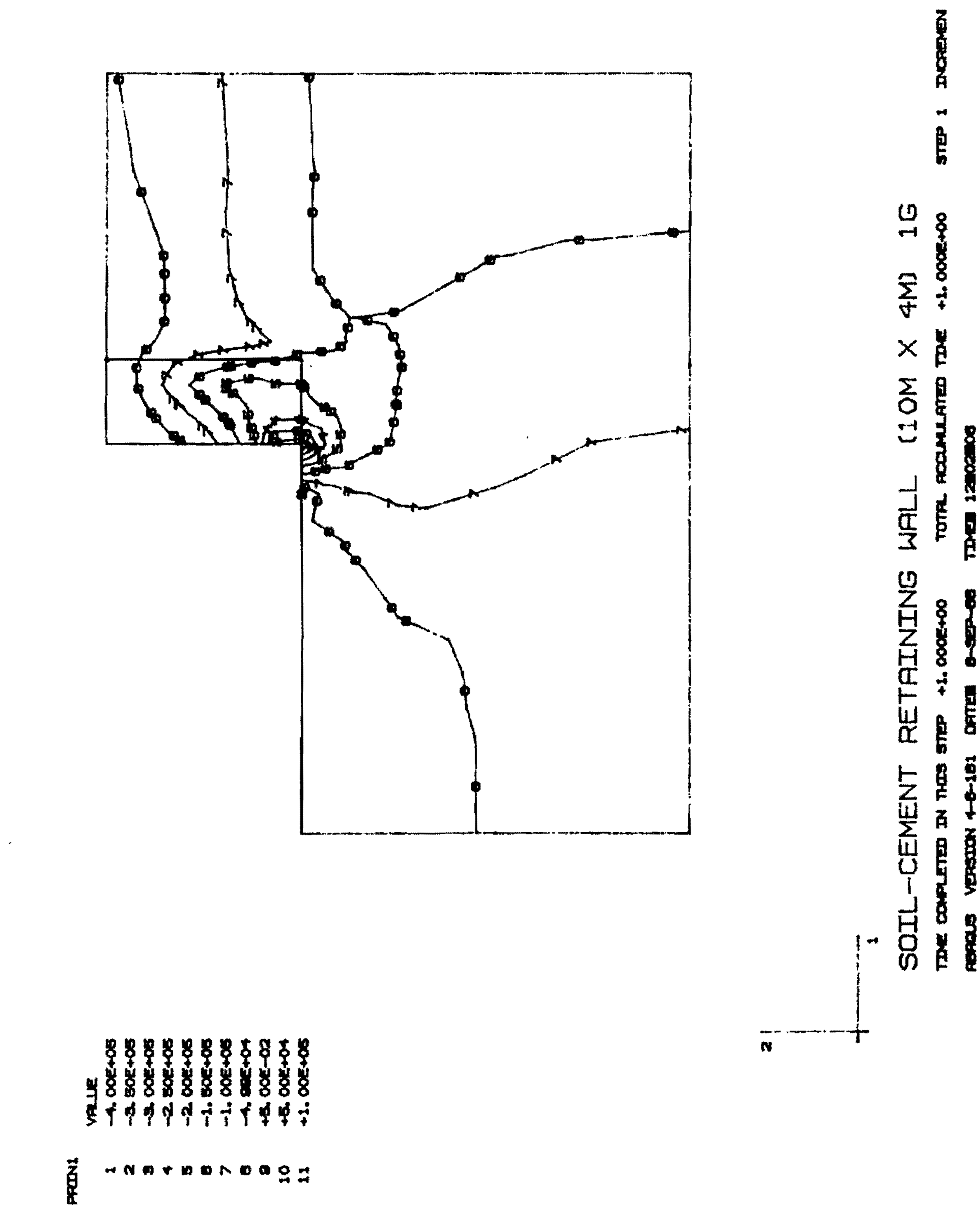


Figure 4.13 Maximum compression stresses for 40% aspect ratio

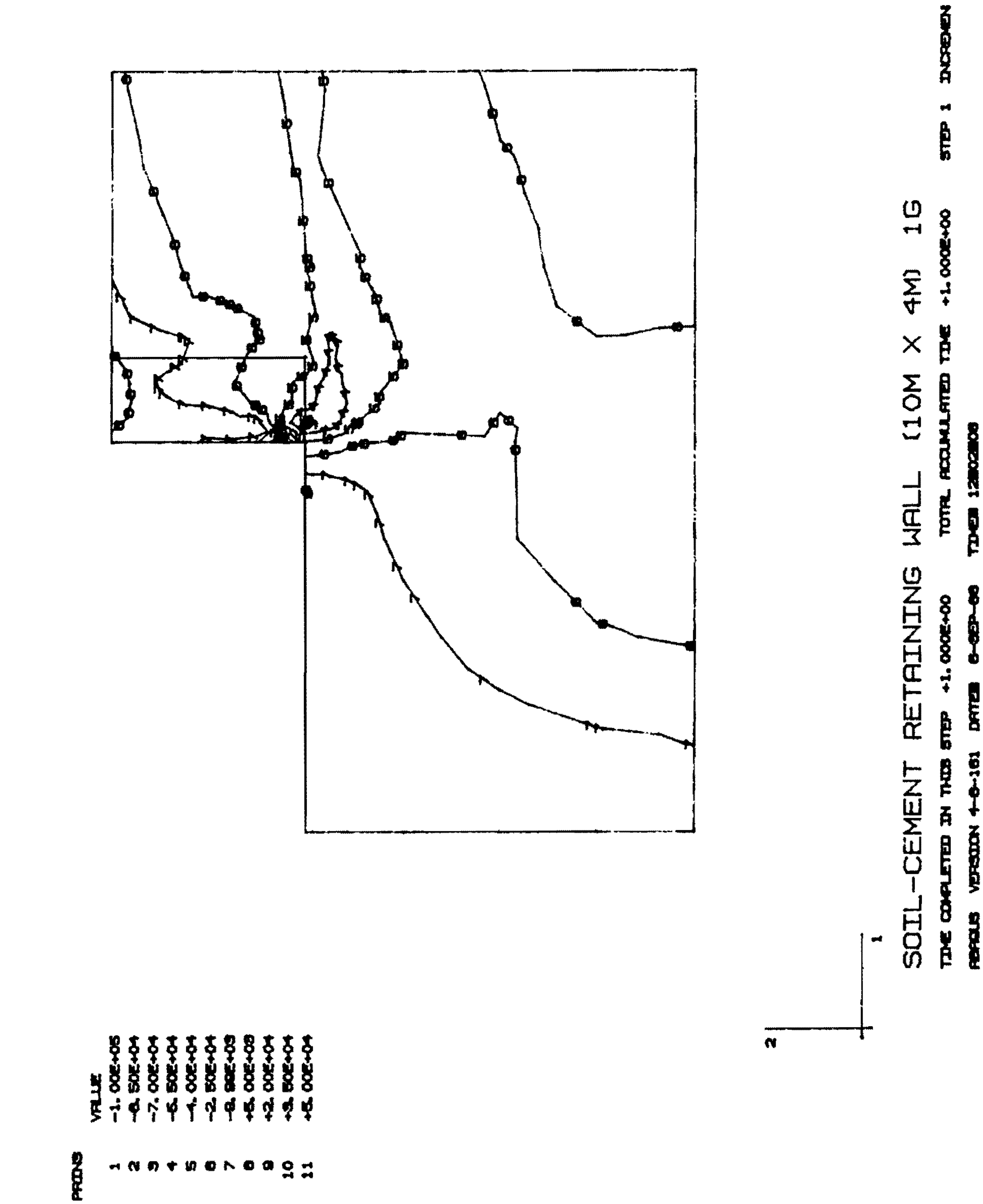


Figure 4.14 Maximum tension stresses

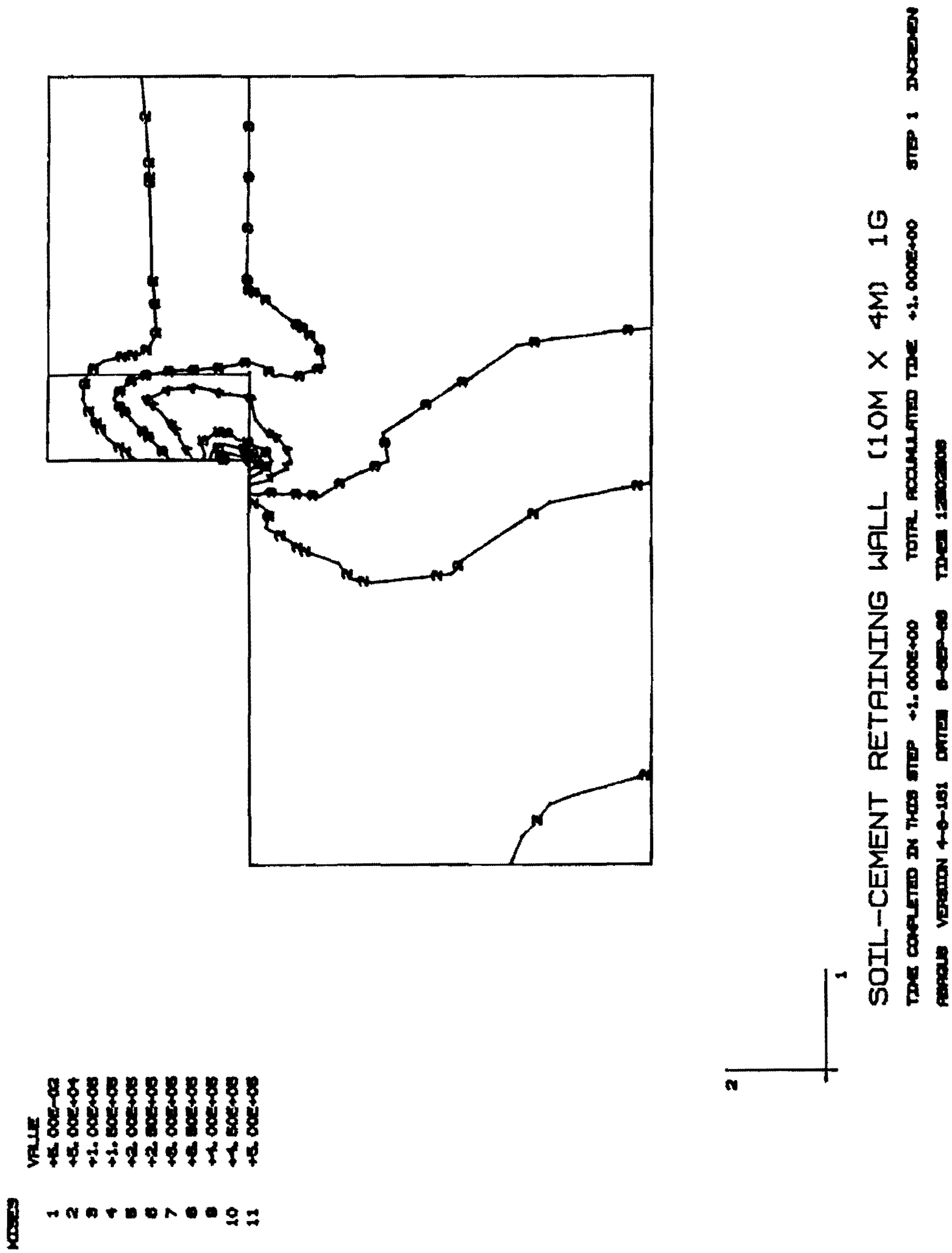


Figure 4.15 Maximum shear stresses

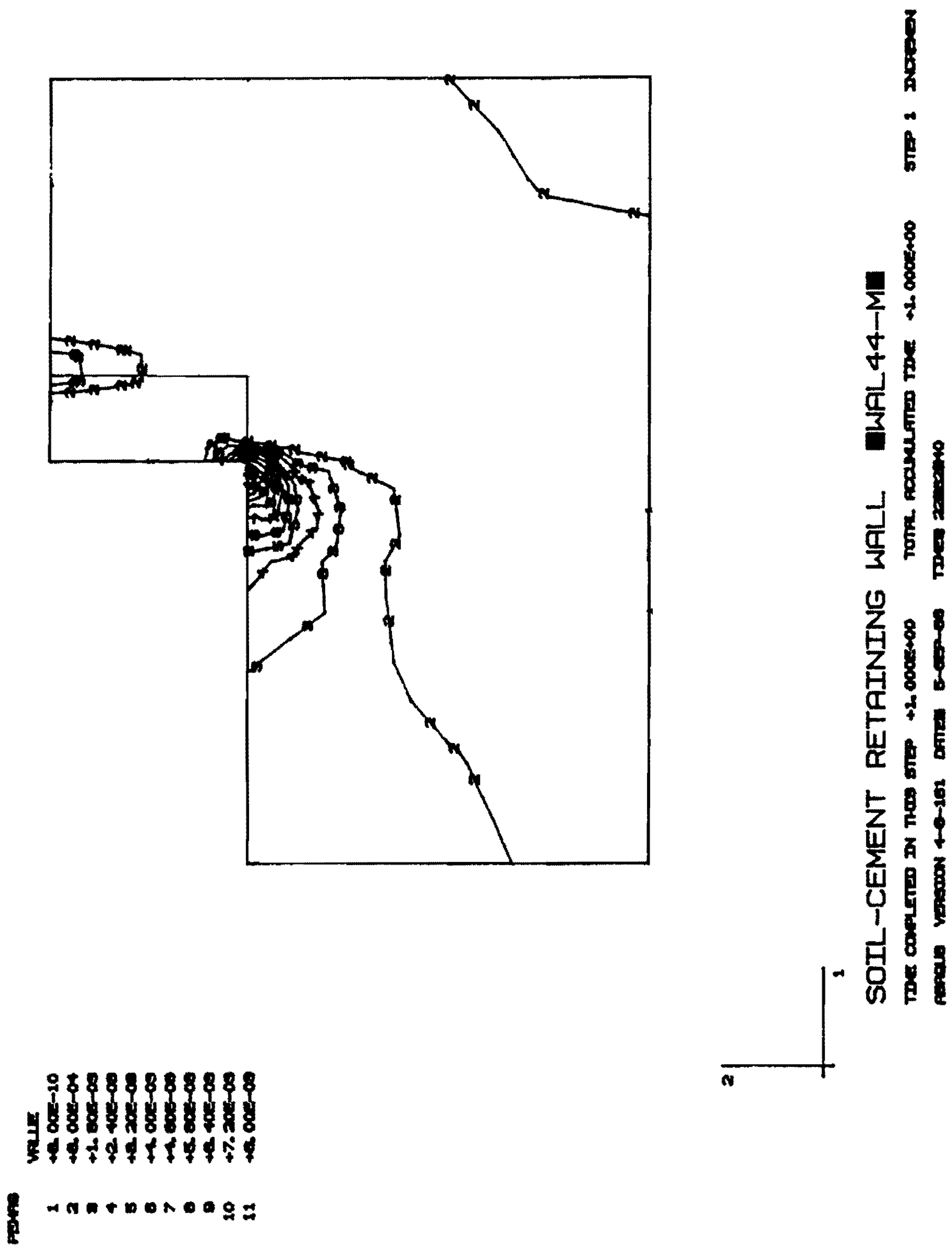


Figure 4.16 Plastic deformations

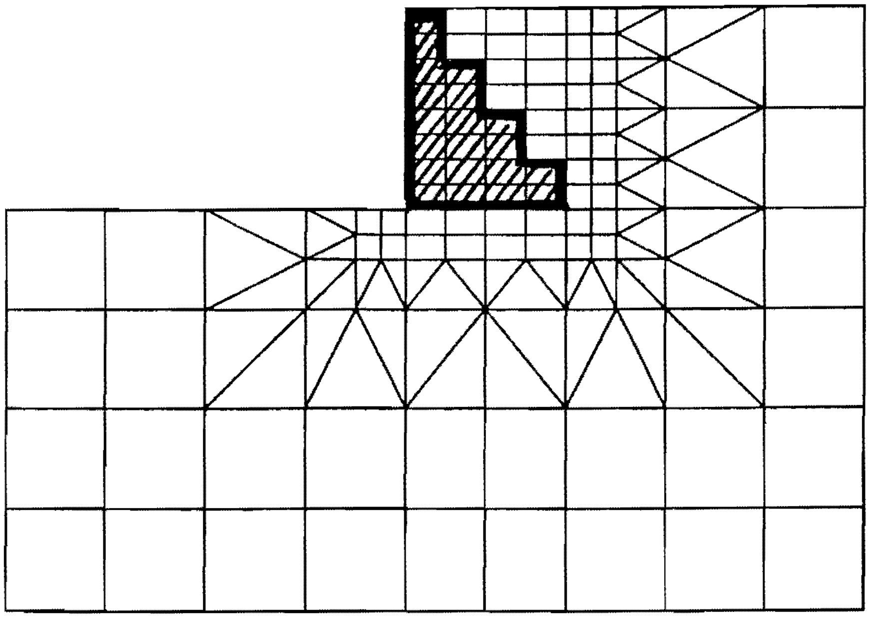


Figure 4.17 Lower triangular section

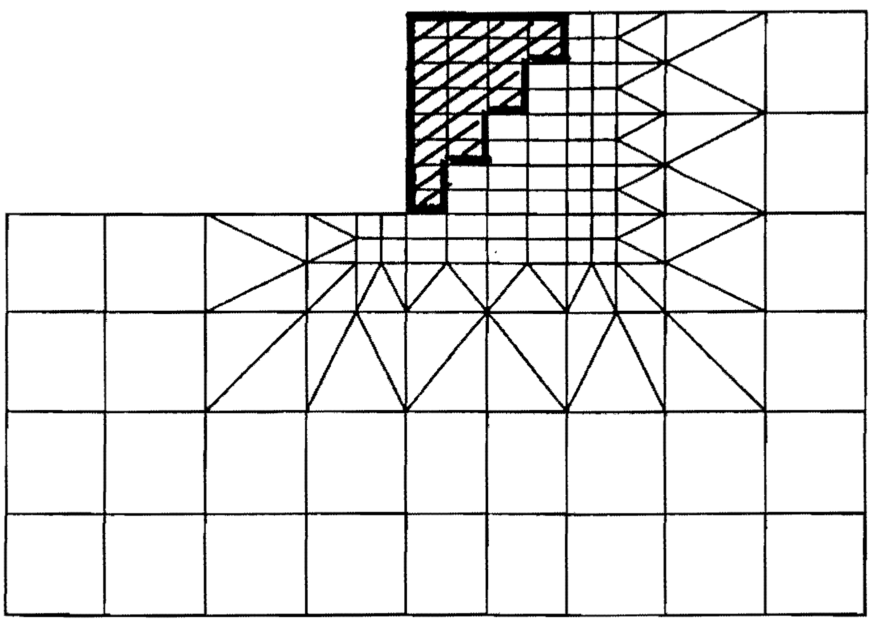


Figure 4.18 Upper triangular section

structure even under ultimate loading conditions. The ultimate failure mode of the system was predicted to be rotation of the wall around its toe, due to bearing capacity failure of the foundation soils. For the conditions given, the wall was predicted to be safe to a scaled height of 245 ft. (75 m), although this was without any safety factor, and on a granular foundation. The analysis also indicated that certain variations in the shape of the cross-section may prove to be cost-effective.

5. PHYSICAL MODELLING

5.1 Introduction

In order to provide experimental data on the performance of the retaining wall design, before full scale field tests, physical model tests were performed on the Schaevitz geotechnical centrifuge at the University of California at Davis. This has an effective radius of 43 in. (1.1 m) and can accelerate a model of about 200 lbs (100 kg) to a maximum of 150 g, although in this series of tests the highest acceleration used was 100 g. The mounting plate is allowed to swing freely on bearings at the end of the arm, so that the overall resultant acceleration always remains perpendicular to the base of the model, and a counterweight is employed to keep the rotating arm dynamically balanced. The overall principle and geometry of testing is shown in Figure 5.1.

The basis of the method is to use centrifugal acceleration to increase the equivalent gravitational acceleration to compensate for the reduction in size, when constructing a model. It is, of course, possible to increase stress levels in a model by other means (e.g., by surcharge loading), but then the distribution of stress (and consequently of strength) in incorrect. Thus, in a conventional model, it is possible either to model the magnitude of distribution of stress and strength correctly - but not both. Centrifugal modeling enables both conditions to be satisfied simultaneously.

If the linear modeling scale is defined as $n>1$, then the general stress level under a depth z, of soil of density ρ, is $z\rho g$. At the corresponding point in a centrifuge model, the linear dimension z, is decreased to z/n, and the centifugal (or quasi-gravitational) acceleration is increased to ng. The stress level at this point in the model will be the same as in the prototype. Similarly, the state of strain will be identical, if both the model and prototype are constructed of the same material. Regarding soil as a continuum with identical bulk properties in the model and the prototype, there is, in principle, no immediate way that corresponding elements of soil can distinguish between the two environments, since levels of stress, strain, pore-pressure, stress gradient, etc., are identical.

It is usually desirable to use the actual prototype soil in the model, so that the bulk material properties of a soil element are identical in both prototype and model. This is valid as long as the grain size is sufficently small, compared to the size of such an

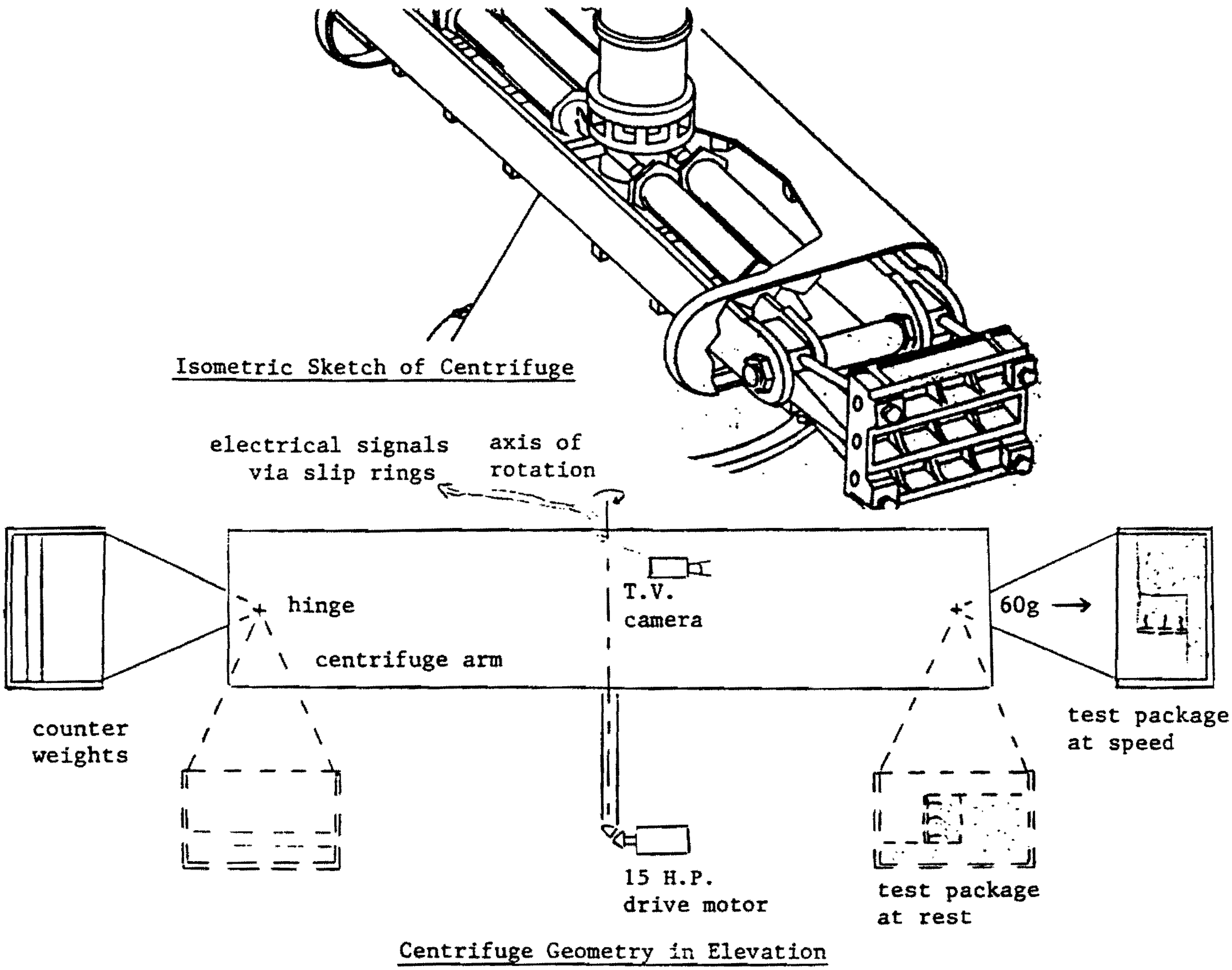


Figure 5.1 **Principle of centrifuge operation**

element, that the element can still be considered as representative of the continuum in general - in other words, as long as the grain size is much smaller than the model size. If the soil in considered as a continuum in the fashion, then it is adequate and preferable to use original material, and there is no need to "model" the soil with a reduced-particle-size powder, as might be supposed.

5.2 Tests Conducted

Models were constructed to fit inside an existing light alloy strong box supplied by the University of California, with inside dimensions 10 in. wide by 9 in. high by 16.5 in. long. A photograph of this is shown in Figure 5.2, mounted on the centrifuge.This incorporated a facing side wall of 0.92 in. thick plexiglas, which allowed photographs to be taken in cross-section both before and after tests, and also in-flight using a stroboscopic flash facility provided. Continuous visual observation in plan view was also possible with a closed circuit high resolution television camera mounted on board, with signals transmitted to an outside monitor via electrical slip rings.

Each model was made using miniature facing panels 5 in. high and 2 in. wide machined from aluminum channel. Holes were incorporated into these to allow for provision of anchors, for which purpose short miniature bolts were used, as shown in Figure 5.3. Steel forms were constructed to allow the wall to be constructed to different dimensions by compaction of stabilized soil behind these wall units. A combination of hand compaction and vibratory compaction on a vibratory table was used to achieve the desired densities. The following photographs show the overall geometry.

Instrumentation of the model was done in several ways. Displacements were measured using miniature potentiometers (shown in Figure 5.5) mounted from a gantry above and in front of the model, shown in Figure 5.4. In this way movements in all three degrees of freedom in the plane of the model could be measured - indeed some redundancy was usually provided. Anchor forces could be measured with miniature semiconductor load cells. Some miniature earth pressure cells were also borrowed from the University of California which enabled some earth pressure measurements to be taken, but these proved not to be reliable, and the results were not used.

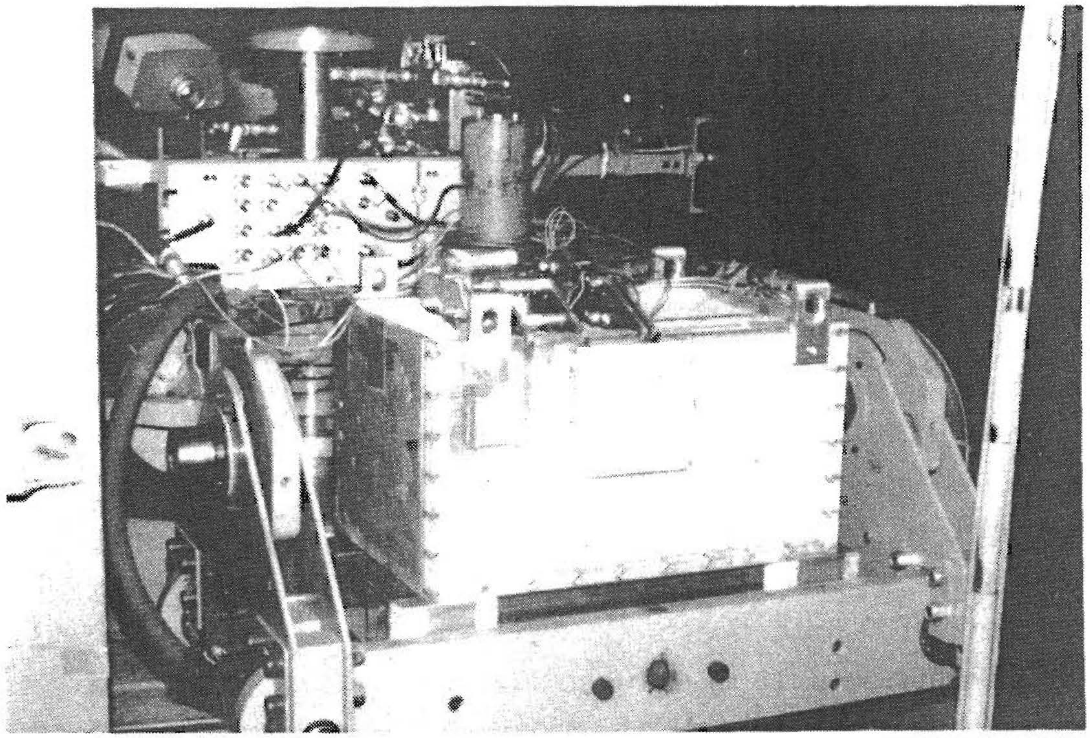


Figure 5.2 Photograph of Schaevitz centrifuge

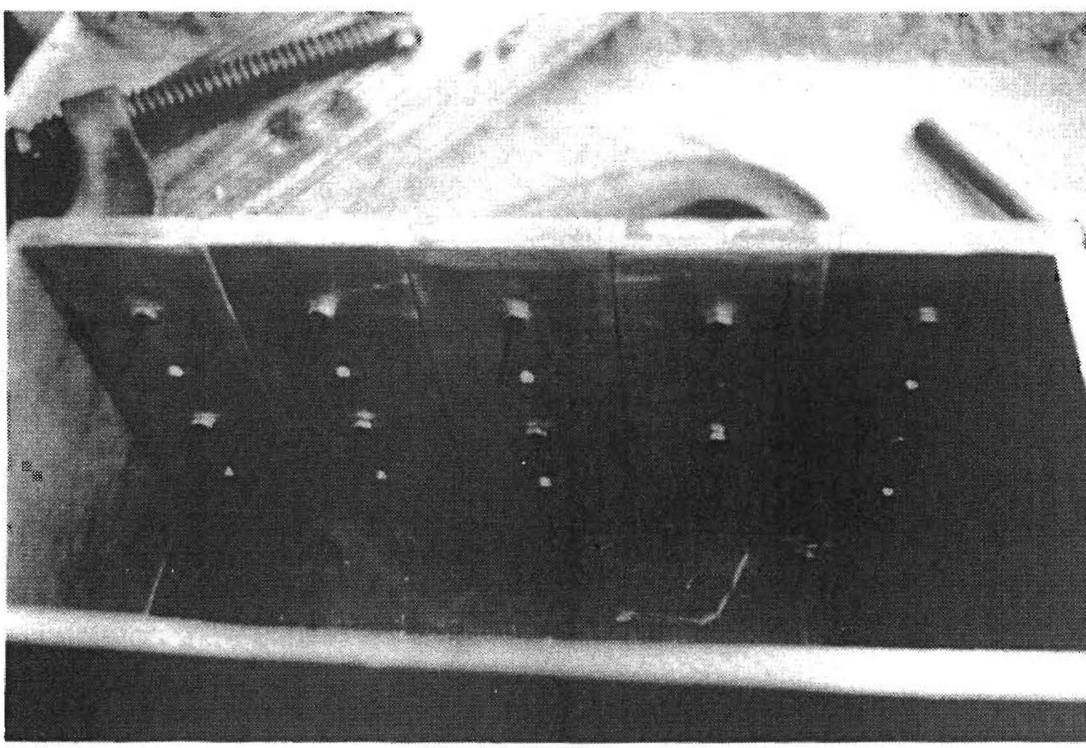


Figure 5.3 Miniature facing units and anchors

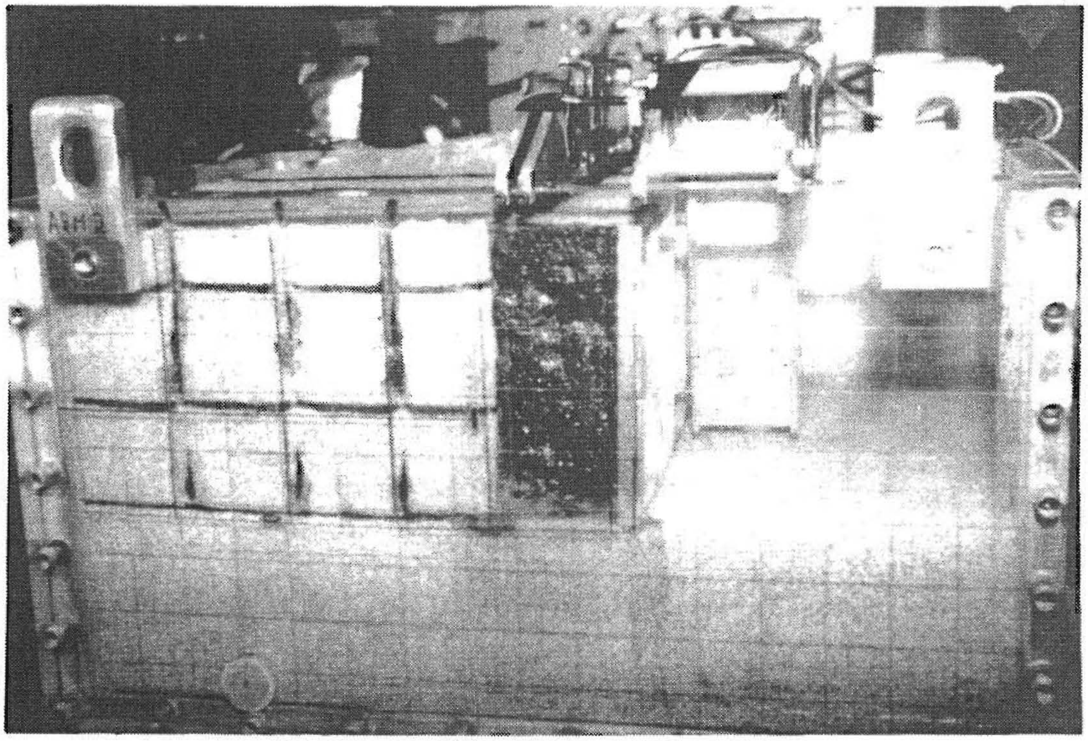


Figure 5.4 A model mounted in strong box

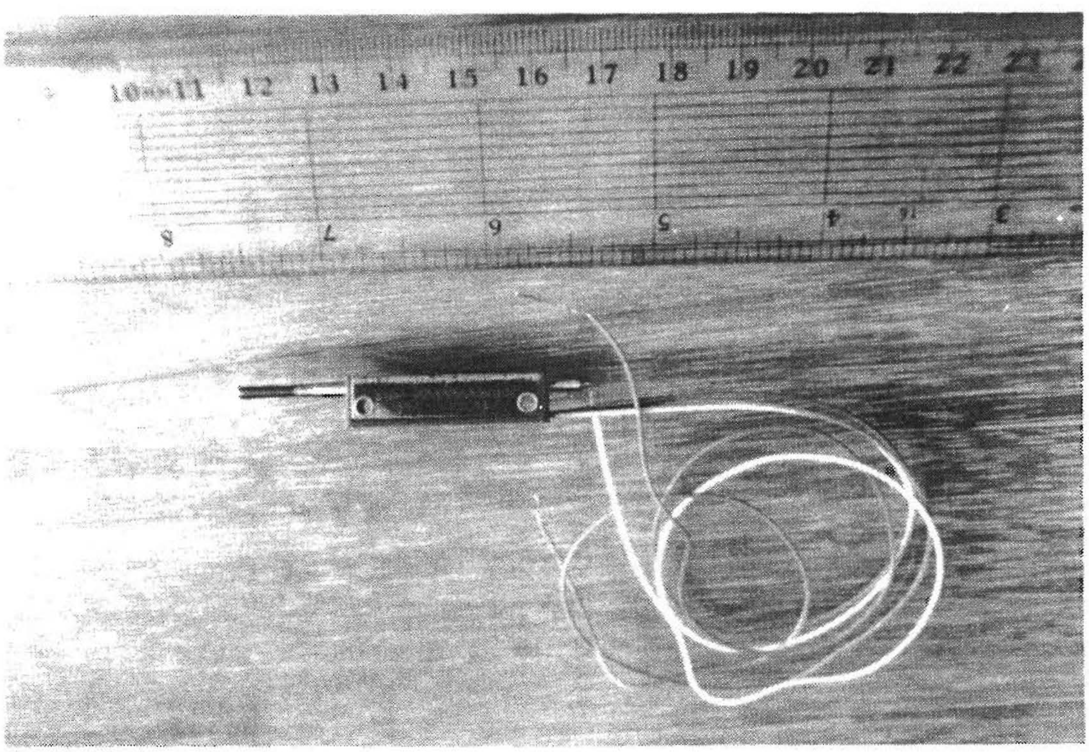


Figure 5.5 Miniature displacement transducer

The testing was divided into three groups where a variety of parameters were evaluated. Among these were: cement content, curing time, aspect ratio, anchor length, and anchor orientation. The testing concentrated on models made from granular soils (less than 10% fines) but a model of cement stabilized gypsum was also tested. The tests were designed to evaluate the failure modes of the cement stabilized backfill retaining wall with specific concentration on crushing failure, overturning failure and anchor pullout failure. A total of eight models were constructed, and from these eight models, a total of thirty-four tests were performed, as listed on Table 5.1. Figure 5.4 shows a typical test with the wall cross-section painted black against the lubricated plexiglass side of the box, ready for a 2 dimensional model test.

5.3 Test Results

In the first set of tests two models were constructed, with a cement content of 7%. The two models varied only in the aspect ratio, being 40% and 80% respectively. The two models were tested four times each, the first up to 80 g's and the second to 100 g's. The preparation of the model in the testing apparatus followed the procedures set for the test box by pluviation. The engineering properties of Monterey sand after pluviation were provided by the University of California at Davis staff. The resultant rotation versus time is shown in Figure 5.6 for the first test, which was performed up to 50 g's.

In the second set of tests three models were constructed. The first was a 30% aspect ratio model, with a cement content of 7%. The second was a model of cement stabilized gypsum, with an aspect ratio of 60%. The last model was cement stabilized sand, with a 5% cement content and a 60% aspect ratio. The small rotations observed in the first set of tests indicated that the models were stable with respect to overturning. In order to achieve failure, extra instabilities needed to be introduced. The first instability investigated was the decreased aspect ratio, and even this test resulted in relatively small displacement. Other factors introduced to decrease stability were: loose backfill (poured rather than pluviated), surcharge on the wall, and surcharge on the backfill. The only combination that resulted in failure was a surcharge on the backfill, in conjunction with a loose backfill. In this way it was possible to test the gypsum cement model to crushing failure, as follows. The combination of factors that resulted in failure were: 26.25 lbs of surcharge on the wall, loose backfill, and removal of the facing units. This model failed at 80 g's and is shown before and after failure in Figures 5.7 and 5.8. The third model was tested up to 100 g's, with no surcharging, and with pluviated backfill.

Table 5.1 Outline of centrifuge tests

TEST NAME	%CEMENT	ASPECT RATIO	CURE TIME	MAXIMUM ACCELERATION	TESTING FOR:
			(days)	(g's)	
AM1A	7	80%	7	80	stability
AM1B	7	80%	7	80	"
AM1C	7	80%	7	80	"
AM1D	7	80%	7	80	"
AM2A	7	40%	10	100	"
AM2B	7	40%	10	100	"
AM2C	7	40%	10	100	"
AM2D	7	40%	10	100	"
AM3	5	60%	8	100	crushing
AM4A	7	30%	28+	100	stability
AM4B	7	30%	28+	100	"
AM5	7 (gypsum)	60%	10	100	crushing
AM5A	"	60%	10	100	"
AM5B	"	60%	10	100	"
AM5C	"	60%	10	100	"
AM5D	"	60%	10	80	"
AM6	7	30%	28+	5	stability
AM6B	7	30%	28+	100	"
AM6C	7	30%	28+	100	"
AM7A	5	60%	28+	80	crushing
AM7B	5	60%	28+	80	"
AM7C	5	60%	28+	80	"
AM7D	5	60%	28+	80	"
AM7E	5	60%	28+	80	"
AM7F	5	60%	28+	80	"
AM7G	5	60%	28+	80	"
AM7H	5	60%	28+	80	"
AM8A	5	60%	8	100	forces
AM8B	5	60%	8	100	"
AM8C	5	60%	8	100	"
AM9A	5	60%	7	100	"
AM9B	5	60%	7	100	"
AM9C	5	60%	7	100	"

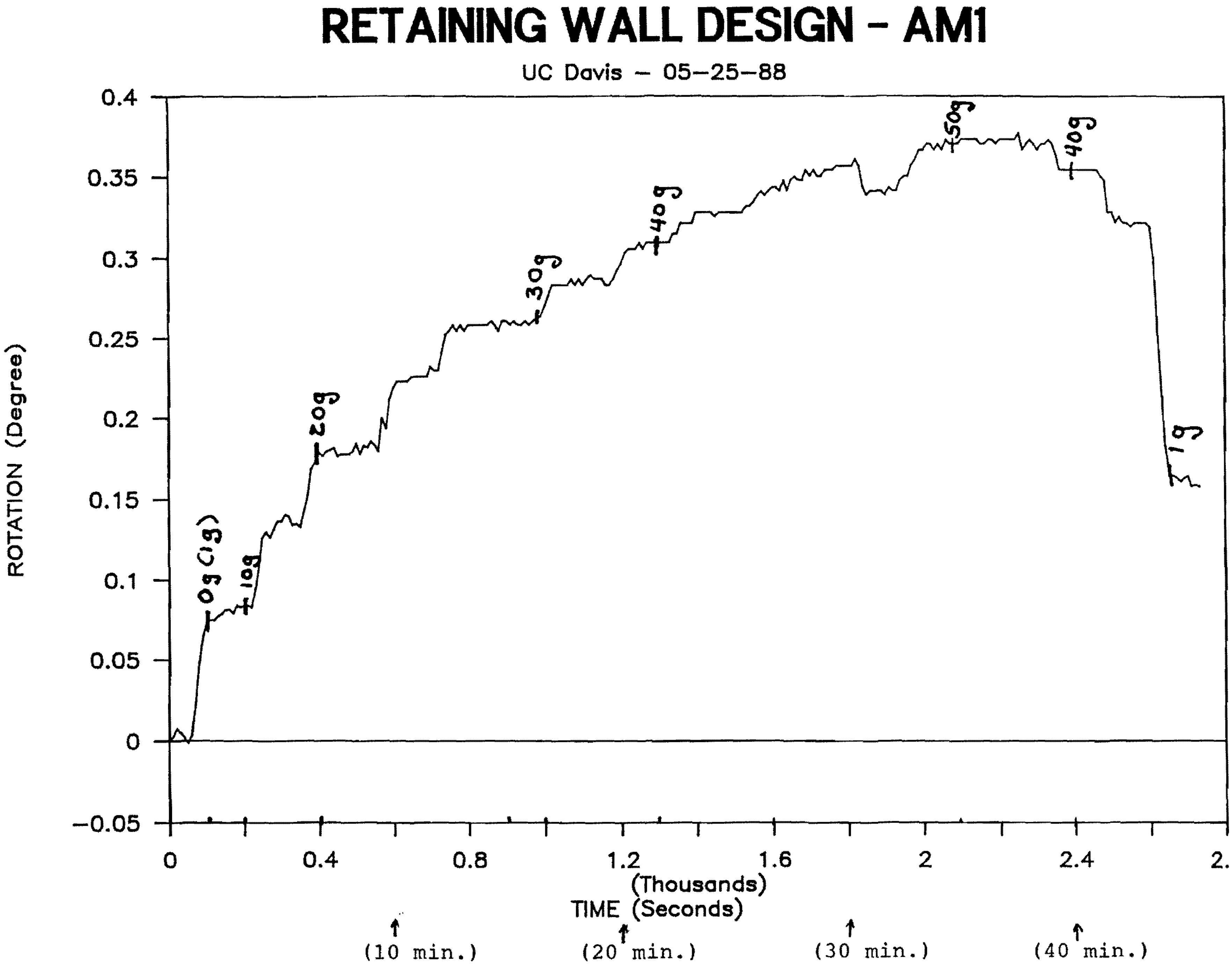


Figure 5.6 Typical model rotation measurements

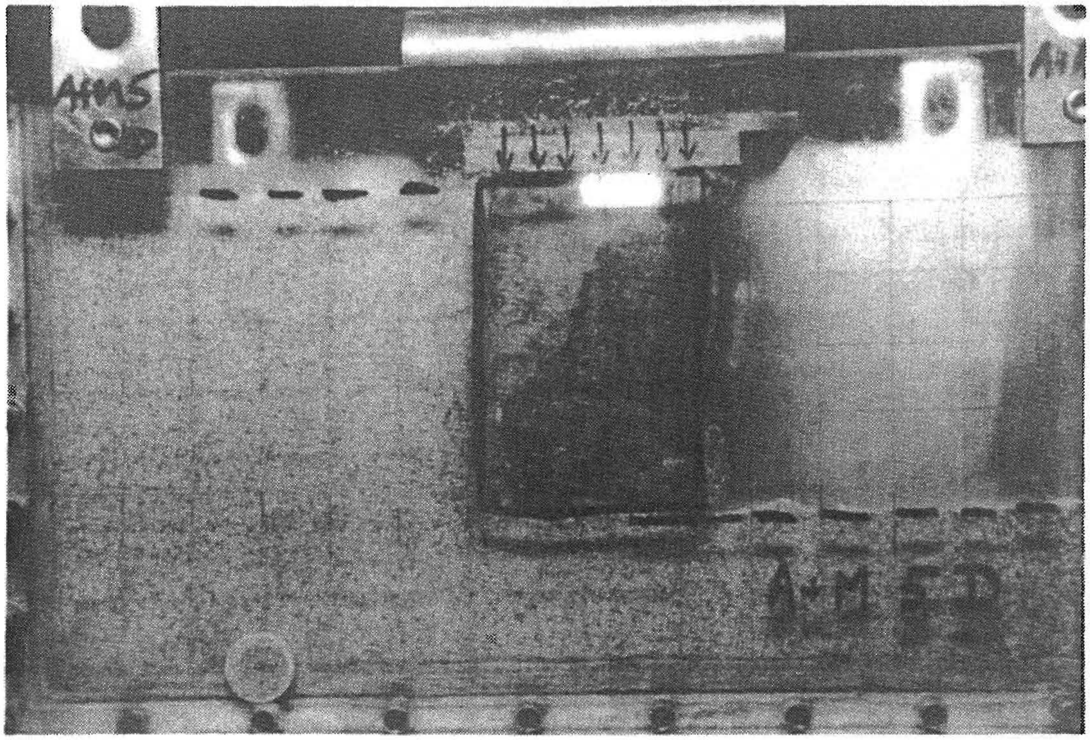


Figure 5.7 Initial conditions - test AM 5D

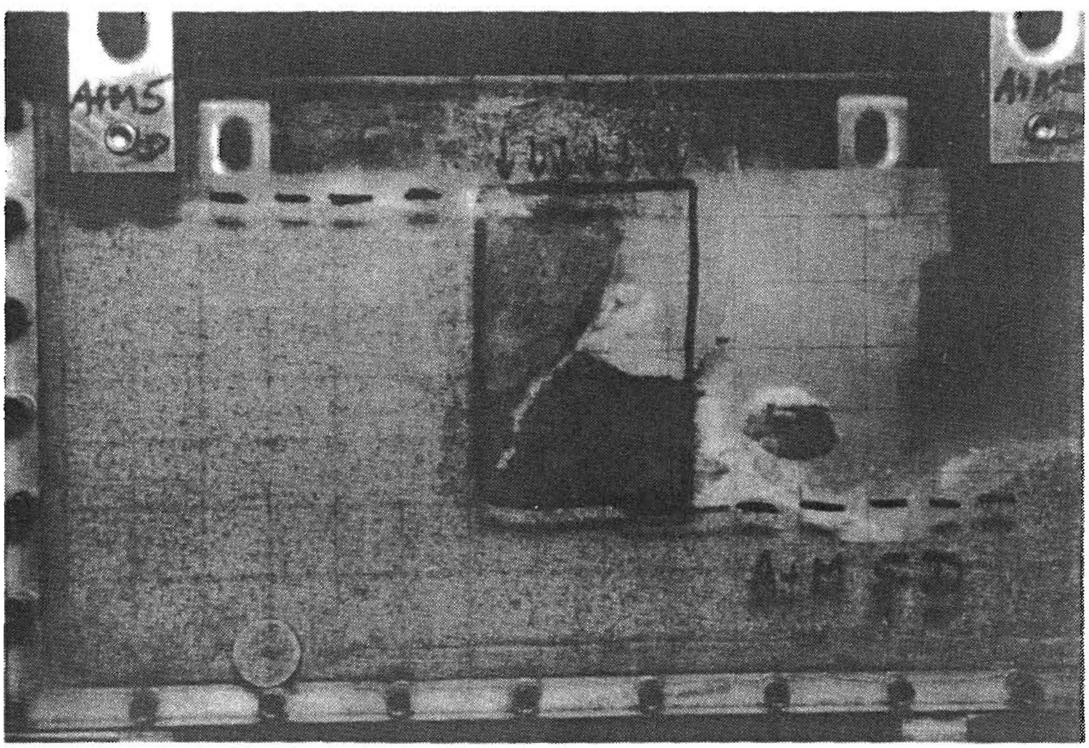


Figure 5.8 Crushing failure - testAM 5D

The model showed little rotation, and relatively small movements. The motion of the wall was 0.5 mm at 100 g's, which corresponds to a full scale motion 1 in., or 0.4% of the height of the wall.

In the third set of tests three models were tested, the first being the 5% cement content model from the previous set of tests. The other two models were 5% cement walls with an aspect ratio of 60% also, but with load cells attached to the middle anchor. In the first tests the model was tested to failure by surcharging the wall with 29.5 lbs (13 kg) of dead weight with the center of gravity of the surcharge at a position 33% of the wall width from the front. This was equivalent to a doubling of the stresses at the toe of the wall. The wall failed in crushing at the toe, at an acceleration of 60 g's, which was equivalent to a 93 foot high wall at full scale.

5.4 Anchor Forces

The next two sets of tests (AM 8 and AM 9) were performed to estimate the forces exerted on the anchors by the facing units. This was done by rigidly attaching load cells to the facing plates, and connecting the measuring lever arm of the load cell to the free end of the anchor. The difference between the two models (8 and 9) was the orientation of the anchor on the face plates. In the first model the anchor was at 60% of the wall height above the toe.

In each test three configurations were tested. The first (denoted A) was with all the face plates bolted together, but with anchors still in place in the other face plates. The second (denoted B) was with the other anchors removed. The third (denoted C) was with all the other face plates removed. The worst case was obviously the second configuration, and in this test the highest forces were measured. The actual anchor forces measured are shown in Figure 5.9 (tension being negative). At 100 g's a force of 7.5 gm was measured, which corresponds to a full scale model force of 165 lbs (0.75 kN.).

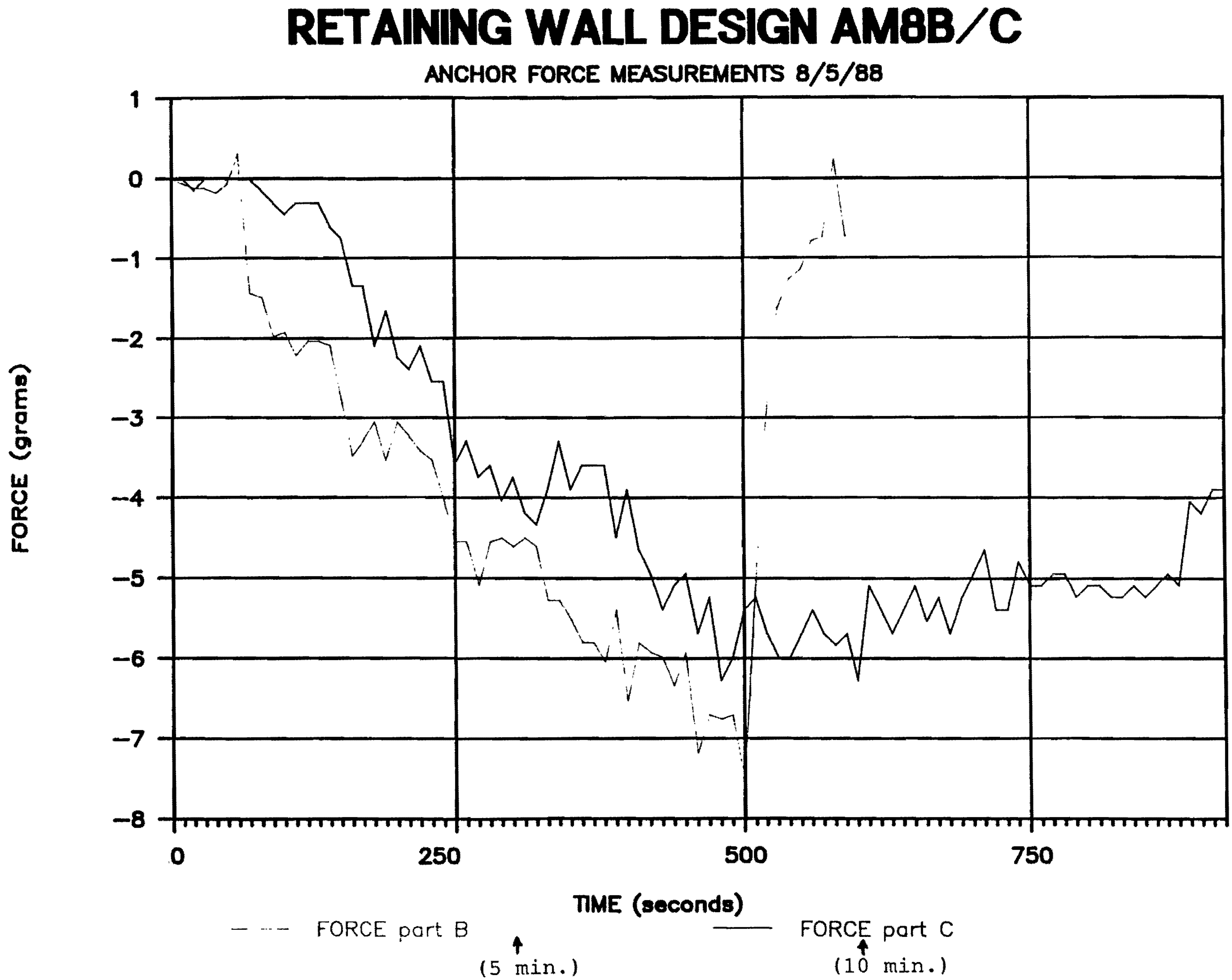


Figure 5.9 Typical model anchor force measurements

5.5 Conclusions

For typical wall geometries and properties, measured movements were small (seldom exceeding a scaled value of a few centimeters), and earth pressures were as indicated by standard theory. Forces in the anchors were also small, typically of the order of 200 lbs (1 kN) full scale equivalent. In general,observed behavior of the proposed design appeared to be satisfactory, as far as could be concluded from such physical model testing.

6 FULL SCALE EXPERIMENTAL CONSTRUCTION

6.1 General

In order to construct a full-scale section of wall to the experimental design, it was felt to be most economical to do this as part of ongoing TxDOT work.

In conjunction with District 12 and the Bridge Division, potential sites for the full-scale field test section were identified, and the optimum site recommended as the Cypress/Fairbanks bypass of Highway 290 north-west Houston. A total of 10 retaining walls were to be constructed at this site, of varying heights and sizes.

The walls on the North-East side of the 290 were suggested as more suitable than those on the South-West side, because they will be more protected from traffic and diversions on the existing road during construction, access may be somewhat easier after construction, and they include also the highest section of wall in this location.

The optimum compromise was therefore felt to be walls no. 7 and 9 to be built in this design. These are walls of 16 ft. and 23 ft. height in their maximum cross-section, which were high enough to be significant, without having too extensive an experimental section. These locations are shown on Figures 6.1 and 6.2. Appendix A shows a detailed layout of the field project.

6.2 Design Procedures/Specifications

Specifications for the letting of the experimental full-scale retaining walls (walls 7 & 9) were completed in conjunction with the bridge division, and incorporated into the bid documents for the April 1989 letting of the Cypress/Fairbanks bypass.

Draft material specifications were prepared for the stabilized fill. This included both materials specifications and recommendations for possible auxiliary test procedures for sulfates and pH. to address any corrosion concerns. Several features of the existing Reinforced Earth specifications and Texas Specifications were retained-in particular Items 274 and 432. The format is also similar, as far as possible. Some AASHTO and ASTM test procedures were referred to, in the absence of appropriate Texas Specifications, as well as some auxiliary test procedures. These are in detail as follows:

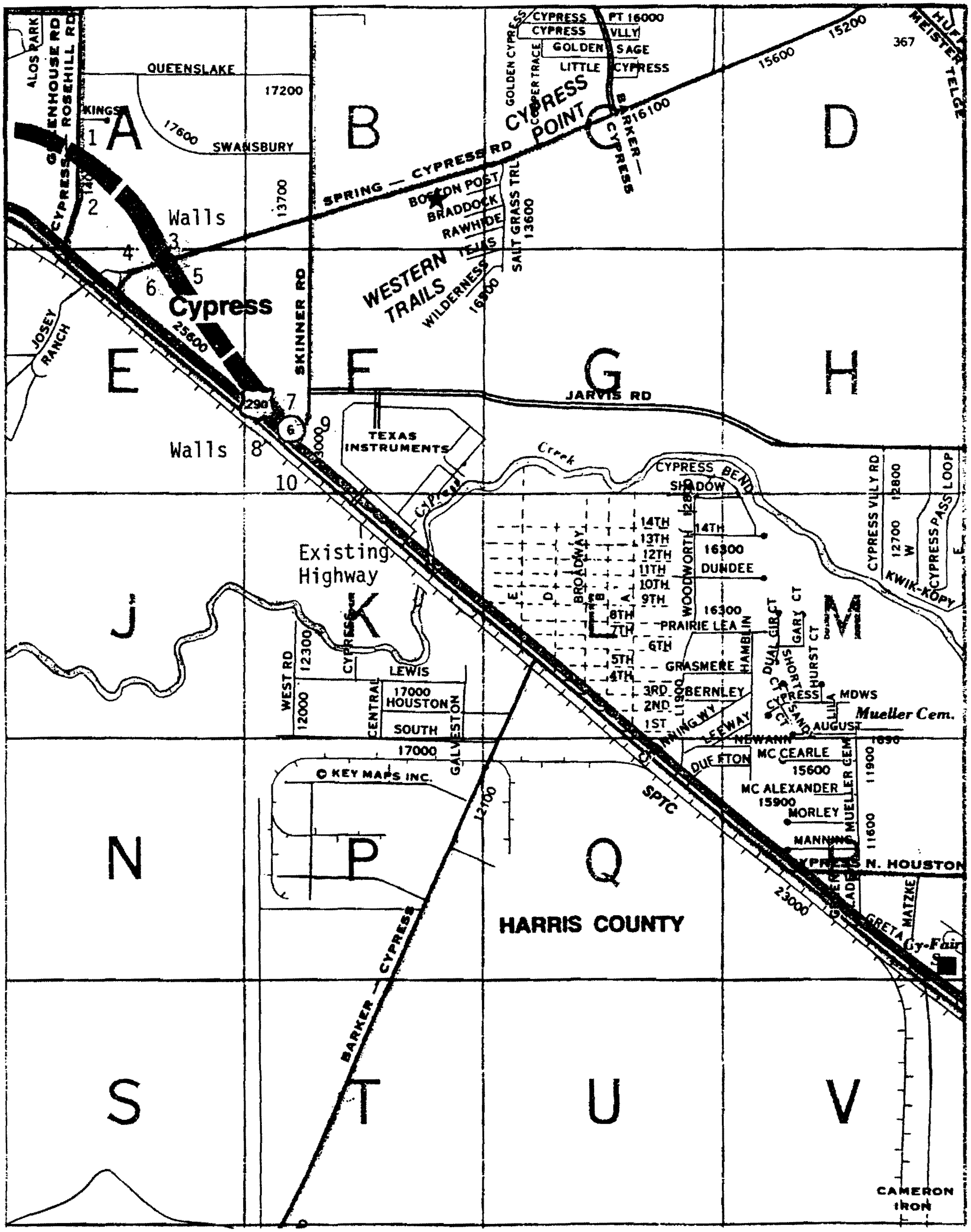


Figure 6.1 Location map of field construction

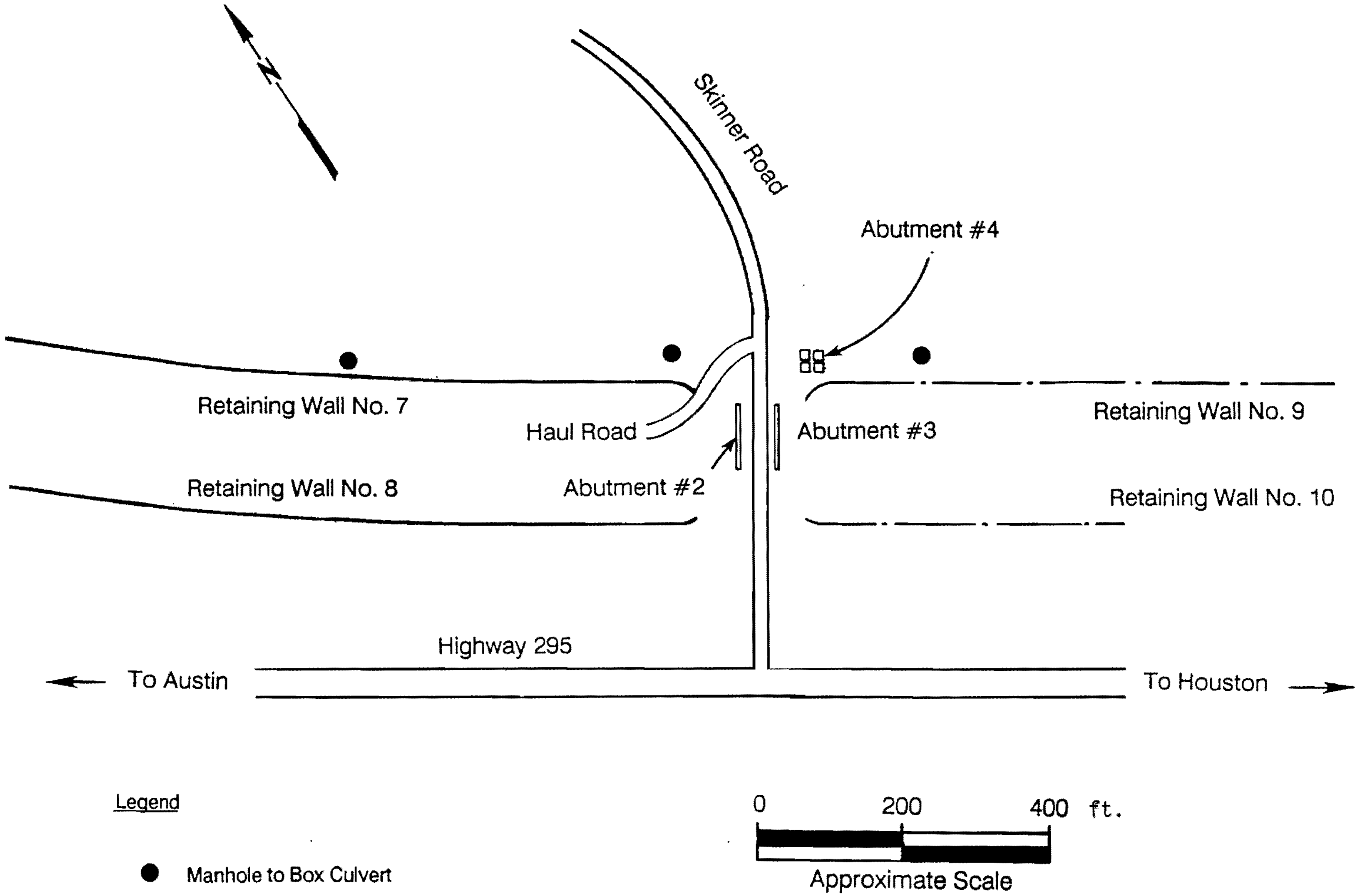


Figure 6.2 Site layout

SPECIAL SPECIFICATION

CEMENT STABILIZED SOIL RETAINING WALLS

Desciption. This item shall govern for the construction of Cement Stabilized Soil Retaining Walls in accordance with these specifications and with the lines, grades and dimensions shown on the plans.

The Contractor shall make arrangements with either The Reinforced Earth Company or The VSL Corporation to provide facing panels conforming to the requirements of other retaining walls on this project.

The Cement Stabilized Soil Retaining Walls are designated as experimental. Instrumentation will be installed and monitored by research personnel. The Contractor shall cooperate with the researchers and allow them access to the retaining walls during construction, and after the walls are complete. In addition, the Contractor shall insure that instrumentation is not damaged or disturbed during subsequent.

Working Drawings. Prior to fabrication, the Contractor shall submit to the Engineer 7 sets of shop drawings. These drawings shall include a numbered panel layout for fabrication and erection purposes, as well as for any required coping when it is prefabricated. They shall further include the horizontal and vertical alignment of the walls, as well as the existing and proposed ground lines, all as shown in the contract plans. The shop drawings shall also reflect all information needed to fabricate and erect the walls including the proposed leveling pad elevations; the shape and dimensions of panels; the size, number and details of the reinforcing steel; the number, size, type and details of the facing anchorage, the size of leveling pad, and any additional details necessary pertaining to coping, railing, drainage or electrical conduit as required by the contract plans. Leveling pad elevations may vary from footing elevations shown on plans. However, the leveling pad elevations shall be such as to allow for transverse and longitudinal drainage structures shown on the plans. Unless otherwise noted on the plans, one foot minimum cover shall be provided from the top of the leveling pad to finish grade.

Materials. Materials for face panels and attachment hardware shall conform to the requirements of the Item "Reinforced Earth Walls" or "Retained Earth Walls".

Backfill Material.

1. All backfill material used in the structure volume shall be free from organic or otherwise deleterious materials, and shall conform to the following gradation limits as determined by Test Method Tex-110-E.

Sieve Size	Percent Passing
3 inches	100
No. 4	75 - 100
No. 200	0 - 15

2. The Resistivity shall be 1,500 ohms-cm or greater as determined by Test Method Tex-129-E.

3. The pH shall be from 5.0 to 10.0 as determined by Test Method Tex-128-E.

4. The Plasticity Index (P.I.) As determined by Test Method Tex-106-E shall not exceed 6.

Backfill material shall be stabilized with cement. Cement content shall not be less than 6 percent of the dry weight of the soil. The minimum design strength of the cement stabilized soil shall be 200 psi as determined by Test Method Tex-120-E.

Testing and Inspection. Testing and Inspection shall conform to the requirements of the Item "Reinforced Earth Walls" or "Retained Earth Walls".

Construction Methods. Construction Methods shall conform to the requirements of the Item "Reinforced Earth Walls" or "Retained Earth Walls", except that references to reinforcing strips or mesh shall apply to facing attachments, and except as shown below.

Not more than 60 minutes shall elapse between the start of mixing and the time of starting the compaction of the cement stabilized soil. Any mixture of aggregate, cement, and water that has not been compacted shall not be left undisturbed for more than 30 minutes, and the compaction shall be completed within 2 hours of the time water is added to the mixture.

Compaction shall be accomplished by use of tamping rollers or other equipment which will leave a surface with at least 1" of relief. The purpose of the rough surface shall be to allow subsequent lifts of backfill to interlock with previously placed material.

If in the opinion of the Engineer, excessive loose material exists on a previously placed lift of material, the Contractor will be required to sweep or wash the surface before the next lift is placed.

External bracing of the facing panels will be required during construction. The bracing shall be sufficient to maintain the alignment of the panels during backfilling and compaction. The bracing will be left in place for a minimum of 2 days after placement of backfill, or as approved by the Engineer.

Measurement. "Cement Stabilized Soil Retaining Walls" will not be measured for payment but will subsidiary to the Item, "Retaining Wall".

No measurement will be made of face panels, connection hardware, fasteners, joint filler, filter material, excavation in back of retaining walls and for footings, backfill, leveling pads and other incidentals; all these will be considered subsidiary.

Payment. The work performed and materials furnished will be paid for at the unit price bid per square foot for "Retaining Walls" measured as specified in that item, as shown in the plans.

The quantity for which payment is made will be that quantity shown on the contract plans and in the proposal, regardless of errors in calculation, except as may be modified by the following:

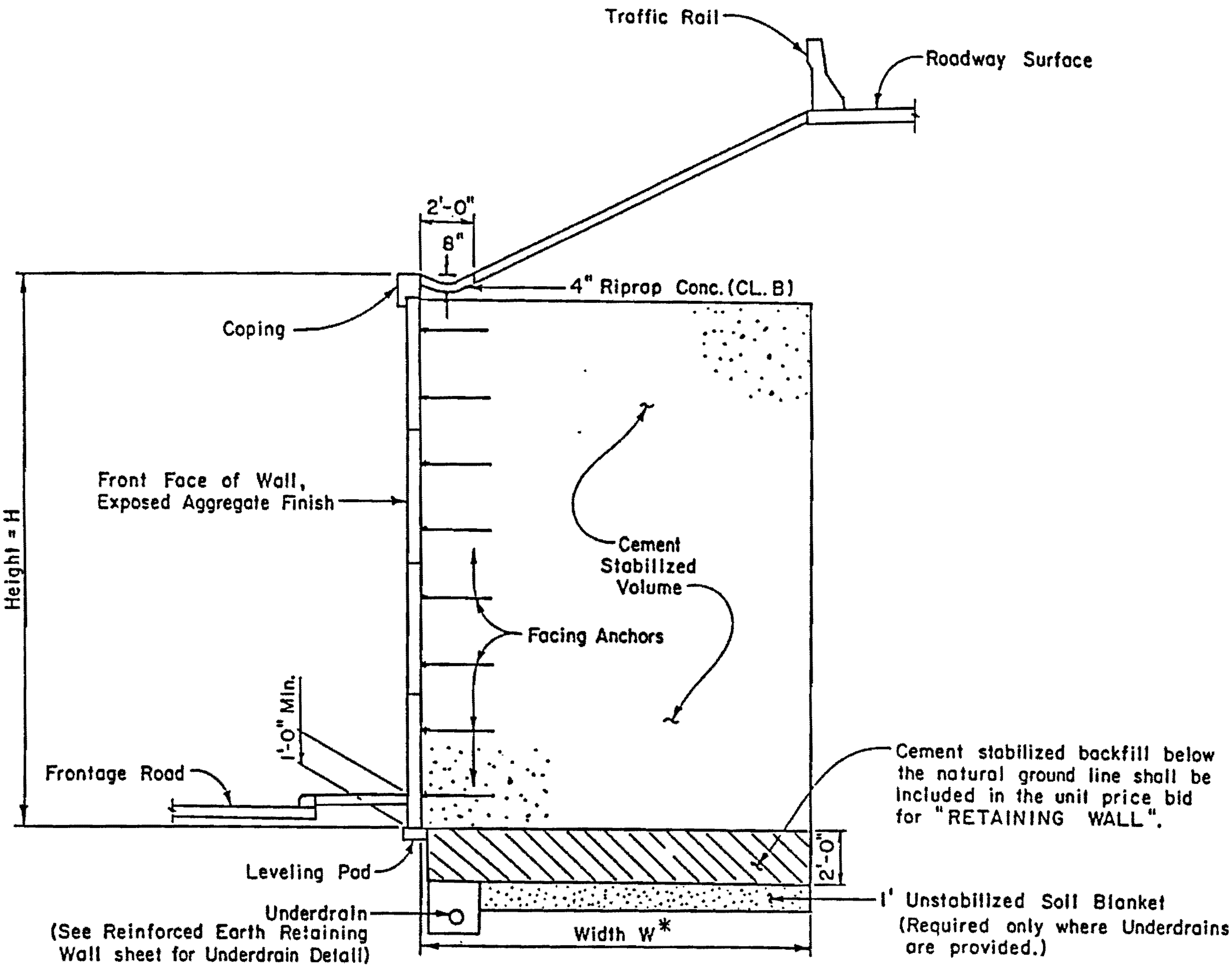
(1) Either party to the contract may request an adjustment of the quantities shown on the contract plans if the area of a single retaining wall is in error by 5 percent or more, or by the incorrect inclusion or omission of one or more complete wall units.

The party to the contract requesting the adjustment shall present to the other, two copies of the description and location, together with calculations of the quantity for the structure involved. When this quantity is certified correct by the Engineer, it will become the revised plan quantity. Quantities revised in this manner will not be subject to the provisions of Standard Specification Article 4.5.

(2) When the wall area only is revised by a change in design, the plan quantity will be increased or decreased by the amount involved in the design change. Quantities revised in this manner will be subject to the provisions of Standard Specification Article 4.5.

This unit price bid shall be full compensation for furnishing, hauling, and mixing all concrete materials; fabricating, curing and finishing all panels; furnishing concrete for foundations; furnishing and placing reinforcing strips, tie strips, fasteners, joint fillers, filter material and incidentals; for all reinforcing steel; for all excavation in back of retaining walls and for footings; for all backfill (except as shown below), including select backfill; for all labor and materials required to construct the concrete leveling pads to the line and grades as shown on the plans; wall erection; sprinkling and rolling for granular backfill materials; for furnishing and placing all temporary shoring, including soldier shafts or piling; and for all labor, tools equipment and incidentals necessary to complete the work.

When Item 131, "Borrow (Delivered)", is used and Class 3 measurement is specified, or when the Special Specification Item, "Embankment", is used, retaining-wall-backfill areas which are also in embankment areas will be measured for payment as "Borrow (Delivered) (Class 3)" or "Embankment", respectively, of the compaction method specified, as applicable. Such backfill material shall meet the requirements of this item.



TYPICAL SECTION

(Wall at bottom of slope)

*See Retaining Wall Layout for width of Cement Stabilized Soil Block.

Figure 6.3 Typical section detail for experimental wall

The typical section detail used for the initial design is shown in Figure 6.3. Note that an underdrain is included just underneath the toe of the wall. At the time this was a standard detail for Reinforced Earth and Retained Earth walls in Texas, although the Texas DOT has now revised this detail. From the point of view of groundwater control, such a drain is undoubtedly beneficial, but can give structural problems, as is discussed in the next two sections. On this project the underdrain was eventually deleted in favor of a much thicker sand layer, and this has proved to be satisfactory both for foundation support and (apparently) for such underdrainage as occurs. This point is discussed in more detail in Section 8.6.

6.3 Field Construction at Test Site

The CyFair bypass was let 27 April 1989, to Williams Brothers Construction of Houston for $25,119,788 (the pre-bid estimate was $30 million). The retaining wall subcontract was to Baytex Construction in VSL for $2,296,028, which bid $1,717,364 for the 78,062 ft^2 of non-stabilized wall (or 22 $\$/ft^2$) and $578,664 for the 24,111 ft^2 of stabilized wall (or 24 $\$/ft^2$, apparently just factoring in the cost of the extra cement).

Their source of borrow material was at the South-East, and they commenced working towards it, from West to East, so that work started with walls 1 and 2, and finished with walls 9 and 10. Construction of the first cement stabilized soil retaining wall (wall 7) was commenced in October 1989 at the Cypress-Fairbanks 290 bypass by the contractor (Baytex Construction) as part of CSJ 0050-06-003. Site supervision was provided by District 12 (Houston) personnel. Surveying control was provided by the contractor. Initial foundation preparation for wall 7 is shown in Figure 6.4.

VSL Retained Earth also agreed to cast suitably sized (4 in. diameter) holes into the lowest middle panels. Some discussion ensued on the matter of providing a bond between successive lifts of cement stabilized fill, and it was agreed that this would be by earth moving equipment. The short anchors used are shown in Figure 6.5.

Figure 6.4 Foundation preparation for wall 7

Figure 6.5 Facing panel anchors at wall 7

7. FIELD PROBLEMS

7.1 General

In the evening of October 31, 1989, several inches of rain fell on the site, and the next morning considerable distress was noted at the experimental wall section, which at this stage had reached a height of about 10 feet. Both T.T.I. and TxDOT staff met at the site on November 2, and at one location, a wall section had bulged considerably, and at another, significant separation had occurred between some facing units and the stabilized fill. This is shown in Figure 7.1, and in Figure 7.2 after some of the panels had been removed by a crane. On examination, massive settlement (over 1 foot) of the levelling pad had taken place as shown in Figure 7.3, (although the panels had remained anchored into the fill), some separation of the fill had occurred at lift joints, and the stabilized fill visible appeared very weak (as shown again in Figure 7.2).

Considerable effort and discussion and planning was undertaken as to how best to expedite progress. Laboratory testing on samples taken from the failed section indicated that compressive strengths were much lower than specified. Several other difficulties were identified, including poor site drainage, and the existence of a loosely backfilled drainage culvert below the front of the retaining wall toe.

The really major distress appeared to be associated primarily with massive settlement of a foot or more, at what appeared to be the location of an underground storm drainage culvert. By all accounts, this had been loosely backfilled with clayey fill, (possibly even sluiced into position), with no engineering control. A close up of this is shown in Figure 7.4.

Site drainage was originally much worse at wall 7 than at wall 8. This was true not only for the base of the wall, but also for the top, since the stabilized fill did not drain as easily - aggravating water pressures behind the facing units. There appeared to be considerable evidence of surface runoff (and mud) over the top of the stabilized fill and down behind the facing units. Softening of the foundation at the toe therefore probably also took place.

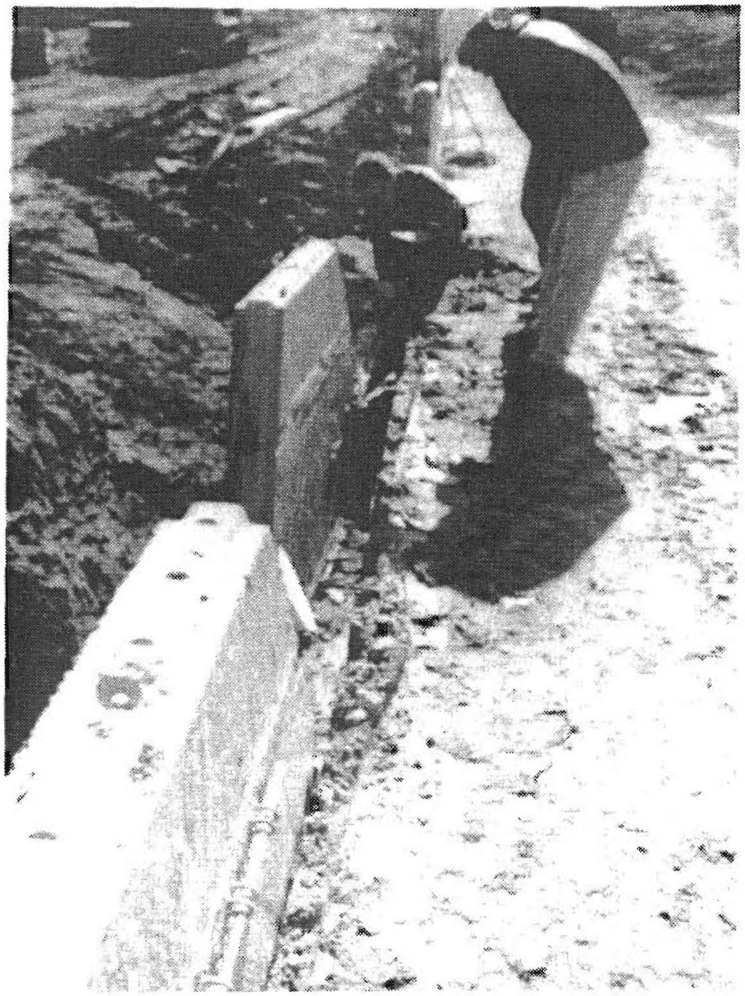


Figure 7.1 Initial separation of facing units

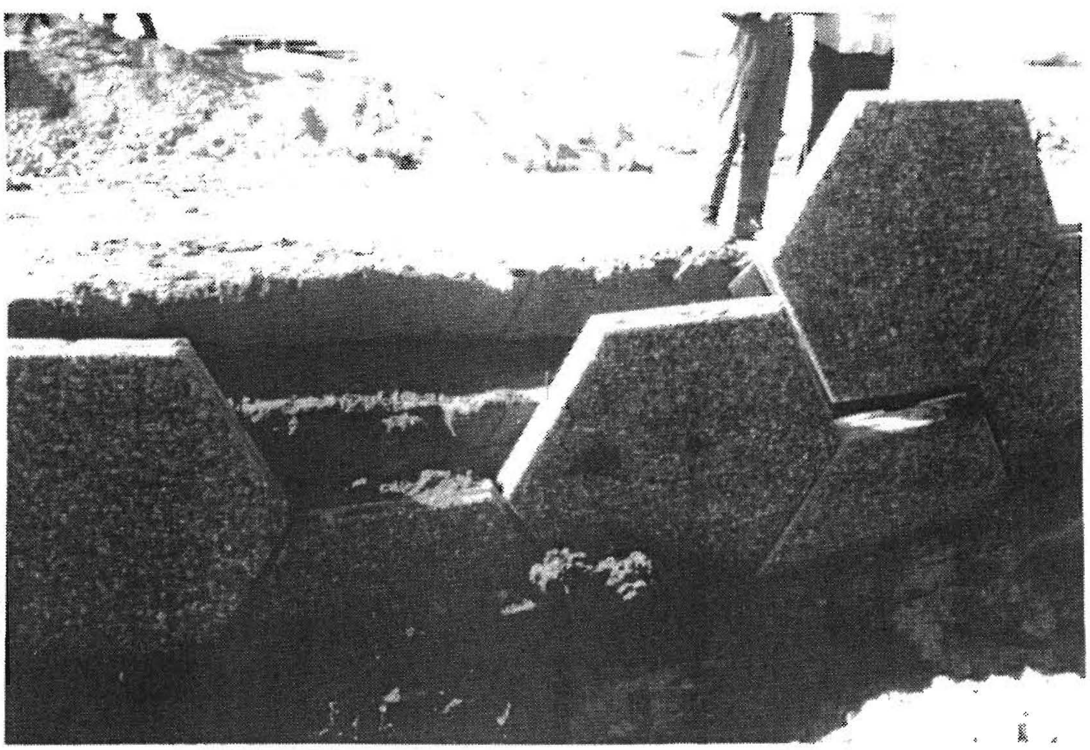


Figure 7.2 Weak fill at face of wall

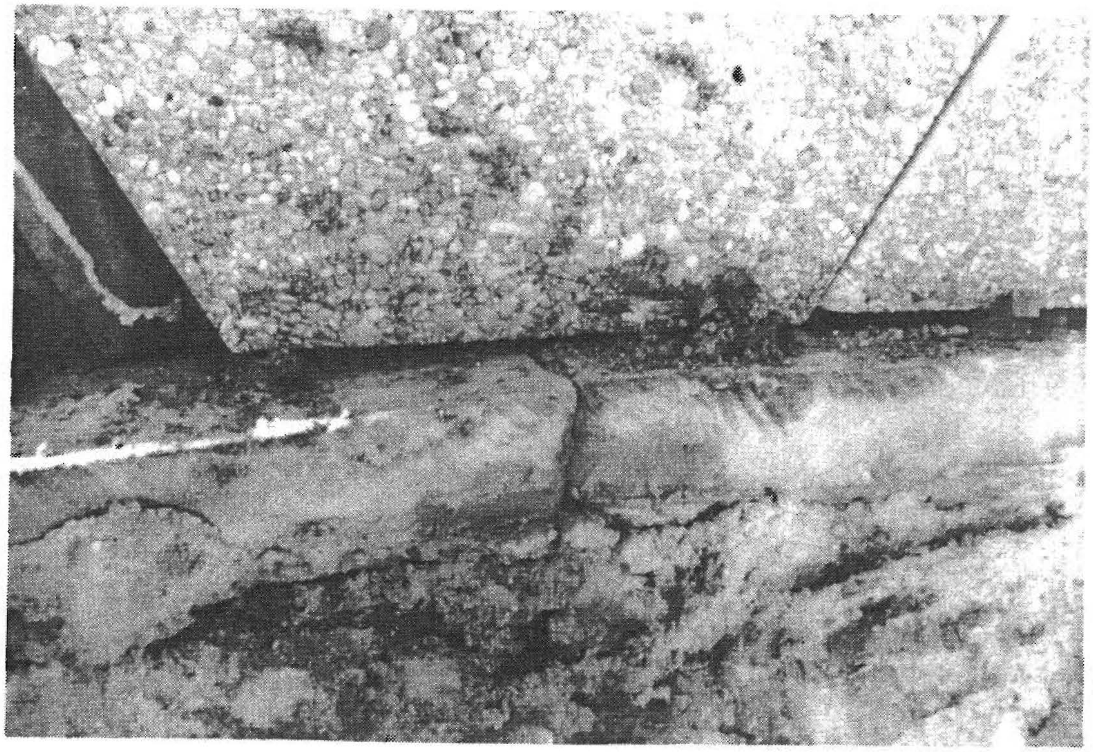


Figure 7.3 Plastic hinge in levelling pad

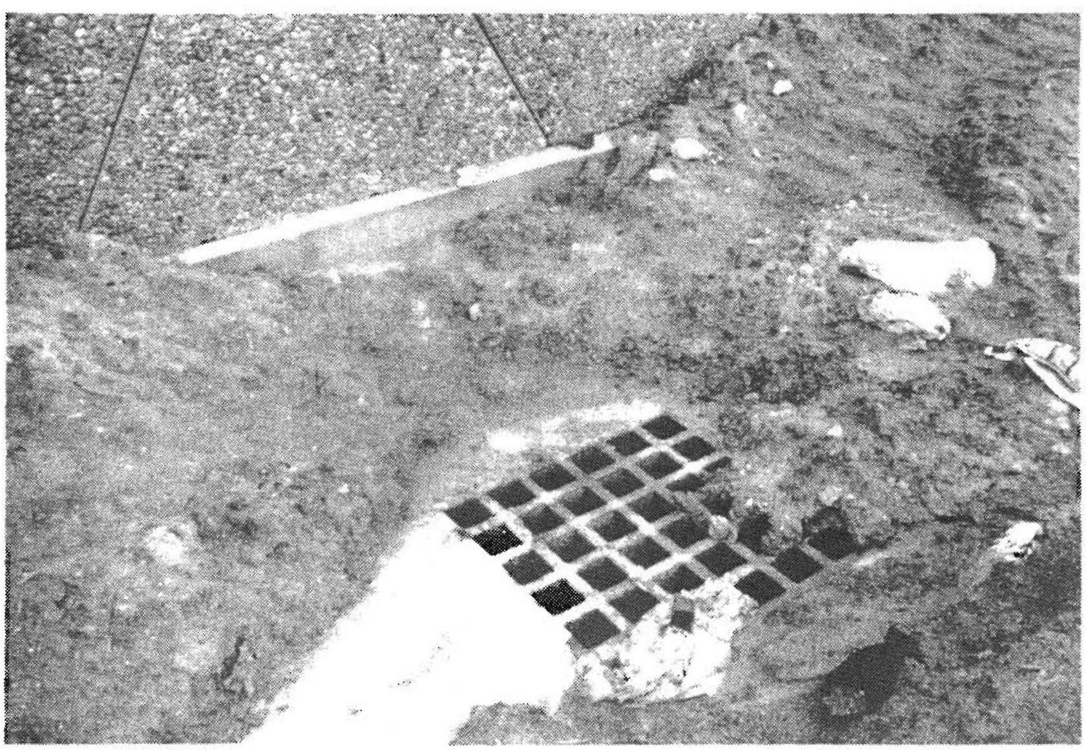


Figure 7.4 Underground culvert manhole close to wall

Some bulging was also noticed at the second section of the wall, towards the southeast. This did not seem to be associated with any obvious settlement of the foundation. This distress may well have occurred as a result of the props being taken off too soon, in conjunction with poor strength development (as well as the poor drainage referred to earlier). Not only was the strength required in the specifications not achieved (even after substantial time periods), but the contractor appeared to have been unaware of the need to keep the props in position for a reasonable curing time (which is of course not necessary for the VSL design). Some longitudinal cracking of the stabilized fill was evident, behind the front face, particularly at this location.

7.2 Cracks within Fill

The longitudinal cracking of the cement stabilized fill at wall 7 was addressed, as it was suggested that it might have been due to slip circle development within the wall itself, and that the stabilized fill strength may therefore have been insufficient to prevent slope failure of the fill.

This is not believed to be the case, certainly if the strength of the fill remotely approached the values required in the specifications (notably 200 psi). A slip circle analysis was performed for the completed wall 17 ft.high. The minimum factor of safety found by a scanning routine (using PC-SLOPE and a Bishop's slip circle analysis) was 7.7. Conservative strength values were used of 100 psi.unconfined compression strength (corresponding to $C = 7200$ lbs/ft^2) and zero friction ($\phi = 0$).

The analysis is shown in Figure 7.5, and was conducted with a grid of centers such that potential failure circles were forced to pass through the stabilized zone - thereby giving factors of safety for failure of the stabilized zone itself. It was possible to produce lower factors of safety for circles that pass underneath the stabilized zone, through unstabilized soil only, but this would correspond to failure of the foundation, and there was no evidence of this.

Stability charts for vertical slopes gave similar answers. The conclusion was therefore that slope failure of the retaining wall was not a credible explanation for the cracks, unless the stabilized strength fell to as low as 15 psi.or thereabouts. It is possible that this may have happened locally, judging from the extremely weak apparent nature of the stabilized fill, right at the front face after the original bulging failure. However the

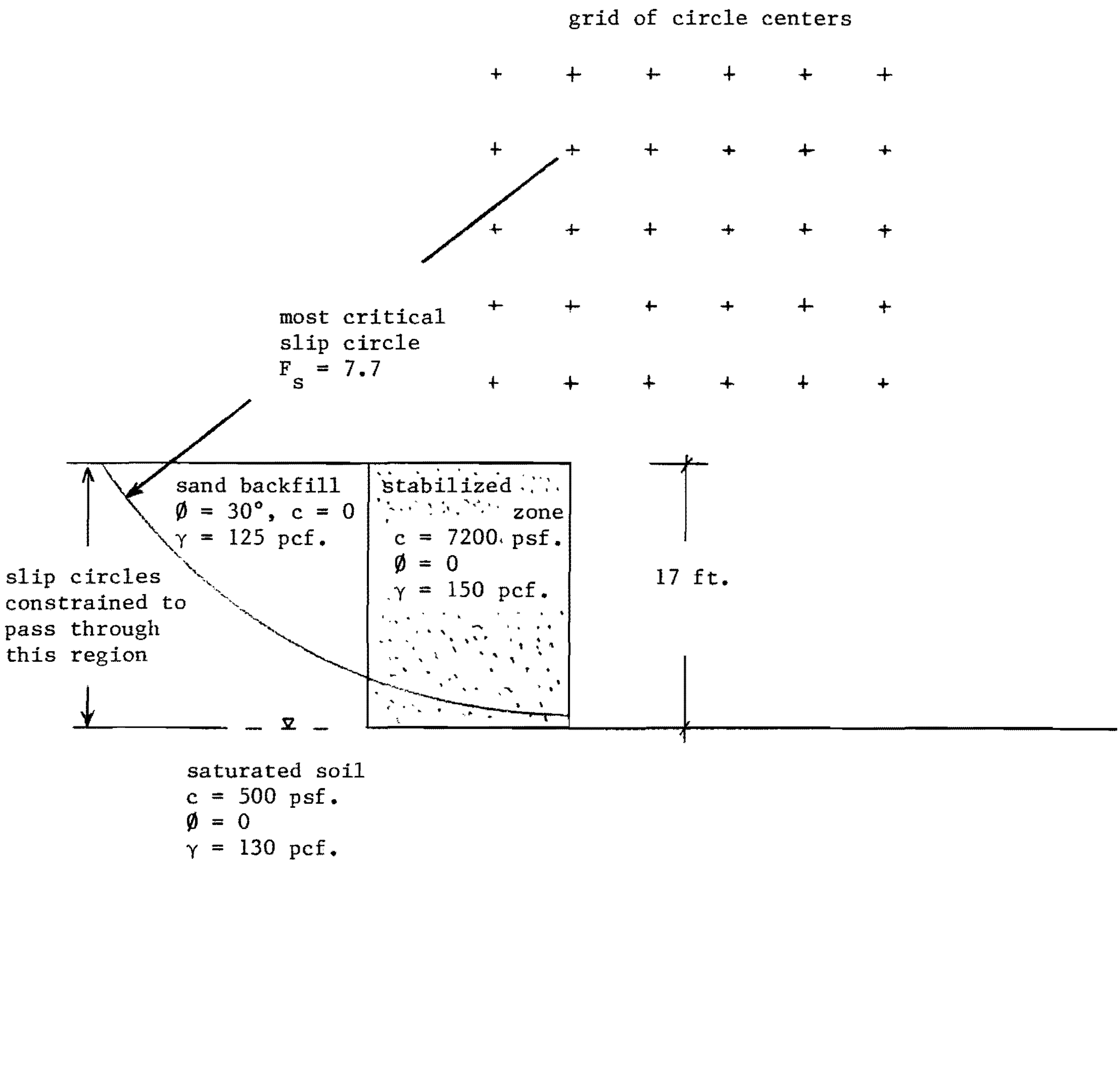


Figure 7.5 Slip circle analysis of wall 7

information from the tests indicate that the situation was not as bad at this, throughout the material as a whole, values ranging from 119 to 185 psi., as summarized in the next section.

The cause of the cracks would seen to be related primarily to the large deformations taking place at this site. This includes both the substantial differential settlement of the subsoil in certain locations, and also the lateral motion of the individual facing units that is liable to take place if the temporary props are not left in place long enough for proper curing and strength development to take place. Thermal cracking was very unlikely at the low cement contents used here. However the existence of weak layers between successive lifts (resulting from failure to clean off mud and traffic dirt between lifts, and also the absence of significant surface relief between lifts, as specified) did not contribute to overall wall integrity.

7.3 Material Testing

A summary of the unconfined compressive strengths (in accordance with ASTM D-1633) as obtained from trimmed bulk samples from the field, and tested approximately 5 weeks or more after placement, is as follows:

TABLE 7.1 - Summary of material test results

Station:	1963+00	1963+50	1963+70	1964+50	1965+00
Strength:	125 psi	185 psi	119 psi	178 psi	137 psi
Cement Content:	6.8%	5%	6%	6.9%	5.5%
Station:	1965+25	1965+35	1966+00	1966+50	1967+00
Strength:	142 psi	127 psi	187 psi	128 psi	142 psi
Cement Content:	5.9%	7.1%	5.3%	6.4%	5.2%

Since the specified strength requirement was 200 psi, the placed fill would clearly have been under strength at the time of the failure (1 to 2 weeks) and even more so at the time that the props were first removed. The specified cement content was 4.5% (or 1.3 sacks/yd3), so that insufficient cement content did not appear to be the problem. In hindsight the sand used was too fine - grained for an adequate mix design, as the contractor just used a local supplier without testing trial mixes, and apparently simply assumed that adding the specified minimum cement content would achieve the required strength. Full details are given in Appendix B.

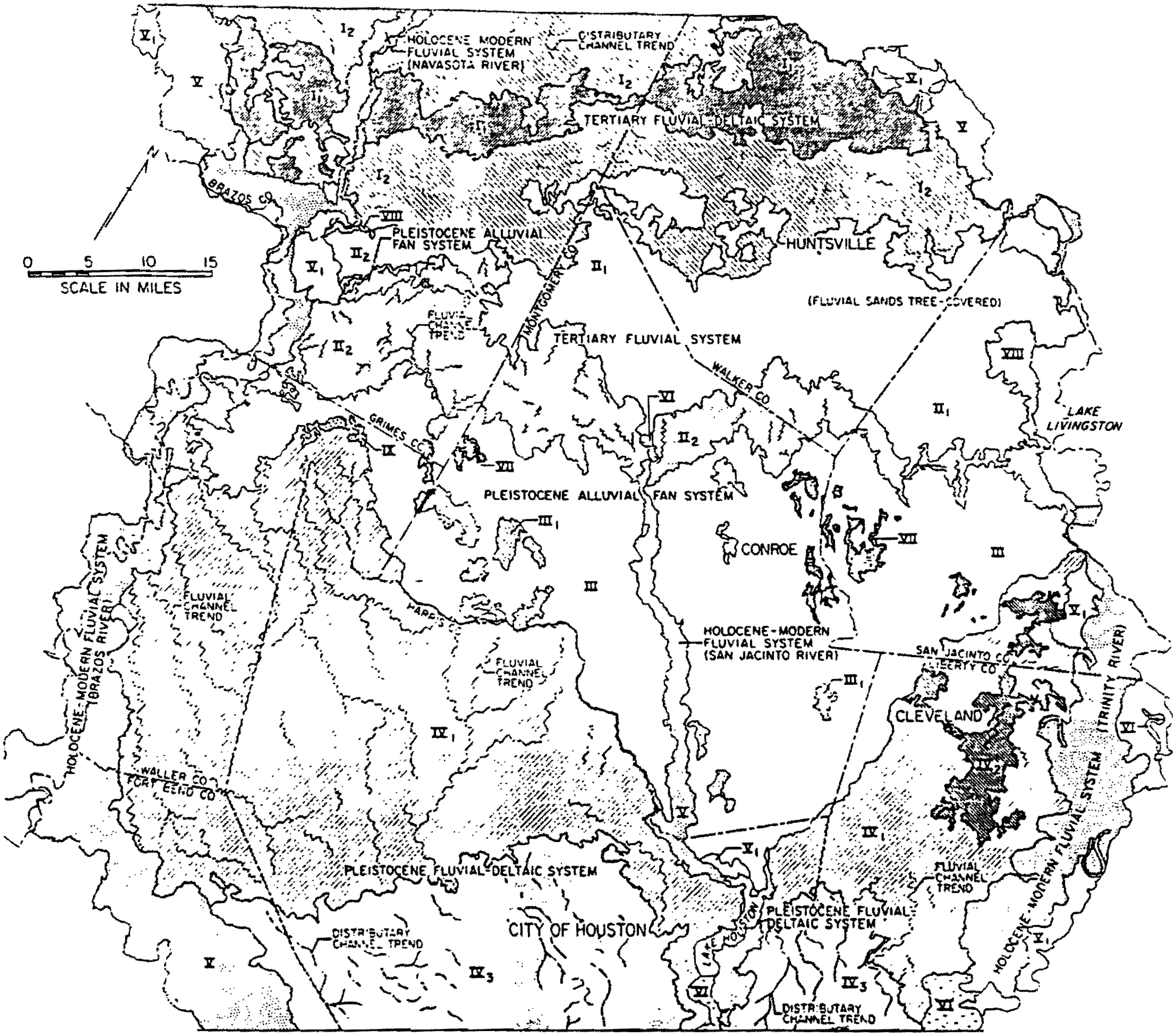
7.4 Foundation Investigation - Site Geology

A geologic review of the history of the site, indicates that the project is located upon the Pleistocene age, Montgomery Formation, illustrated in Figure 7.6. Previously this unit has been mapped and referred to as the Upper Lissie Formation. A general listing of the Montgomery Formation's characteristics is as follows:

> Clay, silt and very minor siliceous gravel of granule and small pebble size, gravel more abundant northwestward, locally calcareous, concretions of calcium carbonate, iron oxide, and iron manganese oxides common in zone of weathering; Fluviatile; Surface fairly flat and featureless except for numerous rounded shallow depressions and pimple mounds; +/- 100 ft.thick.

Previous boreholes had been drilled at the site, in April 1985 and May 1986 by the Texas Department of Transportation. The results of the boreholes were made available, and portions of the data were used to interpret the soil stratigraphy underlying the project site. Specifically seven borings spaced from station 1960+55' to 1972+76' were employed to generate a subsurface cross-section between walls 7, 8, 9, and 10 as shown in Figure 7.7.

As can be seen from the cross section, the geometry of the stratigraphy is typical of the Texas gulf coast in general and fluvial-deltaic complexes in particular. Abundant variation in sedimentary materials, lateral facies changes over relatively short distances and gradation trends are all indicative of the fluvial-deltaic nature of these deposits. The environment of deposition of the large sand bodies is one of relatively high energy. It is likely that these units were formed from ancient distributary channels during deltaic progradation. On the cross section are pulses or sequences of deposition indicated as "fining upward". These are another indicator of a fluvial environment of deposition for the soils at the site. The coarser sediments within these sequences are generally higher energy deposits. As the energy of the system decreases due either to channel abandonment or decreased flow following flood events, finer grained sediments are deposited.



TERTIARY

I FLUVIAL-DELTAIC SYSTEM
I$_1$ FLUVIAL-DISTRIBUTARY SAND, TREE-COVERED
I$_2$ DELTAPLAIN MUD
II FLUVIAL SYSTEM
II$_1$ FLUVIAL SAND, TREE-COVERED
II$_2$ FLOODPLAIN, OVERBANK MUD

HOLOCENE-MODERN

V FLUVIAL SYSTEM
V$_1$ TERRACE SANDS
VI MARSH-SWAMP SYSTEM

PLEISTOCENE

III ALLUVIAL FAN SYSTEM
III$_1$ OVERBANK MUD PRAIRIE
IV FLUVIAL-DELTAIC SYSTEM
IV$_1$ FLUVIAL SAND
IV$_2$ OVERBANK MUD PRAIRIE
IV$_3$ DELTAPLAIN MUD

OTHER UNITS

VII IRONSTONE SOILS
VIII SLOPE UNITS
IX FAN UNITS

Figure 7.6 Geologic map of area

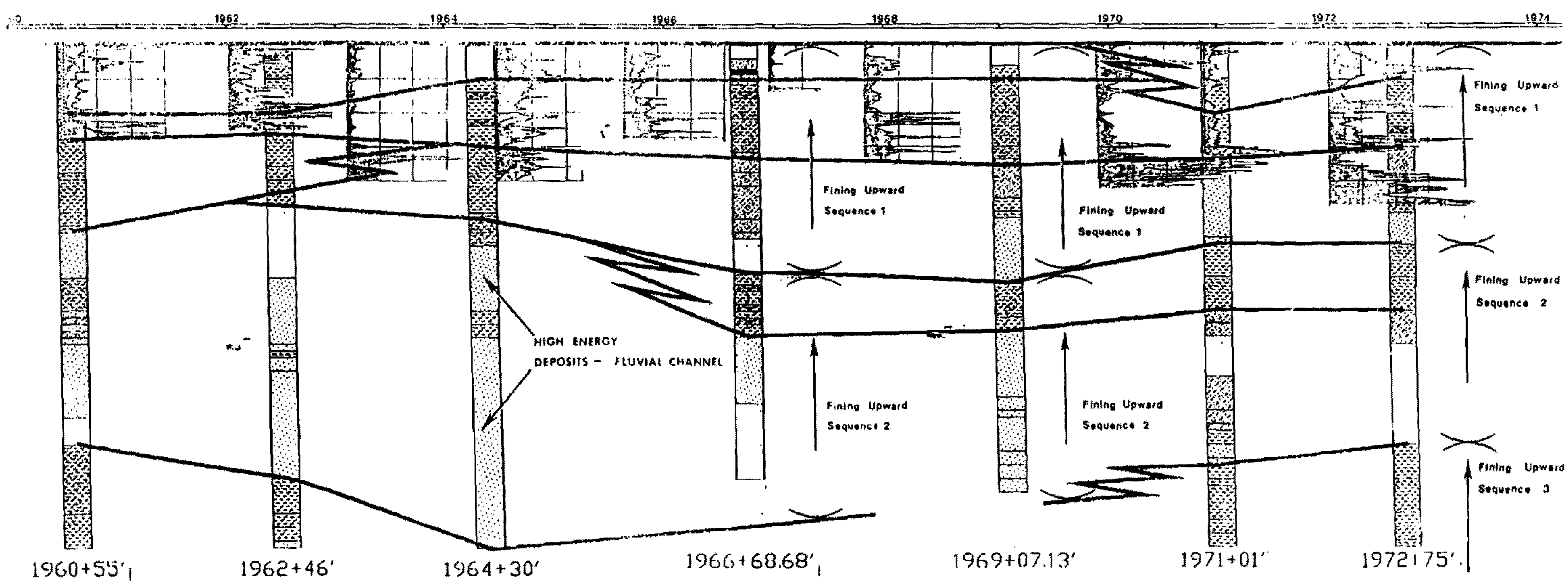


Figure 7.7 Geologic cross-section through site

General trends across the site are:

1. Except for the N.W. end of walls 7 and 8, initial sands are found at a depth of approx.15 to 20 ft.

2. The units tend to become coarser overall with depth up to the data limit of 75 ft.

3. The shear strength of the soil although quite variable tends to increase with depth.

4. Maximum values of shear strength occur at a depth of approximately 50 ft.

5. The soils at a depth of 5 to 15 ft. tend to be stiff.

6. Toward the S.W. (walls 9 and 10) units tend to become more complex leading to greater variations in physical properties.

This confirms the variable and difficult nature of the foundation soils at the site. A more detailed discussion of the site geology is given in Appendix C, together with the original borehole logs at the site from 1985 and 1986.

7.5 <u>Cone Penetration Testing of Foundation</u>

In response to performance problems experienced by the walls during construction, a more detailed analysis of the subsurface characteristics directly adjacent to the walls was deemed necessary.

A major effort was made in examining the true sub-surface conditions as accurately as possible. A program of cone penetration testing was implemented in the areas of wall 7, 8 and 9 to provide a continuous record of soil strength with depth.

In order to allow the soil to be probed as close to the retaining walls as possible, a miniature piezo-cone was specified, which allowed probing within a foot or less of the wall leveling pad. (A standard cone penetration truck would not have been able to probe closer than about 5 ft. to the wall, which would probably not make it between the wall and the suspected buried culvert).

A total of thirteen electric cone penetrometer tests (CPTs) were performed ranging from 6.5 to 23.5 ft. penetration. The CPTs were located in the field by Mike Garrison of the Texas Department of Transportation. The locations of these soundings relatively to the earlier soil borings and the retaining walls are shown on Figure 7.8. It should be noted that the initial depths of the cone penetrations passed through reworked material excavated and recompacted in conjunction with construction of the wall. Therefore analysis of the point resistance could give a good indication of the degree of compaction achieved along the base of the walls.

The miniature electric cone penetrometer unit consisted of an instrumented tip and sleeve. A push rod and a hydraulic ram advanced the tip and sleeve into the ground. The system was mounted on a light-duty truck. The dead weight of the vehicle served as a reaction to the force required to push the penetrometer into the ground. A photograph of this system is shown in Figure 7.9.

The tip resistance against penetration into the soil is the cone resistance (q). A conical tip with an apex angle of 60 degrees and a base of 1.27 cm^2 measures this resistance. The operator pushed the tip into the ground at a constant penetration rate of 2 cm per second. A 25.4 cm^2 cylindrical sleeve located immediately above the tip measured the local side friction resistance (f). Strain gauge load cells built into the tip and sleeve continously measured the penetration resistance components, g and f. Electrical cables, located in the hollow push rods, carried signals from these cells to the on-board computer which digitized the data and recorded it on floppy disks.

The CPT traces produce plot of point resistance, q sleeve friction, f, and friction ratio, f/q. Obtaining geotechnical engineering parameters requires interpretation of the cone data. The soil stratigraphy was identified using Campanella and Robertson's Simplified Soil Behavior Chart reproduced in Figure 7.10. Soil strength parameters were estimated for 0.2 ft. depth intervals using the following relationship:

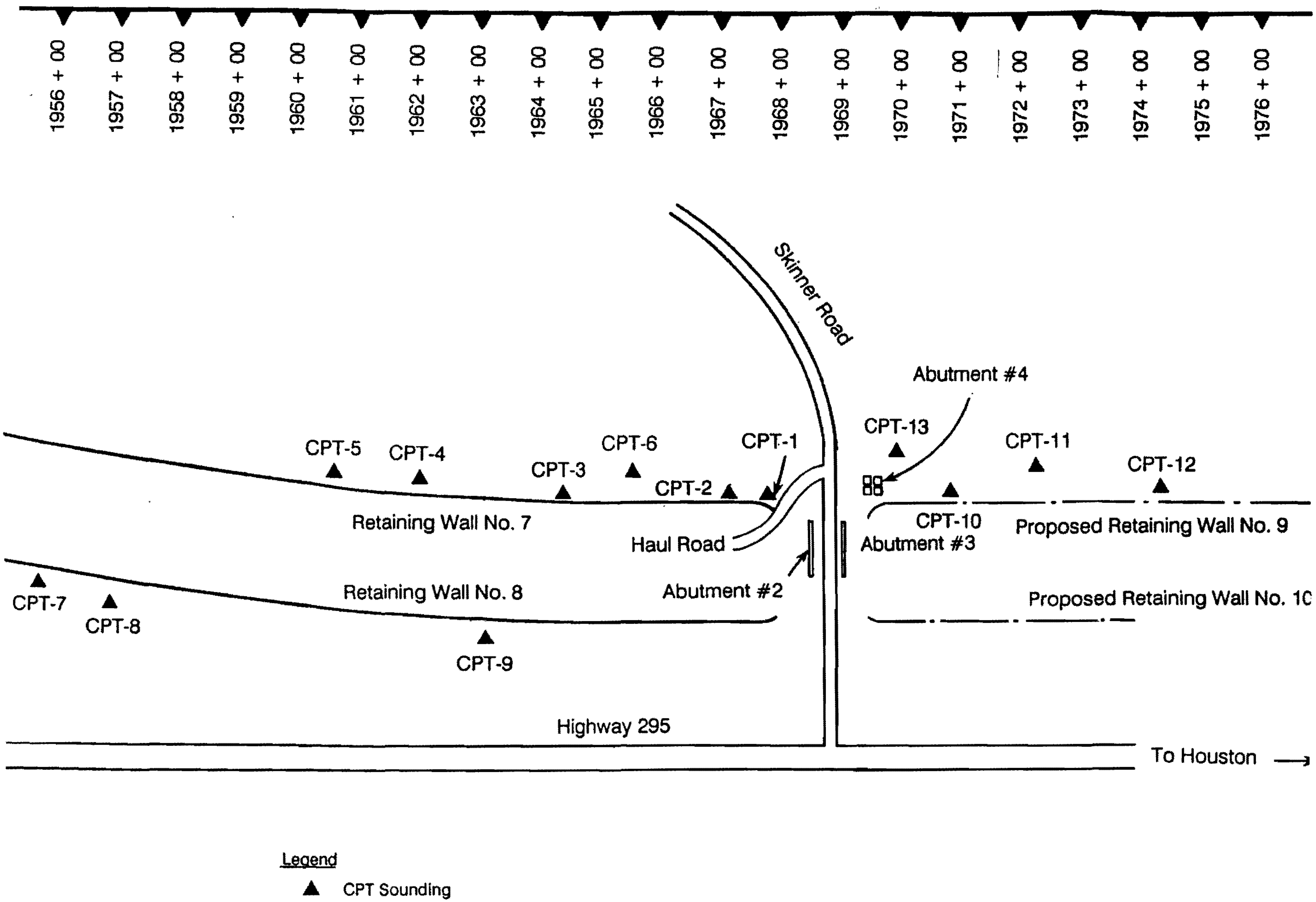


Figure 7.8 Location of cone penetrometer soundings

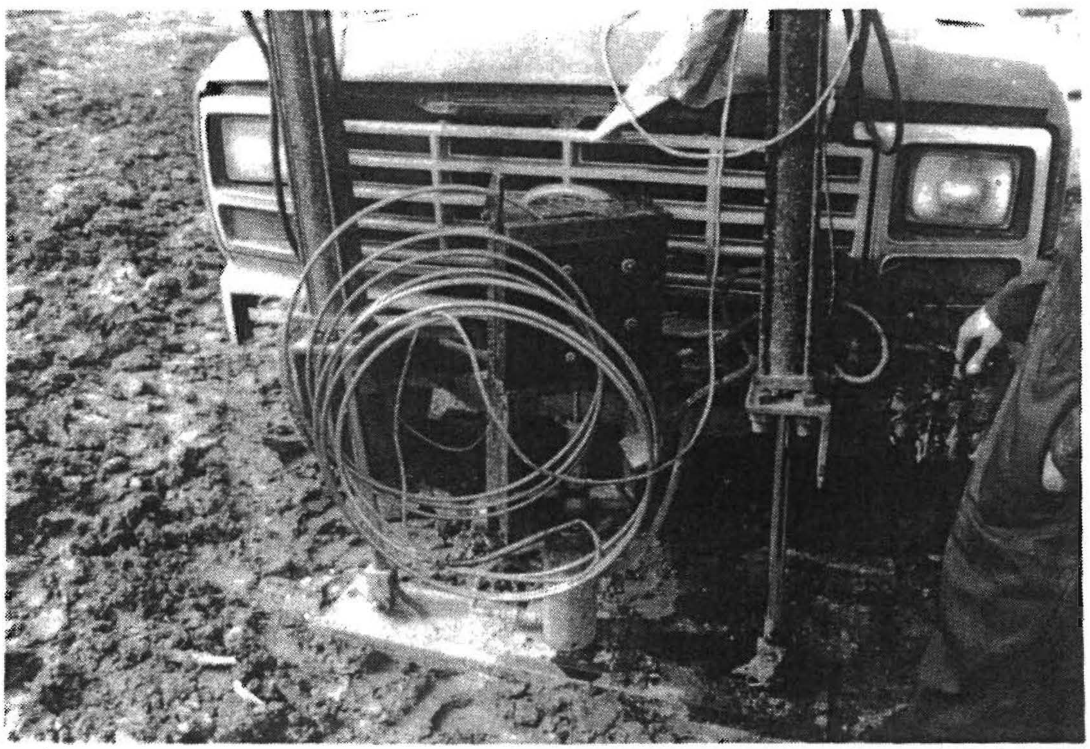


Figure 7.9 Photograph of cone penetrometer sounding

INTERPRETATION OF CONE PENETROMETER DATA

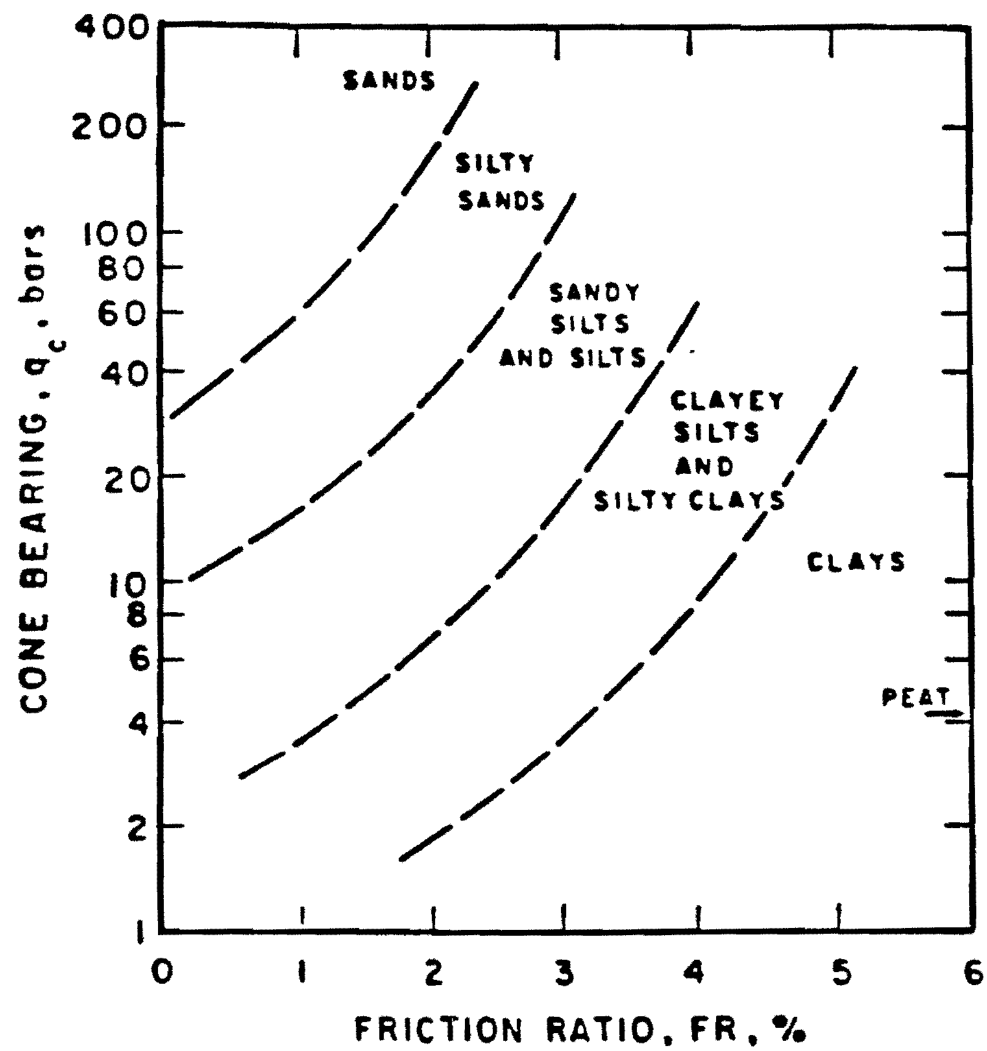


Figure 7.10 Classification chart for cone testing

$$C_u = \frac{q_c}{N_k}$$

where:

	q_c:	Tip resistance
	C_u:	Undrained shear strength
	N_k:	Cone factor, 19.5 for this mini-cone

This showed the following typical values of undrained shear strength (in k.s.f.) in the upper soil:

TABLE 7.2 - Summary of cone penetrometer strengths

Hole:	1	2	3	4	5	6	7
C_u:	1 to 2	0.6	2 to 4	1 to 3	1 to 2	1 to 2	0.5 to 1.5
Hole:	8	9	10	11	12	13	
C_u:	1 to 3	0.5 to 2.0	0.3 to 2	1 to 4	0.6 to 1	0.5 to 3	

Values consistently less than 0.5 k.s.f. (1000 p.s.f. or 7 psi or 50 kPa.) represent a factor of safety of less than 2 against undrained bearing capacity failure of a wall 20 ft. high, and are clearly undesirable. Quite a significant proportion of the subsoil unfortunately appeared to fall into this category.

More complete details of the cone penetration testing performed, are given in Appendix D. Laboratory testing to confirm these findings, was also performed on tube samples by the District 12 soils laboratory, and these are summarized in Appendix E.

7.6 Site Reworking

Initial reworking of the problem areas was carried out by the contractor, and regrading of the ground surface was carried out to improve site drainage. The affected areas included both the experimental and the conventional Retained Earth retaining walls, both of which exhibited significant outward tilting (especially wall 8, which was a conventional Retained Earth wall, shown in Figure 7.11).

Rehabilitating the foundation for the existing experimental wall (wall 7) as well as some of the conventional reinforced earth walls, was suggested by the use of remedial piling through the compressible backfill to form a relatively rigid support for subsequent construction. After consulting Mr. Michael Ho of District 12, the decision was made to incorporate piled foundations underneath walls 2, 4, 5, 7 & 8, based on the combined results of the site investigations carried out by D-12 and T.T.I.. A plan of the design adopted is shown in Figure 7.12, involving 90 ton precast concrete piles installed 40 feet below existing grade level in two offset rows. The innermost row was installed at a 5 vertical to 1 horizontal batter, as can be seen also in the cross-sectional view in Figure 9.4. A photograph of the finished piled foundation is shown in Figure 7.13.

While this was a controversial (although undoubtedly conservative) decision, from the point of view of testing the new design it was obviously preferable to be able to build an experimental wall on original earth. This would clearly be most representative of normal conditions, as conditions are seldom so bad as to require compaction piles under retaining walls.

Fortunately, the consensus of opinion was that foundation piling would probably not be required at the location of wall 9, which meant that wall 9 (the higher of the two cement stabilized sections) would still be available for monitoring and testing on an original soil foundation. While this wall section here was somewhat higher than the cement stabilized section first constructed at wall 7, the results of the soils lab testing indicated that the subsoil properties at wall 9 were not as bad as at wall 7 and did not require expensive foundation remediation. Furthermore there was evidence that the contractors had been somewhat more careful about their soil backfilling at this location, as this was one of the last locations. The site investigation information here is also available directly underneath the position of the proposed wall, whereas at the previous

Figure 7.11 Outward tilting of conventional wall 8

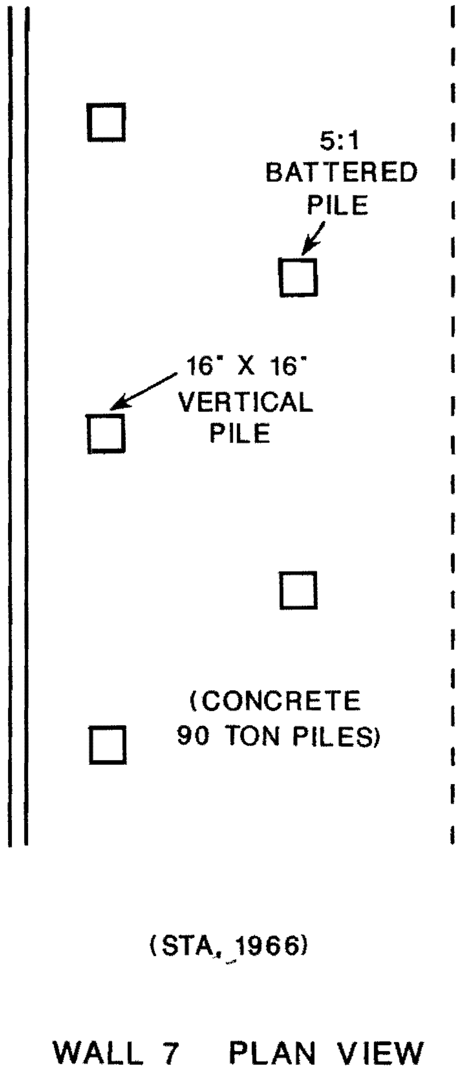


Figure 7.12 Plan of piled foundation for wall 7

Figure 7.13 Photograph of piled foundation for wall 7

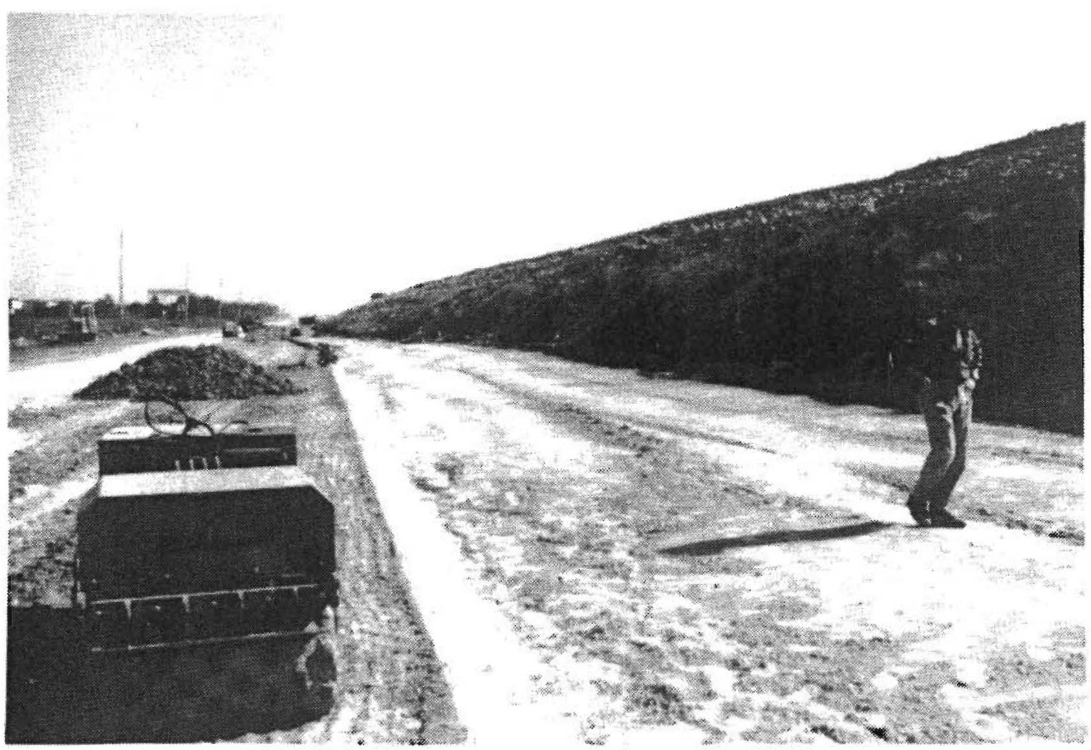
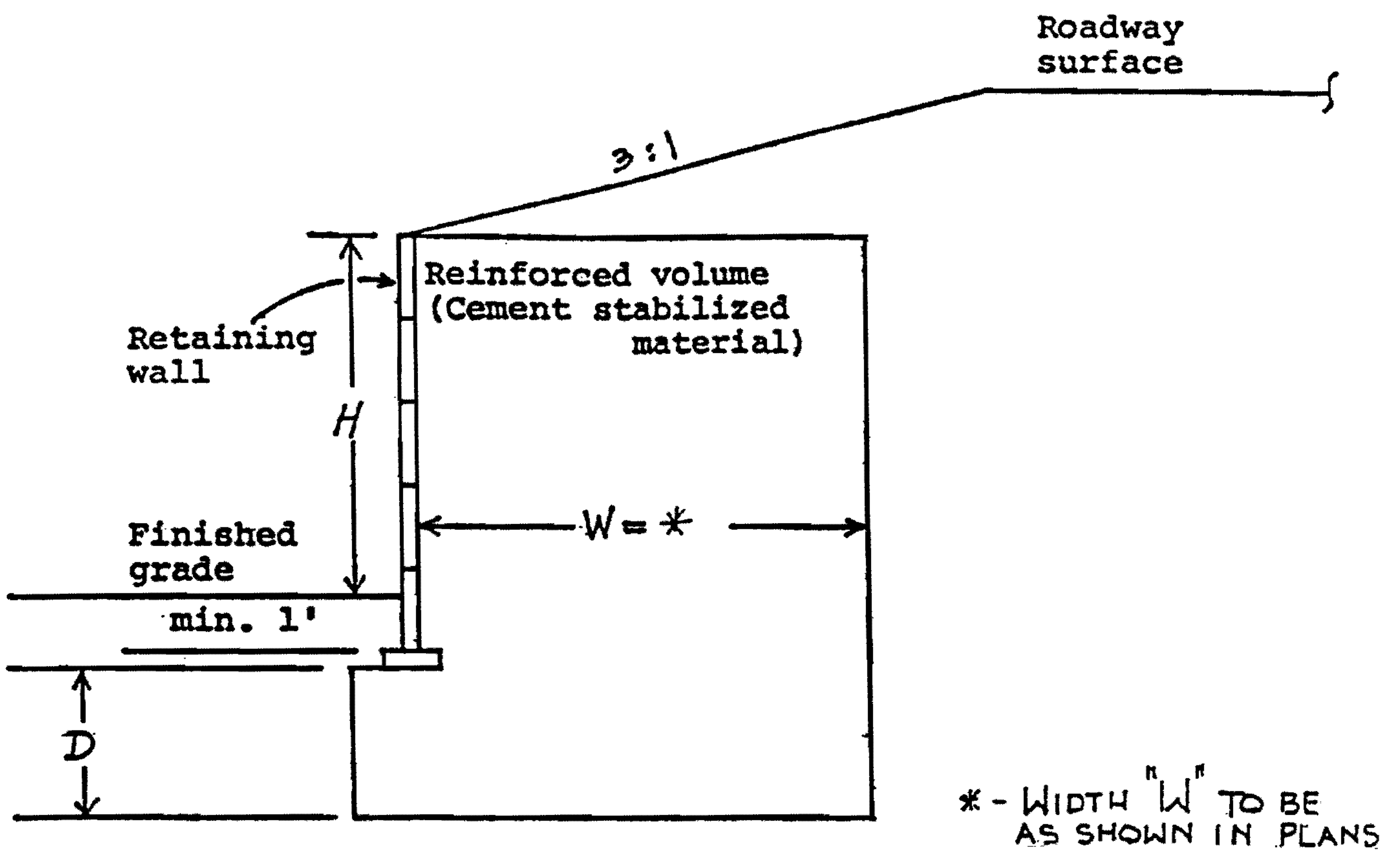


Figure 7.14 Foundation preparation for wall 9

locations, information is only available to one side of the existing wall locations. Subsequent observation and behavior of field performance at this site, has confirmed this assessment of the situation.

As an additional precaution, an additional 6 feet of stabilized fill was also specified underneath the proposed construction at wall 9, to provide a substantial reinforcement and improvement of the foundation to a reasonable depth. Figure 7.15 shows the modified design detail for wall 9, and this is believed to be significantly more tolerant of subsurface settlement, although it is also somewhat more expensive. (In reality, the passage of time will also tend to help things, as any improperly compacted soil settles and consolidates).

Figure 7.14 shows a photograph of the foundation of wall 9 after preparation, with the stabilized fill extending well under the levelling pad; and this installation has performed satisfactorily.



Not to scale

Section view

Sta.1969+50 to Sta.1971+80 : D = 6 feet
Sta.1971+80 to Sta.1972+70 : D = 5 feet
Sta.1972+70 to Sta.1975+80 : D = 4 feet
Sta.1975+80 to Sta.1976+20 : D = 3 feet
Sta.1976+20 to Sta.1979+95 : D = 1 feet

Figure 7.15 Modified design detail for wall 9

8. CONCLUSIONS FROM FIELD PROBLEMS

8.1 General

As a result of considerable investigations of the initial retaining walls constructed at the Cypress-Fairbanks bypass location, a number of conclusions can be made. These resulted from observations not only at the experimental wall (which at that time was only wall 7), but also of the conventional VSL Retained Earth walls, which also had significant problems (as shown in FIgure 7.11), and in most cases had to be rebuilt. The conclusions can be divided into the following general categories, as follows.

8.2 Strength Requirements

As outlined in section 7.3 "Material Testing", it is clear in hindsight that the requisite strength requirements of the initial specifications, were not met. Average compressive strength values of bulk samples recovered as long as several weeks after field construction, were significantly less that the required value of 200 psi at 7 days, even though they had had the advantage of a much longer curing time.

Ironically, the cement content of the stabilized fill did satisfy the specifications, averaging 6% when the minimum value specified was only 4.5%. In conversation with site personnel, part of the problem seems to have been that the original sand used in the mix, was just locally available sand, with a very fine gradation (almost 15% passing the number 200 sieve), barely satisfying the gradation specification, and with rounded to sub-rounded grains. No separate strength testing was done on the material at the time of placement, apparently because it was assumed that if the gradation specifications were satisfied, and also the cement content specification was satisfied, then the strength specification would automatically be satisfied.

This of course turned out not to be the case, and was particularily true in areas where adequate compaction had been neglected - notably close to the facing units (see next section). This region was especially critical for this design, and when coupled with the use of sub-strength material, gave rise to particular problems there.

In the final construction and design, as mentioned in section 9, a much more carefully controlled sand was trucked in, for use in the mix. This proved to be much more suitable (primarily, it transpired, because of much greater grain angularity) and did not have any difficulty in achieving the design stabilized strength of 200 psi at 7 days. Indeed this strength was generally comfortably surpassed, test specimens achieving 400 to 500 psi. However, it did highlight the importance of paying attention specifically to strength requirements, rather than simply assuming that gradation and cement content requirements would automatically be satisfactory.

8.3 Propping Requirements

It was noted also that there was a tendency by the contractor to remove facing unit props, relatively soon after the anchors were embedded in the backfill. This was a practice that would be possible with a conventional reinforced earth retaining wall, as the frictional force between the retaining straps and the granular backfill is essentially developed immediately. However, for this design, where a significant time period is required to allow the cement component to harden and develop strength, removal of props must be done much more carefully.

The original specifications required that the external bracing be left in place for a minimum of 2 days after placement of the backfill. It seems likely that even this relatively easy restriction was circumvented initially, while the first experimental wall section was being constructed largely as a conventional reinforced earth wall. When taken in conjunction with the very low strengths initially achieved close to the facing units, it is perhaps not surprising that some of them partially separated from the backfill under water pressures.

The second experimental wall section has performed adequately, without any change to the propping specifications (except that the contractor was more aware of its importance on this occasion). Consequently no further change has been suggested at present. However, it is possible that the bracing should be left in place for at least 2 days, or "as long as is required to develop the design strength", whichever is greater. If the stabilized soil mix is well designed (as was the case for the second experimental wall), then the design strength will probably be developed in 2 days, and this would not cause any inconvenience. However, in marginal situations, the developed strength requirement might control the bracing specifications, to a much greater extent than for conventional reinforced earth walls.

8.4 Compaction Requirements

The actual compaction densities achieved on this site, were mostly adequate, and did not cause problems globally. However, there were problems locally, at the facing units and at the anchors, where the importance of maintaining good compaction control appears not to have been always appreciated.

As mentioned in section 8.2, fill strengths were particularily low right behind the facing units, where compaction was poor. In some cases, actual voids of uncompacted fill appear to have been left behind the facing units (see Figure 8.1).

Again, this appears to be primarily a legacy of accepted practice with conventional reinforced earth walls, for which good compaction is not necessary right behind facing units, as they are retained by frictional straps that run for long distances into the frictional fill. Indeed, for conventional walls, good compaction right behind the facing units can actually be bad, as this can induce larger horizontal stresses than would otherwise be the case, and can cause localized "bulging" and movement of sections of the wall.

However, with the stabilized fill design, good compaction is required right up to the facing units, in order to develop acceptable strengths and bond for the anchors. If necessary, hand compaction in this region could be specified, although an alternative (if good compaction here proved to be difficult and expensive) might be to lengthen the anchors so that adequate anchorage could be relied on within the bulk of the fill. This would however start to counteract the advantages of the experimental design.

An additional point to be considered, is that compaction lifts should preferably not stop at the anchor elevations. Figure 8.1 again illustrates this point. If construction lifts are broken at these levels, then a plane of weakness automatically exists here, and it becomes somewhat easier for an anchor to be pulled out under excessive loads. Again, this a concern that does not exist for conventional reinforced earth walls, as only frictional

Figure 8.1 Compaction voids close to wall face

strengths need be developed between adjacent lifts, and no cementitious bond is required. If construction lifts were always continued to a level between anchors, then the presence of any joints between lifts would not have an effect on anchor capacity.

This is an example of a construction detail that differs significantly from the common practice in the standard reinforced earth design, even though the two appear superficially similar. Inexperienced contractors are of course liable to approach construction planning of the two designs in much the same way, even though there are clearly significant differences.

The bulging at the construction face would probably have been somewhat less likely if full-height one piece panels had been used, which would have been more successful in bridging over successive construction lifts. However, this would probably have required a separate contract for this project. In the event, adapting the VSL panels enabled the bid process for the project to be greatly streamlined, in view of other contractual project delays. In hindsight, extra care (particularly in the temporary supports) is probably required for the cement-stabilized design, if segmental panels are used.

8.5 Placement Conditions

Other factors that were important for the experimental design, and were not really appreciated by the contractor in initial construction, were the importance of maintaining surface relief between lifts, good site drainage, and the importance of removing loose soil between lifts. If these factors were neglected (as in some instances they were) then good bond between adjacent layers of placed fill could no longer be relied upon.

Good surface relief had been mentioned in the specifications, but was not consistently maintained initially. In some instances, rubber-tired rollers were used to compact fill, which resulted in an unnecessarily smooth surface finish. The original specifications were undoubtedly adequate in this respect, and when this was pointed out, sheepsfoot rollers were brought in for compaction. These were much better in producing a surface with significant relief, and no doubt contributed to the greatly improved wall integrity after the initial problems.

Site drainage at wall 7 had also been inadvertently arranged so that surface runoff from the soil embankment behind the wall, drained onto the top of the working surface of the cement stabilized fill. This exacerbated problems with surface bond, as any heavy rainfall would tend to wash loose soil onto the next construction joint, producing a plane of weakness and reducing bond there. The problem was much less marked subsequently, when drainage was arranged to carry surface water away from the face of the wall, and away from the working surface of the cement stabilized fill.

In the same vein, a lot of loose soil appeared to be present initially, on the top of placement lifts, if they had been allowed to stand for a while. The importance of fully cleaning these off, had not necessarily been fully appreciated initially, as it would not normally be a major concern with a reinforced earth wall, even though the specifications mentioned specific precautions in this regard. Again, this potentially resulted in a plane of weakness in the resulting cross-section. If this occurred at the elevation of a line of anchors (as was often the case), then this would serve to reduce even further the anchor bond strength, that might already have been reduced by insufficient compaction close to the wall.

8.6 Foundation Conditions

Another design condition that was not entirely appreciated initially, was the increased sensitivity of a more brittle design like this, to the deformation of the foundations. Although a cement-stabilized wall like this, will always be more brittle than a purely mechanically reinforced wall, the sensitivity of the cross-section to subsurface settlements can be substantially reduced, if accommodated in the initial design.

The original design cross-sections (shown in Figure 6.3) did include a layer of sand underneath the cement stabilized fill, but this was included primarily as a drainage layer, and was stopped short at the levelling pad used to support the first row of facing panels. In reality, a layer of sand here, serves to act as a "bedding layer" to accommodate a certain degree of differential subsidence, as well as providing drainage. Wall 7 was built according to this cross-section. Although the main cause of distress was almost certainly subsurface lack of compaction, as discussed in the next section, it was then realized that this zone could in fact be used creatively, and make life easier for the overall structure.

Consequently, in the revised drawings issued for the construction of wall 9, the thickness of the stabilized sand layer was increased to 6 feet (2 m.). The internal drainage pipe was in fact deleted (as underdrainage of the wall cross-section appeared to be a secondary design consideration, in any case). Most importantly, the sand layer was extended well past the toe of the wall to provide a significant layer of support for the wall above, especially under the leveling pad that is initially required to support at least part of the weight of the facing panel units. In this way, (especially if the sand was well compacted) this zone would help to accommodate and relieve the large stresses and stress concentrations that otherwise tend to build up at the toe of a gravity wall like this. Figure 7.15 shows the revised cross-section, and this is believed to be a significant improvement in this respect.

Such a foundation detail would probably allow a cement-stabilized design to be built on even relatively weak soils. In extreme cases, a piled foundation could be used, as was done for wall 7, although this is probably normally not necessary. Wall 9 was constructed to a height of 28 ft. (8.5 m.) on relatively soft soil, using the details shown in Figure 7.15 (without piling), and has so far performed well.

The absence of a specific drainage pipe does not seem to have been deleterious so far, as the sand is probably sufficiently permeable in respect to the other compacted fill and original earth, that the layer will act as a drain anyway. When the roadway and surface drainage details are completed, the amount of rainwater infiltrating into the backfill from the road surface will be greatly reduced from its present value. However even before completion of the roadway, drainage does not appear to have been a problem at this site, and certainly is secondary to other factors.

As was mentioned in section 6.2, the original standard detail for conventional Mechanically Stabilized Earth retaining walls has in the meanwhile been modified by the Texas DOT. The experience from this project indicates that structural support is a more important function of such lower layers, than underdrainage - at least for the relatively soft and clayey soil geology prevalent in the Houston area.

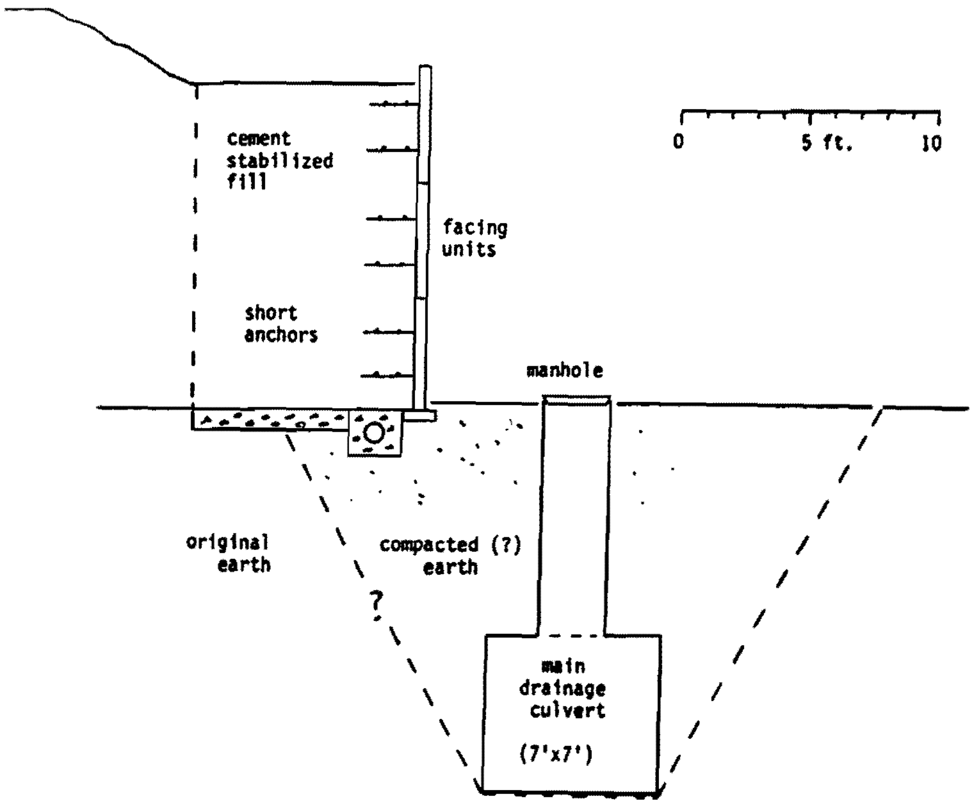


Figure 8.2 Cross-section of buried culvert

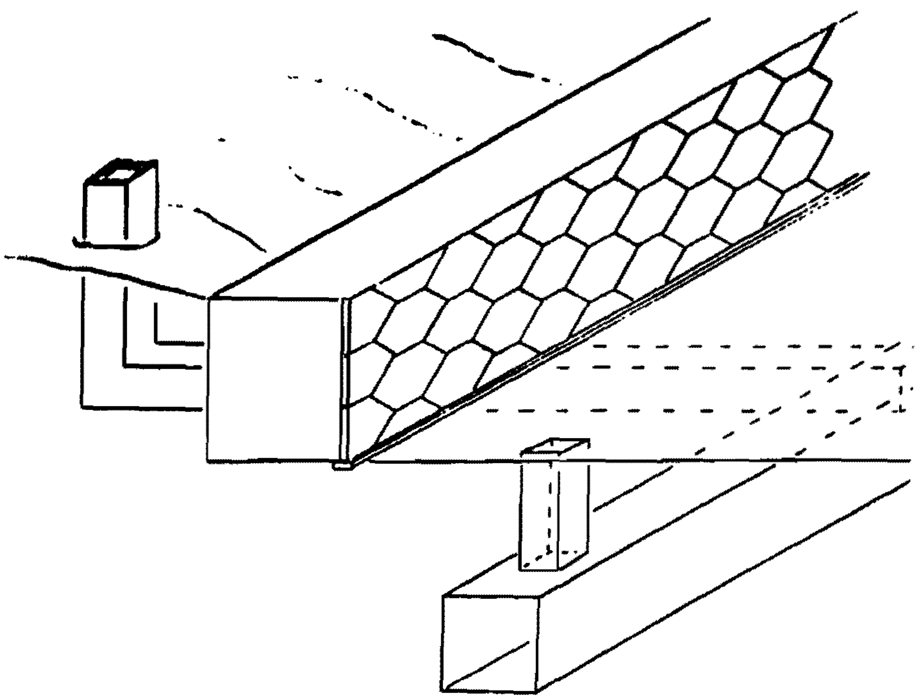


Figure 8.3 Diagram of buried culvert and side drain

8.7 Underground Structures

Almost certainly the biggest single contributor to the distress originally experienced at wall 7, was that an underground drainage culvert appeared to be founded too close to the toe of the wall. A close-up of manhole access is shown in Figure 7.4. This underground drainage structure had been constructed a year or two beforehand, and control of the compacted backfill appeared to have been slim to non-existent at the time. On the assumption that the sides of the trench had been sloped back to at least 1H. to IV. (to satisfy OSHA requirements), this meant that portions of the front toe of the retaing wall, were resting on compacted fill of uncertainly quality as is illustrated in FIgure 8.2. Local soil is very clayey, and the same material was almost certainly used for backfill.

Collection lines to the drop inlet intakes from the eventual road surface also ran underneath the wall at intervals, to deliver surface runoff to the storm drains. This is shown diagramatically in Figure 8.3. Compaction of the backfill over these buried conduits was also uncertain, and it was at such location that the major distress occurred at wall 7.

The relatively obvious conclusion is that care must be taken in constructing major earthfill structures over buried subterranean structures, especially if backfill placement is not controlled. It is certainly possible to do so safely, as evidenced again by wall 9, which was constructed satisfactorily over similar underground drainage installations. However, as discussed in the previous section, considerable attention was paid to the foundation details under these circumstances.

In general, manmade underground structures are best kept out of the zone of influence of such retaining walls.

8.8 <u>Stabilized and Non-Stabilized Cross-Sections</u>

The question has been raised as to whether or not cement stabilization of the cross-section is in general beneficial. It has been the practice in the Houston area for contractors to add cement to backfill material, whether required or not, even for conventional mechanically stabilized walls. The normal assumption is that the additional cement stabilization can only be a good thing, as it will increase the overall strength of the cross-section. However, one circumstance where cement stabilization may make things worse, is in respect to peak pressures at the toe of a retaining wall. Since a stabilized wall will initially act as a rigid body, the overturning moment from active pressures will cause a maximum pressure at the front toe.

Because a non-stabilized reinforced earth retaining wall is less rigid, and will "give" internally as soon as there are significant non-uniform stresses on the foundation, this effect will be less pronounced. Some non-uniformity of foundation stresses will still be present, as this is necessary to satisfy moment equilibrium, but internal wall deformations will allow the peak toe pressure to be partially redistributed over the rest of the cross-section. This would be an advantage if the wall were founded on particularly soft soil.

Redistribution of foundation stresses will also occur eventually with a monolithic cement stabilized cross-section founded on soft soil, but only after subsurface yielding and slight angular rotation of the cross-section. Because unstabilized cross-sections can deform internally at much lower stresses, the redistribution of stress would take place much sooner, although at the expense of some internal strains <u>within the wall</u>, rather than <u>within the foundation</u>, as is the case with a more rigid cross-section. The use of deformable compacted sand bedding layers underneath the retaining wall (ideally extending out in front of the toe) is suggested as being of potential benefit under these circumstances.

Interestingly enough, District 12 experience appears to be that settlement problems have increased on projects where cement stabilization has been omitted. This has been true in a number of locations, and implies that overall settlement is still the controlling factor in most situations.

9. FINAL FULL SCALE CONSTRUCTION

9.1 General

Due to the distress that occured in wall 7 during November 1990 the design procedures and specifications for the construction of the retaining walls at the Cypress-Fairbanks site needed to be revised. The revisions were to address the large settlement problem, the weakness of the stabilized fill, and methods for countering them. After considerable discussion, the revised design procedures and specifications were drawn up and implemented for wall 9, whereas remedial measures were used to reconstruct wall 7.

To determine the in situ performance of walls 9 and 7 a monitoring study of the site was undertaken. The goals of the study were to judge the behavior of the walls and to provide a history of performance of the stabilized fill. To accomplish this, various instrumentation was installed behind the two walls in both the stabilized fill and the soil. The instruments used were inclinometers, both vertical and horizontal, earth pressure cells, and load cells.

9.2 Revised Specifications

Although there was no opportunity in the time scale of this project, to issue revised specifications, it is suggested that the wall specifications could be modified, in view of the problems observed. Possible revised specifications might be as follows:

DRAFT ITEM

PORTLAND CEMENT STABILIZED GRAVITY RETAINING WALL

(a) Description. This item specifies materials and construction of portland cement stabilized gravity retaining walls as required by the plans.

(b) Materials.

(1) Portland Cement. Cement will be Type I, II, or III Portland cement conforming to Item 524, "Hydraulic Cement". Use of Types I or III may require additional testing under Item c.

(2) Water. Water will meet the requirements for mixing water in Item 360, "Concrete Pavement".

(3) Waterproofing. Materials for waterproofing will be of the type shown on the plans and will conform to the material requirements of Item 458, "Waterproofing".

(4) Drainage. Filter material and drain pipe will conform to the material requirements of Item 556, "Pipe Underdrains".

(5) Soil to be Stabilized. Coarse grained, durable soils and/or aggregate materials will be used. Material sources will be approved prior to use. The material must conform to the following requirements when tested according to Test Method Tex-110-E:

Sieve	Percent Passing
3.0"	100
1.75"	75-100
No. 4	75-100
No. 40	15-95
No. 200	0-15

The fraction passing the No. 40 sieve will meet the following requirements when tested in accordance with Test Method Tex-106-E:

The Plasticity Index shall not exceed 6.
The liquid limit shall not exceed 35.

If the engineer has reason to suspect that the soil is acidic or contains organic or sulfates in amounts which might adversely affect the strength of the final product, the pH test and/or sulfate tests given in section (j) should be performed. If the pH measured in the pH test of section (j) is less than 12.0 and/or the percent sulfate determined using one of the sulfate determination tests of section (j) is greater than 1.0 percent, the wet/dry and freeze/thaw tests of section (d) must be performed and the Engineer will determine material acceptibility based on these tests.

(c) Equipment. Paragraph 274.3 of Item 274, "Cement Stabilized Base" applies.

(d) Proportioning of Mixes. Cement content shall not be less than 4.5 percent of the dry weight of the soil. The material should be compacted to not less than 95 percent of Compaction Ratio Density, Test Method Tex-114-E. The proportioning is based on strength design. The strength design allows indirect consideration of local climatology and wall geometry. Unless otherwise specified on the plans, a minimum average unconfined compressive strength of 200 pounds per square inch is required at an age corresponding to the expected time of removal of temporary external bracing or at an age of 7 days, whichever is shorter. In addition, the average strength must equal or exceed that indicated in Figure 1 for the expected in situ stress state and age.

(e) Evaluation of Failure by Fracture. If the Engineer has reason to suspect that changing stress states induced within the wall during construction or after completion could induce catastrophic failure by fracture within the wall, the evaluation described in this section should be accomplished. Test Methods Tex-117-E and Tex-120-E and should be used as guides for conducting the test to ascertain whether or not the material complies with the strength requirement shown in Figure 9-1. Note that, as the wall is built up, the age and expected triaxial compressive stress state in the lower layers changes, which necessitates testing at the new conditions. A minimum of three minor principal stress states should be used.

(1a) The unconfined compression test (0 psi confinement) should be conducted for age mentioned above (7 days maximum) and

(1b) for the minimum age expected in the event of early completion of the entire wall (no testing required using ages greater than 90 days if Type II cement is used).

(2) A second test should be conducted at the in situ radial confining pressure expected at the bottom of the wall at the age mentioned above (7 days maximum).

(3) A third test should be conducted at the age mentioned above (7 days maximum) and at the in situ radial confining pressure expected in the wall considering the final overburden, surcharge, and/or expected vehicle loading.

(4) A fourth test should be conducted at the in situ radial confining pressure expected in the wall considering the final overburden, age (no testing required using ages greater than 90 days if Type II cement is used), surcharge, and/or expected vehicle loading.

Three replications of the laboratory tests will be appropriate if consistent results (i.e. within 10 percent of the average) are obtained. The tests at the early age (7 days maximum) can also be use to determine the angle of internal friction to be used for calculating panel facing anchor pullout capacity.

Enter Figure 9.1 with the minor principal stress/unconfined compressive strength ratio for the appropriate age and expected confining pressure on the horizontal axis. Move vertically to the line marking the boundary of the acceptable region. Move horizontally to determine the acceptable ratio of maximum principal stress to the same unconfined compressive strength used in the first step. If the average value of the maximum principal stress/unconfined compressive strength ratio at failure from the laboratory triaxial testing does not exceed this value, precautionary measures should be taken to ensure that the material will not fracture catastrophically.

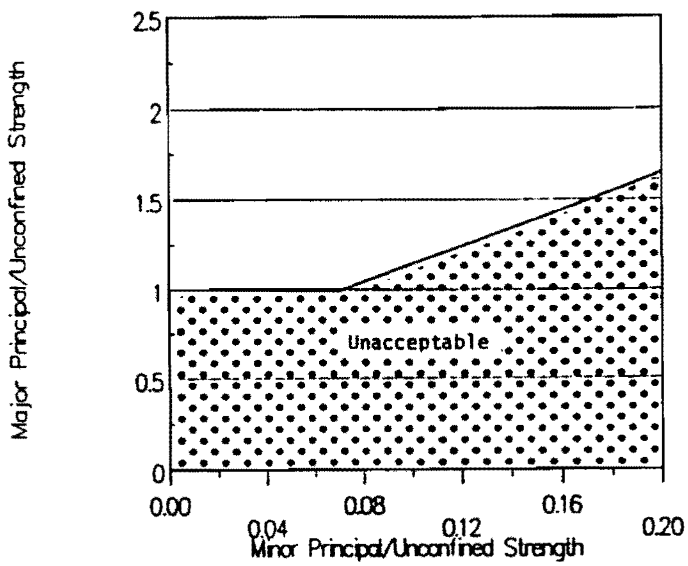


Figure 9.1. Griffith crack strength criterion

Environmental Considerations. If the material will be placed in an area where, or in an application in which, wet/dry cycles can be expected, the maximum weight loss in the wetting and drying test (AASHTO T135 or ASTM D559 but using Texas sample preparation techniques) should not exceed 14 percent. If the material will be placed in a freeze/thaw prone application, the maximum weight loss in the freezing and thawing test (AASHTO T136 or ASTM D560 but using Texas sample preparation techniques) should not exceed 14 percent. The volume change should be less than 2 percent.

Sampling and Testing. In addition to tests mentioned in paragraph 424.4, tests for moisture and density will be made on the material immediately following placement, and a minimum of three specimens will be made for unconfined and confined (one confining pressure minimum) compression tests for each 1000 tons of material or fraction thereof placed each day.

Construction Methods. Construction methods are substantially the same as described in paragraph 423.3 of Item 423 "Retaining Wall" and paragraph 274.6 of Item 274 "Cement Stabilized Base".

If the site investigation indicates that problem soils are present (e.g. swelling clays, or support which may lead to significant differential settlement), the Engineer may wish to specify a foundation treatment such as that found in Item 230 "Roadbed Treatment", Item 260 "Lime Treatment for Materials in Place", or Item 270 "Portland Cement Treatment for Materials in Place".

At the discretion of the Engineer, retardant additives such as calcium lignosulphonate or hydroxylated carboxylic acid can be specified in small amounts (e.g. 0.25 pounds per bag of Portland cement) if significant delays are expected between mixing and compaction or if bonding between layers appears to be less than desirable.

9.3 Instrumentation and Installation

During the construction of wall 9 and the reconstruction of wall 7, field instrumentation was installed to determine the movements and stresses in the soil and the stabilized fill, along with the forces applied on certain wall anchors. The movements were measured using vertical and horizontal inclinometers, and earth pressure cells and load cells measured stresses and forces, respectively. Figure 9.2 to 9.5 illustrate cross sectional and elevation views of walls 9 and 7 indicating the instrumentation used and their locations.

The construction of wall 9 and reconstruction of wall 7 occurred in stages and was performed during late 1990 and early 1991. After the remedial piling was placed beneath wall 7, the leveling pads for the two walls were installed. The construction of the two walls then proceeded with lifts of stabilized fill being placed behind the leveling pads. Additionally, the front panels of the walls were put into place on top of the leveling pads with anchors embedded in the stabilized fill. Props were used to keep the front panels in place and were left for a reasonable amount of time to let the stabilized fill cure and the bond strength of the anchors to develop. As the lifts of stabilized fill and front panels were put into place the field instruments were also installed.

9.4 Inclinometers

(a) Vertical Inclinometers

A Sinco Digitilt Inclinometer, Model 50309-M with Model 50325-M sensor, and a Sinco Digital Indicator, Model 50309, were used to determine the lateral deflections of the vertical inclinometers. The inclinometer sensor was operated in a permanently installed vertical casing. The inclinometer casing consisted of 2.75 in OD plastic pipe that was capped at one end and which contained longitudinal grooves. The sensor was placed in the casing ensuring that the wheels of the sensor lined up with the grooves and was lowered into the casing by an interconnecting cable. The principle of operation is shown in Figure 9.6. Readings were taken every 0.5 m (1.64 ft) to determine the deflection of the casing in the direction of the grooves. The readings were recorded on the indicator and transferred to data sheets manually.

At wall 9 two vertical inclinometers were installed, one in the stabilized fill and the other in the soil (Figure 9.2). The capped end of the inclinometer casings were installed to a depth equivalent to the existing ground surface. The grooves of the casings were aligned parallel (ie. N-S) and perpendicular (ie. E-W) to the wall. These are the expected directions of movement of the wall, stabilized fill and the soil. Stabilized fill and soil were placed in lifts around the inclinometer casings, depending upon their location, to provide support. Figure 9.7 shows a photograph of the installation of the vertical inclinometers.

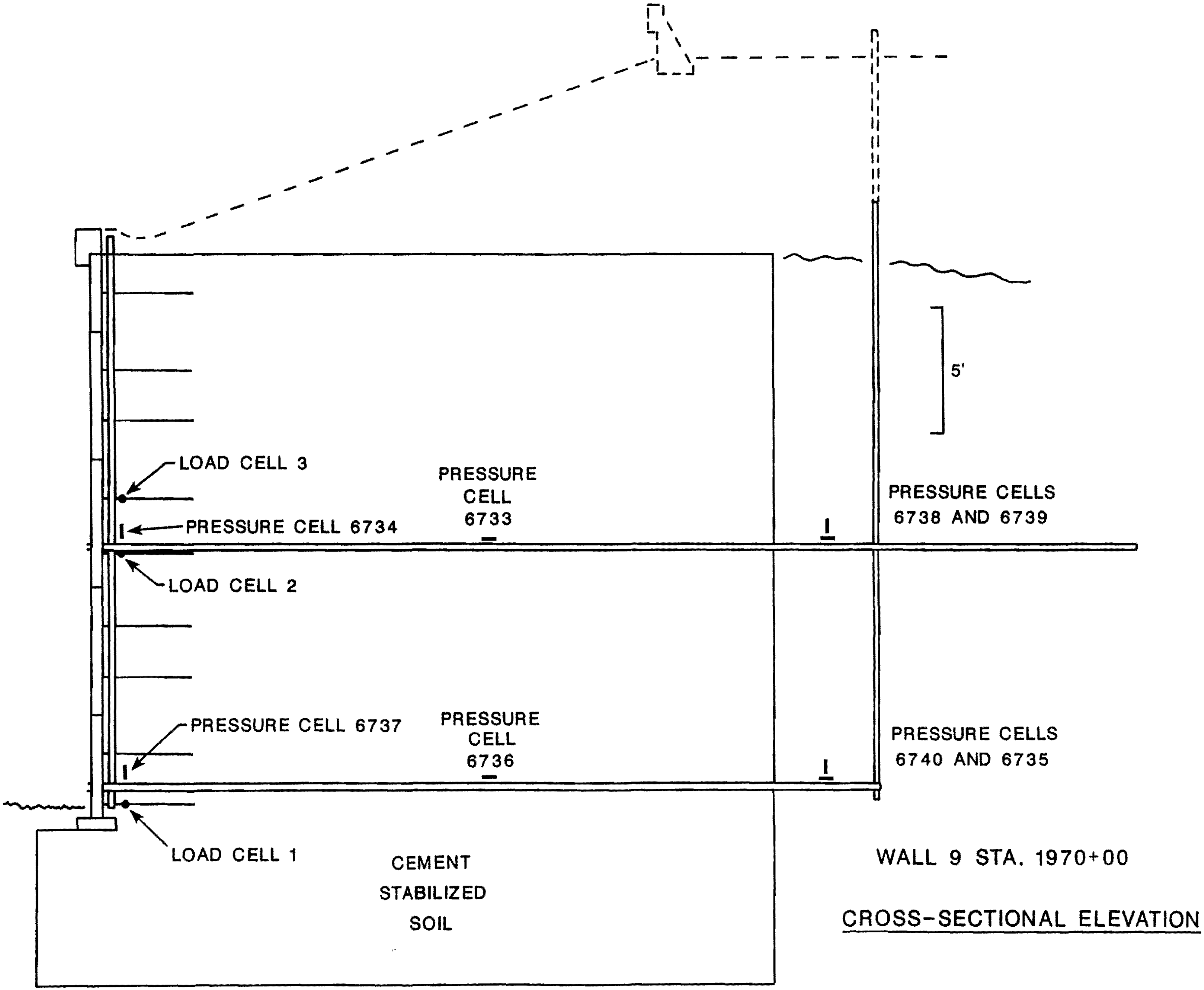


Figure 9.2 Cross-section of wall 9

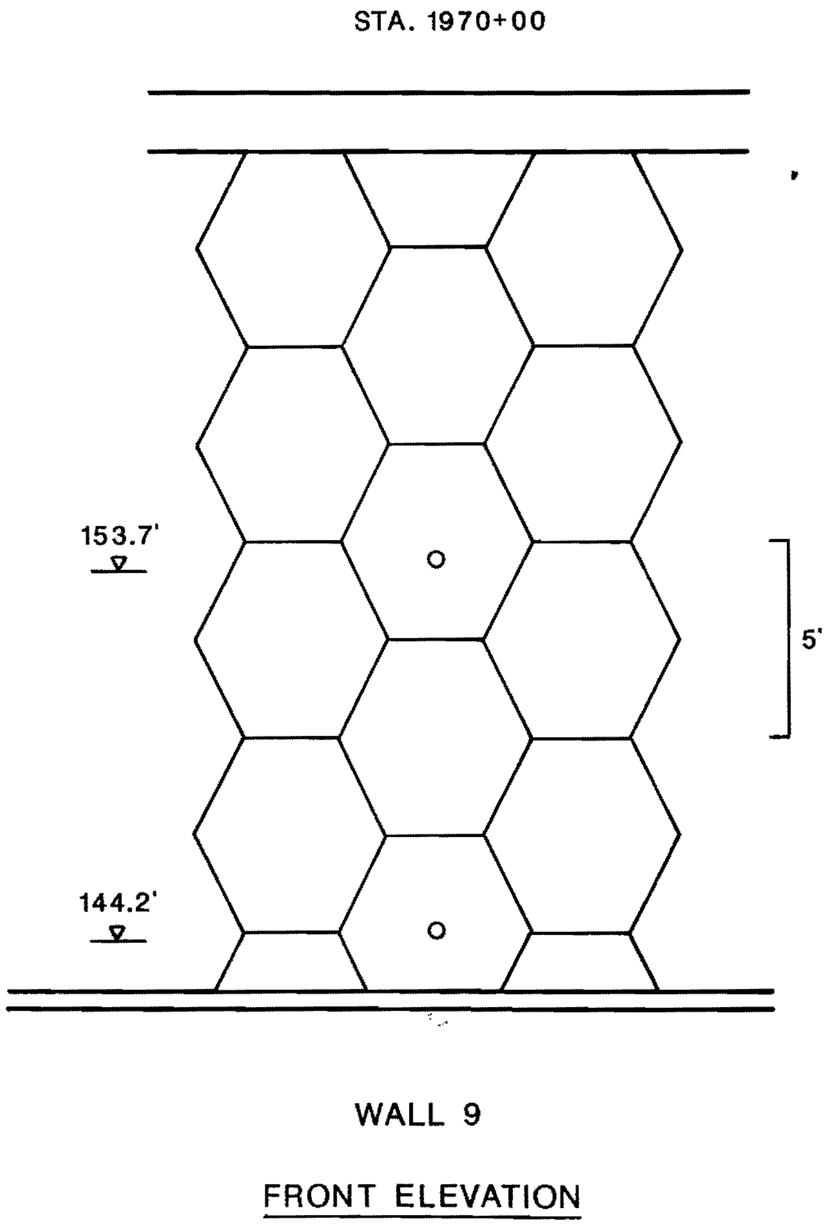


Figure 9.3 Front elevation of wall 9

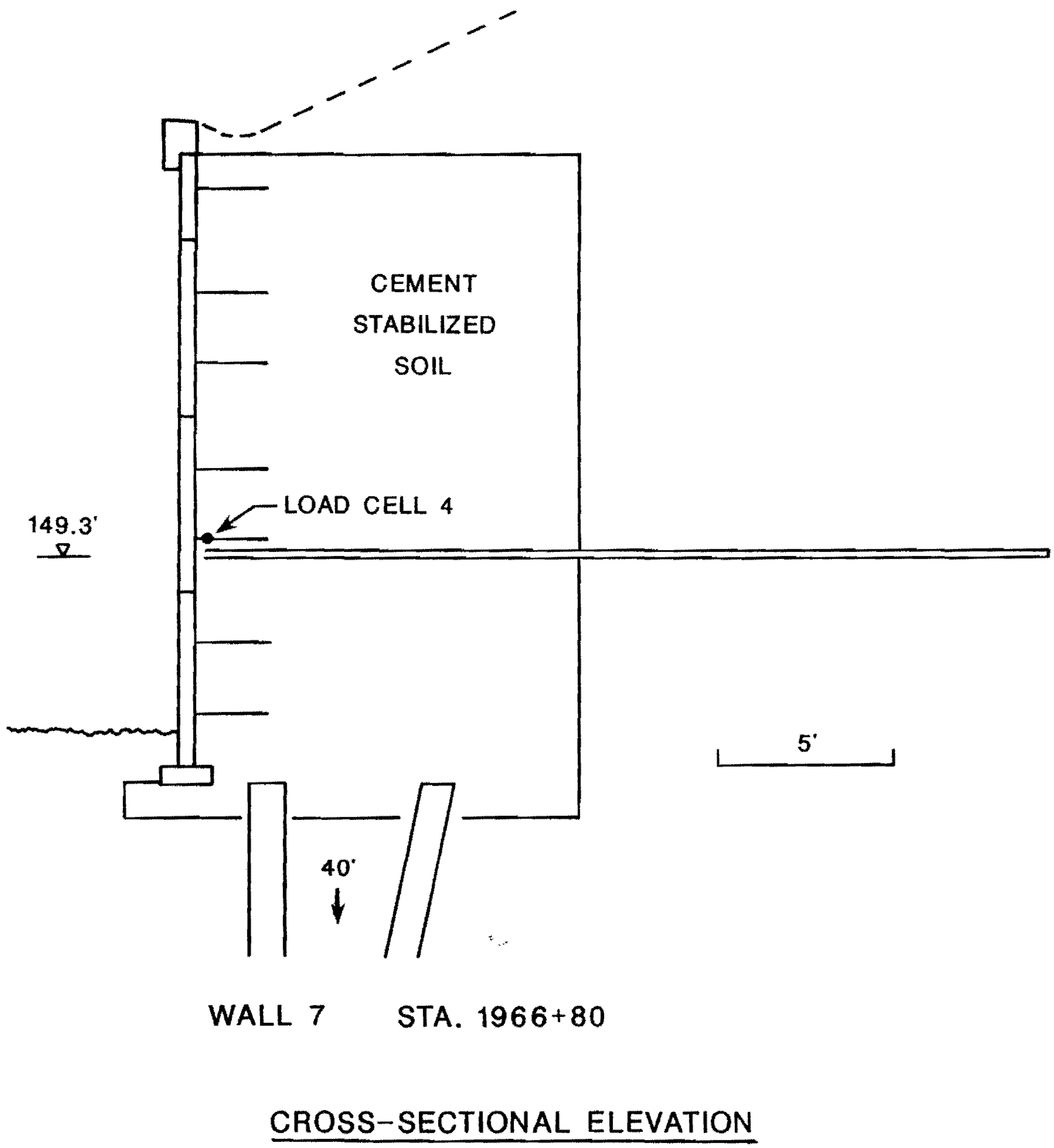


Figure 9.4 Cross-section of wall 7

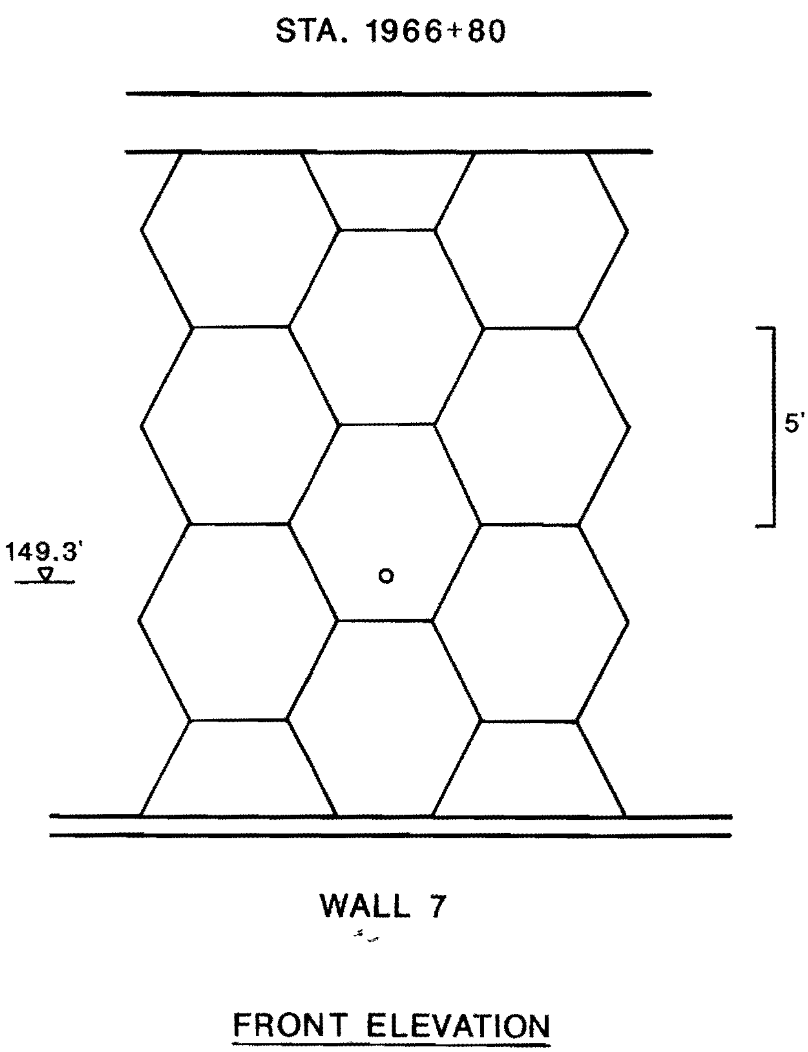


Figure 9.5 Front elevation of wall 7

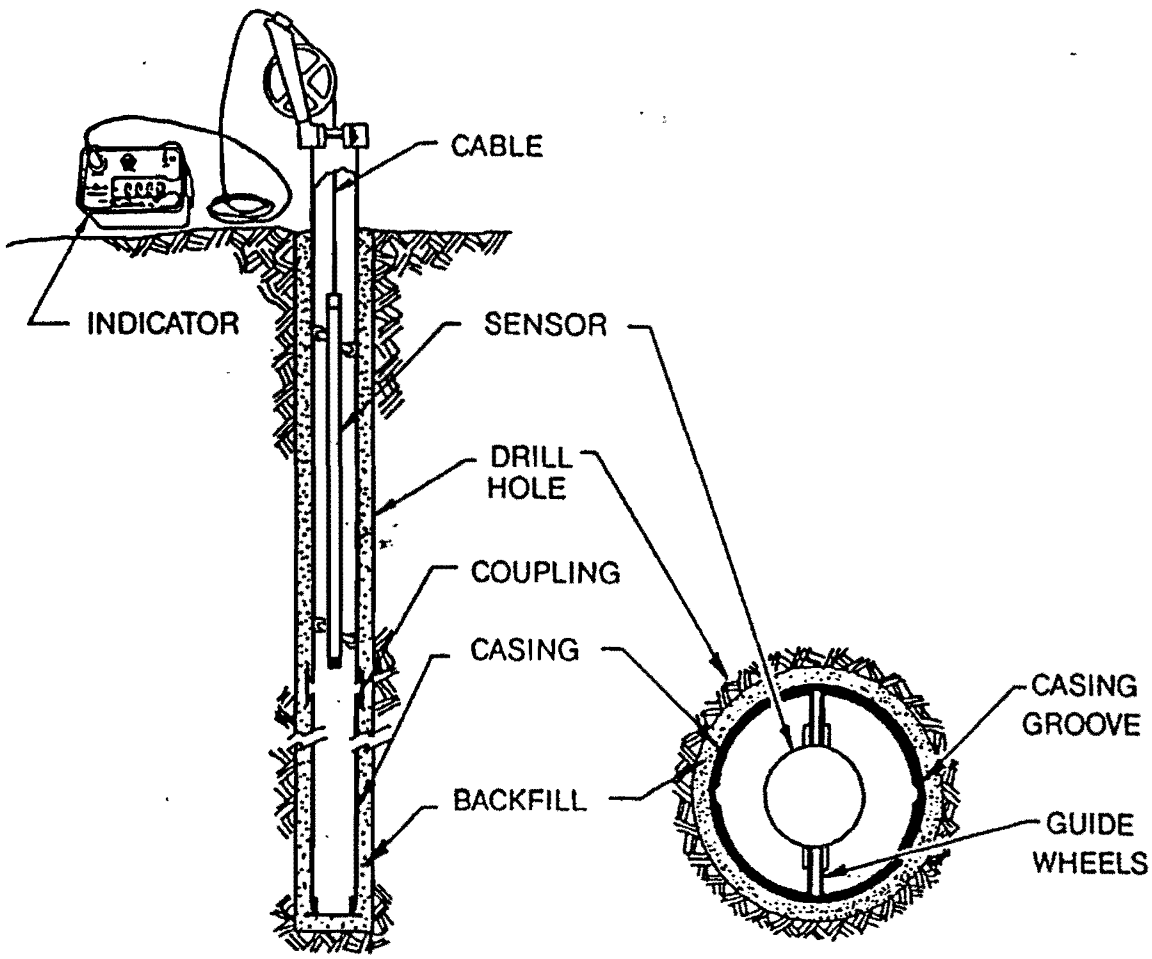


Figure 9.6 Principle of operation of vertical inclinometer

Figure 9.7 Inclinometer installation at half height

(b) Horizontal Inclinometers

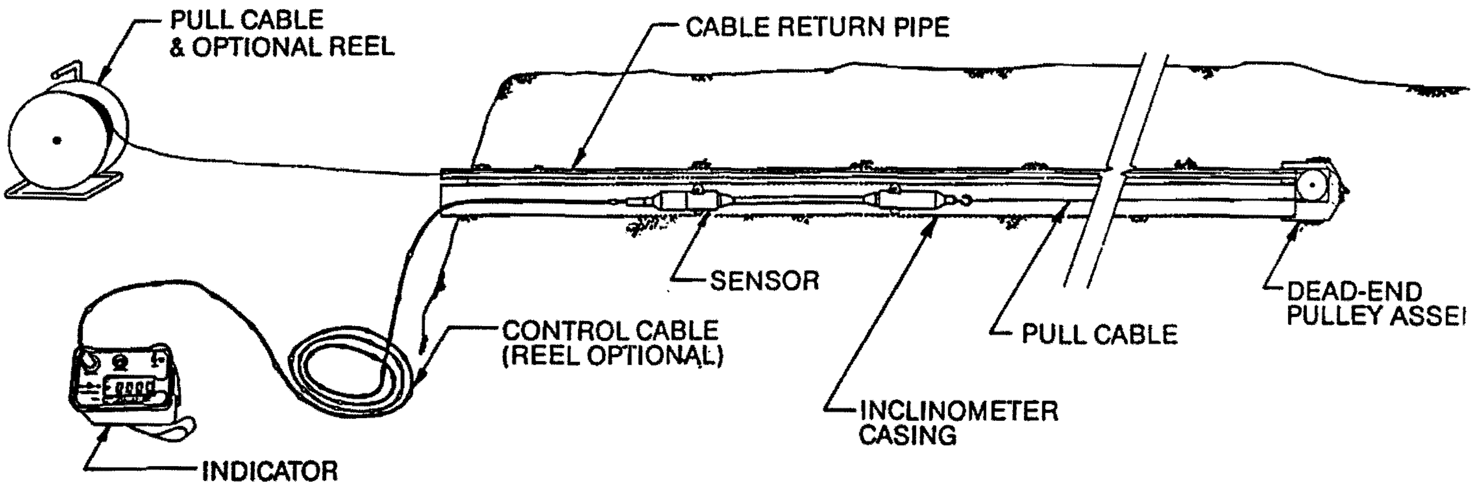
A Sinco Horizontal Digitilt Sensor, Model 50329, and a Sinco Digital Indicator, Model 50309, were used to determine the vertical movement within the stabilized fill and soil. An individual horizontal inclinometer system for walls 9 and 7 consisted of permanently installed horizontal inclinometer casings with a pull cable arrangement. The inclinometer casings consisted of a grooved 3.34 in OD plastic pipe, while the cable wire was stainless steel. The sensor, which was linked to the digital indicator by an interconnecting cable, was attached to the pull cable and placed in the casing. The pull cable was then used to advance the sensor, with readings being taken every 0.5 m (1.64 ft) to determine the deflection of the casing. The principle of operation is shown in Figure 9.8.

Since the casing was only accessible from one end, a dead end pulley assembly was used to return the cable. For this a second plastic pipe of 0.75 in OD was attached parallel to the casing to house the returning cable. At the inaccessible end the two pipes were connected to a steel box, which housed the pulley. Consequently, the pull cable ran through the inclinometer casing, around the pulley and returned in the attached plastic pipe. The dead end pulley system was used to avoid the risk of the probe or sensor becoming entangled with the return wire. Figure 9.9 shows a close-up of the end of a horizontal inclinometer installation, with the dead end pulley system.

For both walls 9 and 7, the horizontal inclinometers extended from the face of the wall to the soil (Figures 9.2 and 9.4). During installation of each of the inclinometers a small channel was dug, at the appropriate elevation, for the inclinometer casing and the dead end pulley assembly. The dead end pulley assembly was placed in the soil (Figure 9.9). The grooves of the inclinometer casing were placed vertically (ie. D-U) and parallel to the wall (N-S), so that the relative and absolute settlement and vertical heave could be determined. Stabilized fill and soil were then placed over the inclinometers, in the appropriate locations, to permanently embed them.

9.5 <u>Anchor Load Cells</u>

Four electrical resistance load cells were used, three on wall 9 and on wall 7, to determine the forces on the wall anchors. The load cells consisted of two electrical resistance strain gauges, bonded to the outer periphery of a steel cylinder. The strain



Horizontal Inclinometer System with Dead-End Pulley Assembly

Figure 9.8 Principle of operation of horizontal inclinometer

Figure 9.9 Photograph of pressure cells and return pulley

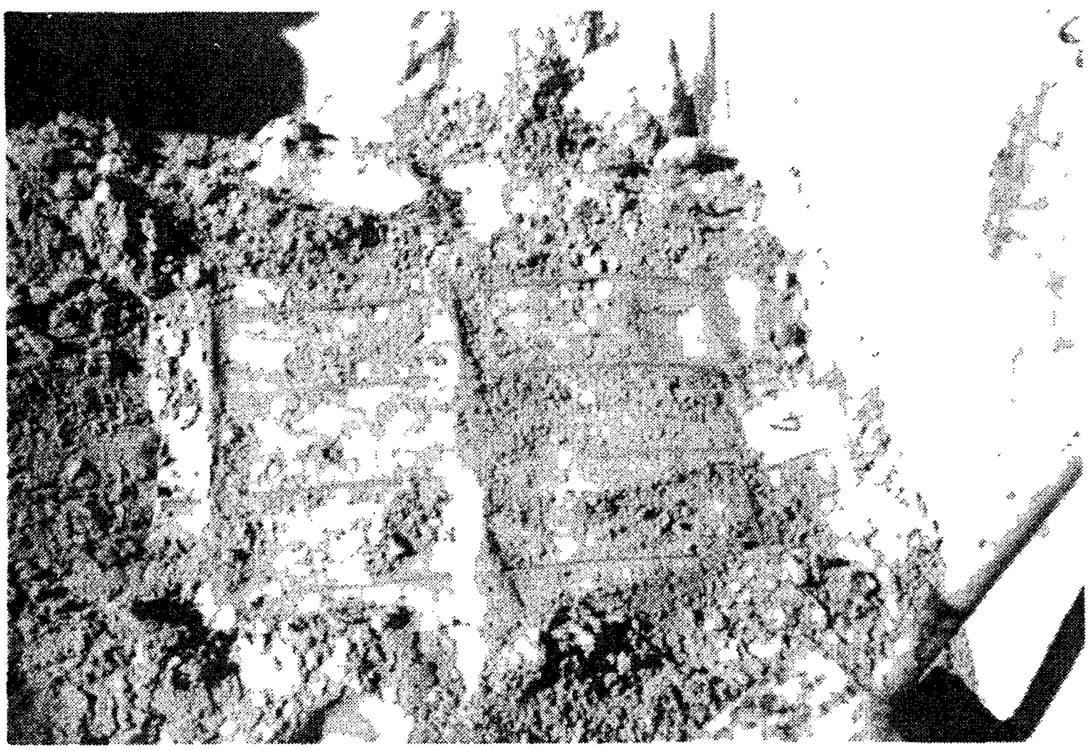


Figure 9.10 Photograph of anchor load cells

gauges were oriented parallel and perpendicular to the axis of the cylinder to measure the axial and tangential strains. The strain gauges were connected to form a single full bridge network, which reduces errors by integrating the individual strain gauge outputs. The strain gauges and full bridge were wrapped in electrical tape around the steel cylinder. The entire assembly was placed inside a larger diameter cylinder. This was done to protect the gauges from mechanical damage. A waterproofing compound was used to fill the void between the cylinders and to protect the gauges from water damage.

Each load cell was tested at Texas A&M University, before installation, to determine the calibration curve for the resistance measured and the corresponding force applied. In the field a 12 volt battery supplied the current to the strain gauges. The resistance change of the strain gauges due to deformation was read in millivolts and measured using a volt meter. The resistance change was then converted to the corresponding force using the calibration curve for the individual load cell.

The installation of the load cells in the field consisted of rigidly attaching the individual load cells to the wall anchors. For convenience the load cells were mounted to the wall anchors adjacent to the locations of the horizontal inclinometer casings, so that the wires for the load cells could be run parallel to the casings. Stabilized fill was then placed around the anchors and the load cells, so that when the fill cured it would bond to both of them. Figures 9.2 and 9.4 illustrate the locations of the load cells for the two walls. A photograph of an installed load cell is shown in Figure 9.10.

9.6 Earth Pressure Cells

A total of eight earth pressure cells were installed in both the stabilized fill and the soil behind wall 9. The earth pressure cells were made by Kulite Sensors Limited and are of the diaphragm type. A diaphragm type of earth pressure cell is a circular disk in which the upper surface has a flexible circular membrane on it that deflects under the external soil pressure. The deflection of the membrane is measured by the strain gauges bonded to the lower surface of the membrane. These particular devices incorporated a full bridge of semi-conductor strain gauges diffused into a silicon diaphragm, for high sensitivity, and required only an input voltage to produce an appropriate output signal. The resistance changes measured by the strain gauges were recorded on a Sinco Digital Indicator, Model 50309, and were converted to stress using the factory provided calibration curve.

The earth pressure cells were installed at the same time as the horizontal inclinometer casings. The wires from the earth pressure cells were run along the inclinometer casings. The locations and orientations of the earth pressure cells were selected to try and determine the stress state in the soil, the vertical stress in the stabilized fill and the lateral stress on the wall. Consequently, the earth pressure cells were oriented both horizontally and vertically, with the vertical earth pressure cells having their axes parallel to the wall. Figure 9.2 illustrates the locations and orientations of the earth pressure cells. A photograph of an installed earth pressure cell is shown in Figure 9.9.

10. FIELD BEHAVIOR AND DATA REDUCTION

10.1 Inclinometers

Four inclinometers were installed behind wall 9 and one inclinometer was installed behind wall 7. All of the inclinometers are functioning properly, except that the inner vertical inclinometer in the soil behind wall 9 has not yet been used, because a construction road was built adjacent to it leaving its upper portion unsupported. Table 10.1 indicates the dates of the initial readings of the inclinometers, as well as the directions of the fixed reference orientations for the positive deflections. Each time the inclinometer is surveyed the spring loaded wheels are first set in the grooves of the casing parallel to the reference direction and the readings are taken every 0.5 m (1.64 ft). For completeness, the sensor is rotated 180°and readings then taken again. This is so that the algebraic difference of the two readings is equivalent to the average of the two sets. This eliminates the zero drift between the readings. For each of the positive inclinometer directions the algebraic difference of the initial reading is subtracted from each of the differences of the subsequent readings to yield the change in slope of the inclinometer casing.

To obtain the cumulative slope change, the slope changes at each interval for an individual reading were summed starting from the reference end of the casing (i.e. the bottom of the casing for the vertical casings and the face of the wall for the horizontal casings). The cumulative slope change values at each interval were then multiplied by the instrument scale factor to yield the deflection. In the case of the vertical inclinometers, the deflection is the absolute deflection, however for the horizontal inclinometers this yields the relative deflection. To obtain the absolute deflection, surface surveying measurements of the reference end of the inclinometer casing (i.e. at the wall) must be added to the relative deflection. Using the above method for reducing the data ensures that the positive deflection is parallel to the reference direction.

Figure 10.1 shows a photograph of horizontal inclinometer measurements being taken on wall 9.

TABLE 10.1 - Inclinometer orientations

Instrument	Date of Initial. Reading	Positive Reference Direction	Positive Reference Direction
Horiz. 1 - lower	Dec. 20, 1990	Down	North
Horiz. 2 - upper	Jan. 11, 1991	Down	North
Vert. 1 - outer	Dec. 20, 1990	East	North
Vert. 2 - inner	Not Used Yet		
Horiz. 3 - wall 7	Apr. 4, 1991	Down	North

10.2 Anchor Load Cells

The load cells installed on the anchors behind the walls are working properly, except that two of the load cells (1 and 2) appear to have shorted out. The cause of the short is unknown, although in the case of load cell 1, it is probably due to becoming submerged under the new watertable. To determine the force on the anchors the resistance change in the strain gauges within the load cells was measured. These readings were converted to a force using the calibration curves.

10.3 Earth Pressure Cells

The earth pressure cells installed behind wall 9 are all working and recording the stresses in the soil and stabilized fill. The initial resistance changes in the strain gages within each of the pressure cells were recorded on December 11, 1990. Additional readings were taken at select times thereafter. To determine the stress level on each pressure cell, the initial reading is subtracted from each reading taken and then is converted to stress using the factory supplied calibration factor.

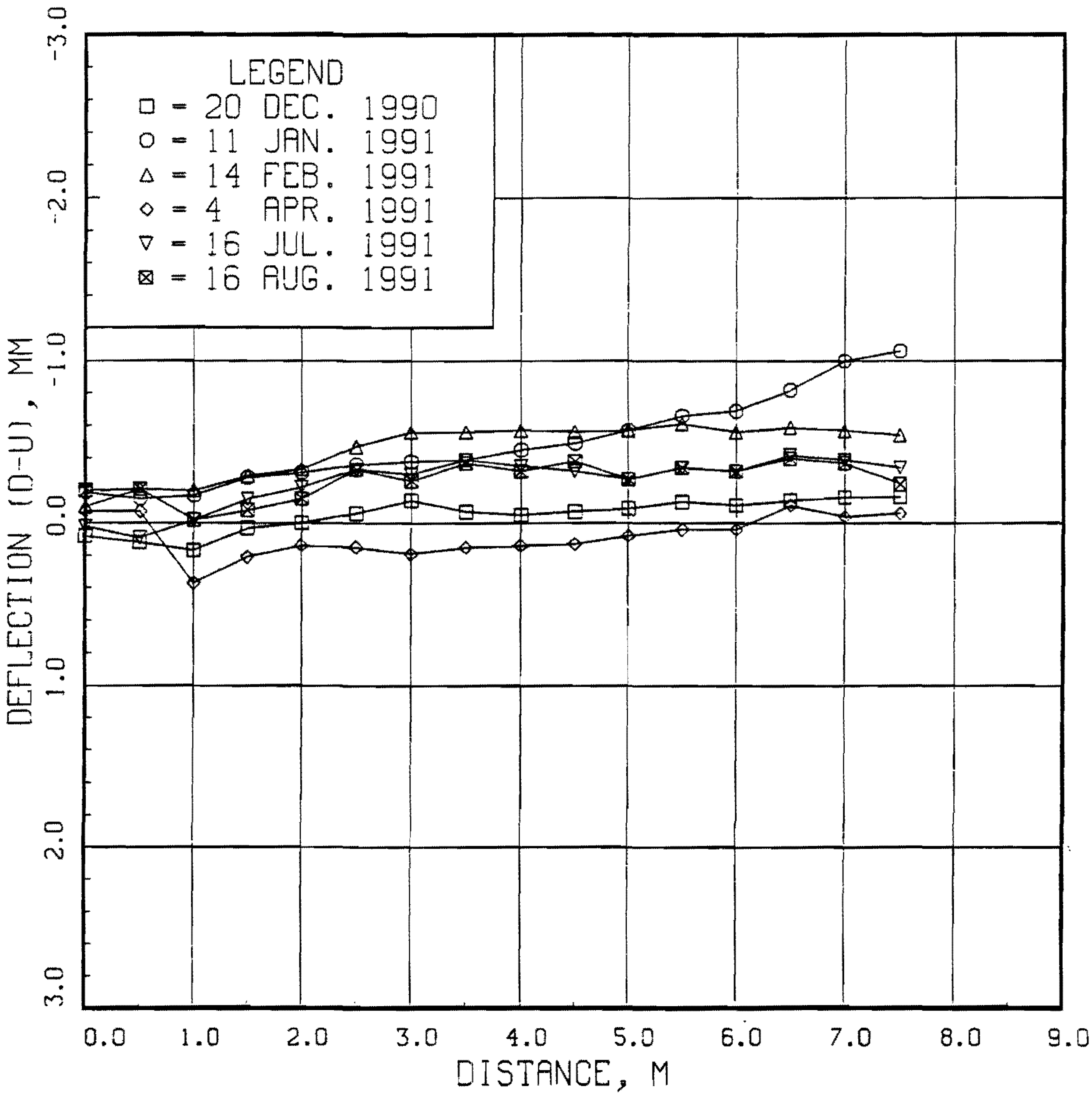


Figure 10.2 Relative deflection, horizontal inclinometer 1, wall 9

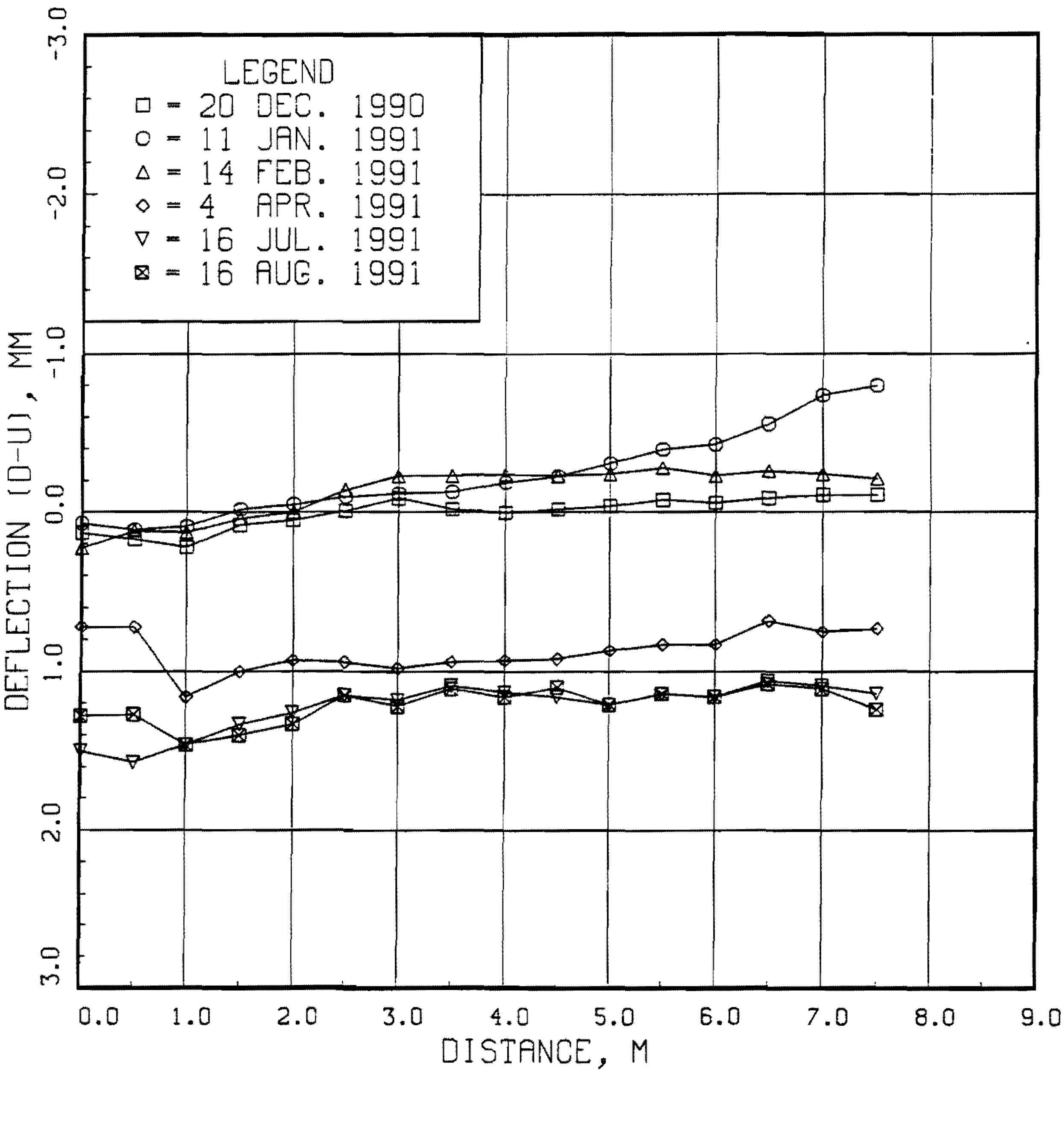


Figure 10.3 Absolute deflection, horizontal inclinometer 1, wall 9

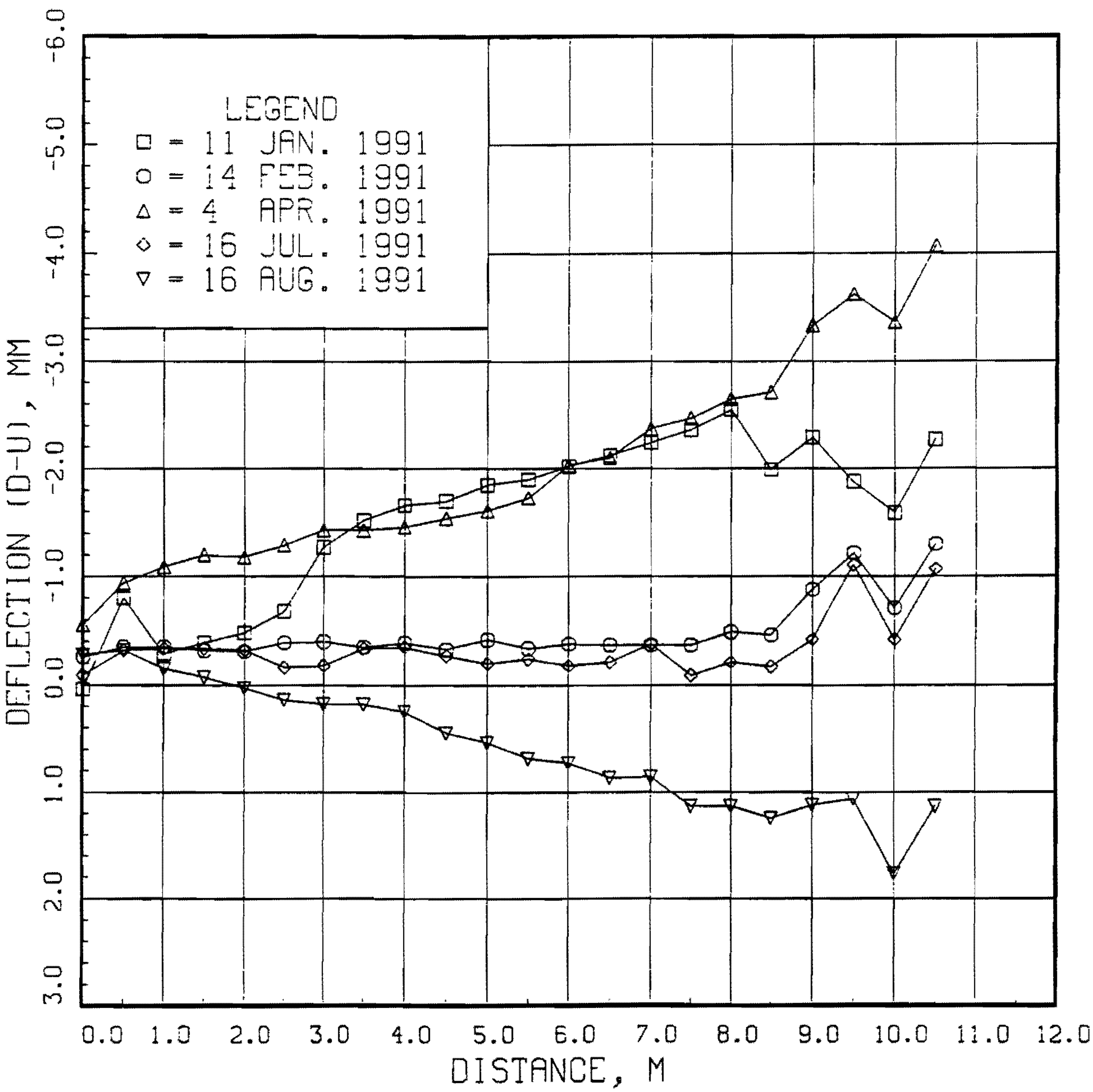


Figure 10.4 Relative deflection, horizontal inclinometer 2, wall 9

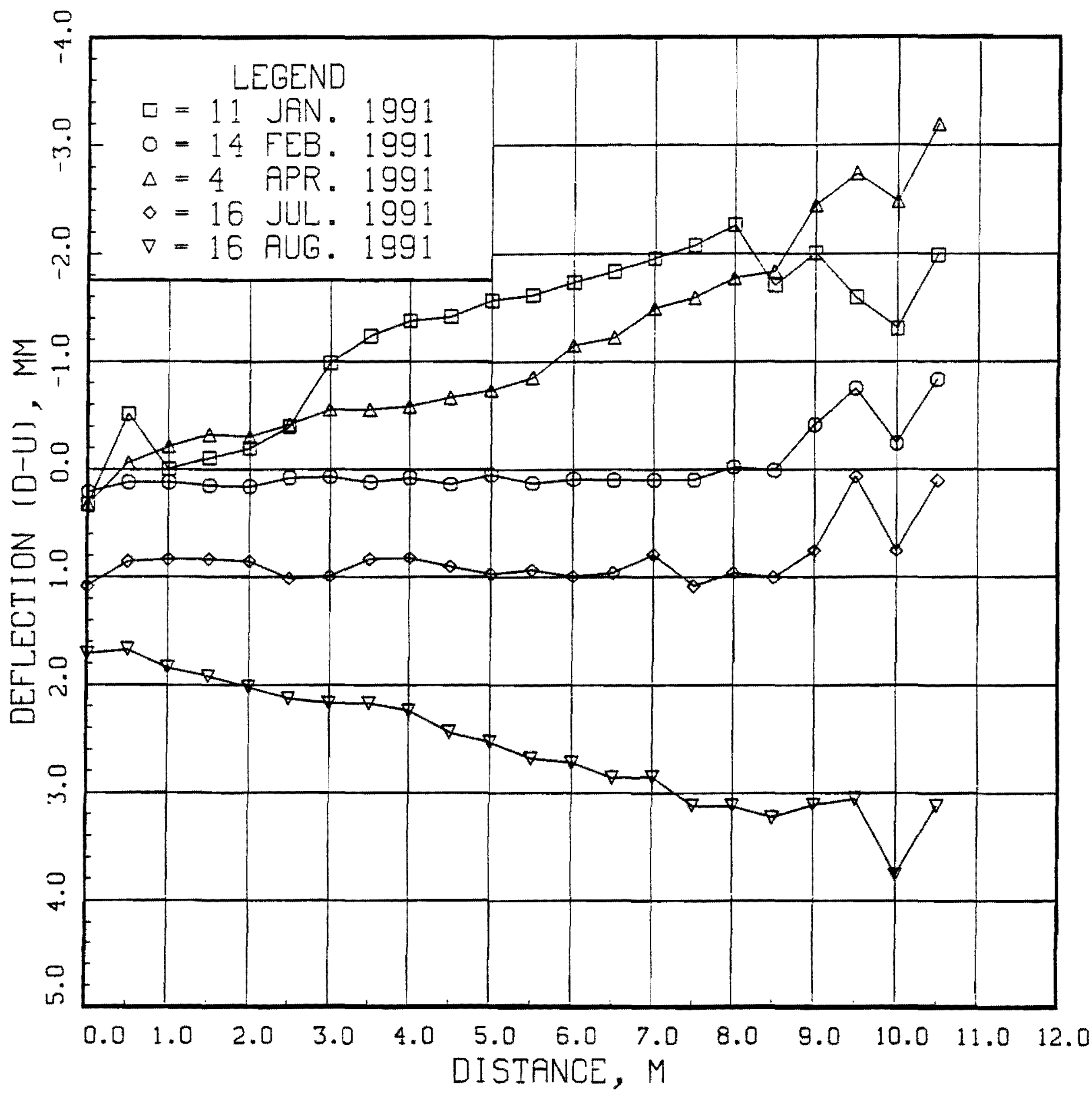


Figure 10.5 Alsolute deflection, horizontal inclinometer 2, wall 9

10.4 Results - Wall 9

(a) Horizontal Inclinometers

The settlement and/or vertical heave for the stabilized fill and soil is illustrated by the relative and absolute deflections of the two horizontal inclinometer casings shown in Figures 10.2 to 10.5. In general, both the stabilized fill and the soil behind the wall are settling uniformly. The magnitude of the settlement is 1.65 mm (0.065 in), which quite small. In addition, the wall itself is settling as essentially a rigid body. Table 10.2 indicates the change in height between the two inclinometer holes at the wall face for various times. The values were determined by subtracting the difference of the absolute deflection values for the two inclinometers, at the wall face (i.e. zero distance), from the initial distance between the inclinometers (i.e. 9.5 ft., see Figure 9.3).

TABLE 10.2 - Separation between inclinometers

DATE	SEPARATION
1/11/91	9.49 ft.
2/11/91	9.50 ft.
4/4/91	9.51 ft.
7/16/91	9.50 ft.
8/16/91	9.48 ft.

(b) Vertical Inclinometer

The deflections of the vertical inclinometer, both parallel and perpendicular to the wall are shown in Figures 10.6 and 10.7. The deflection of the inclinometer perpendicular to the wall indicates that the upper portion of the wall is rotating towards the wall face, whereas the lower portion is being deflected into the stabilized fill (Figure 10.7). The magnitude of the differential deflection, between the top and center of the wall, is 1.5

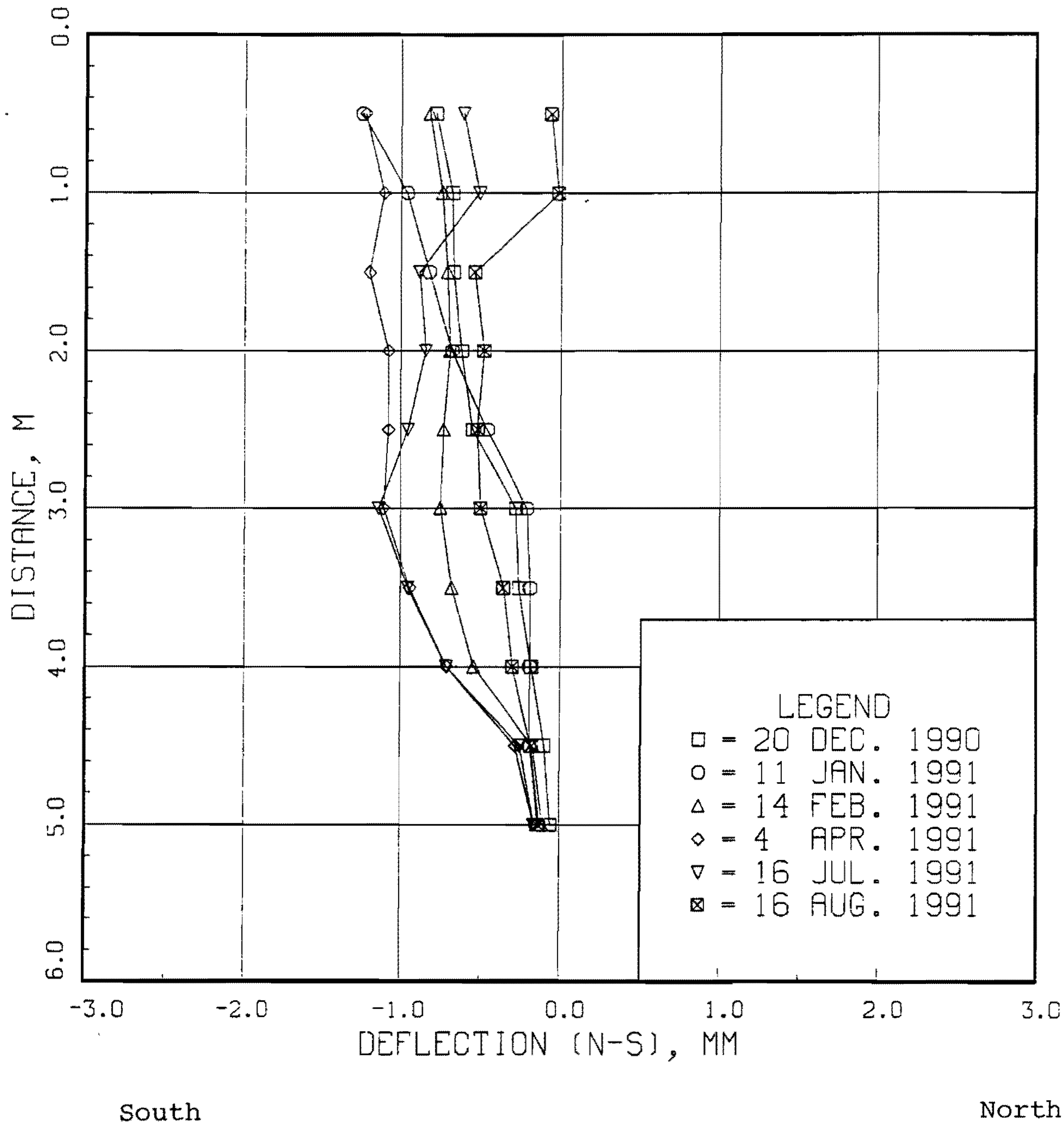


Figure 10.6 Vertical inclinometer deflection, parallel to wall 9

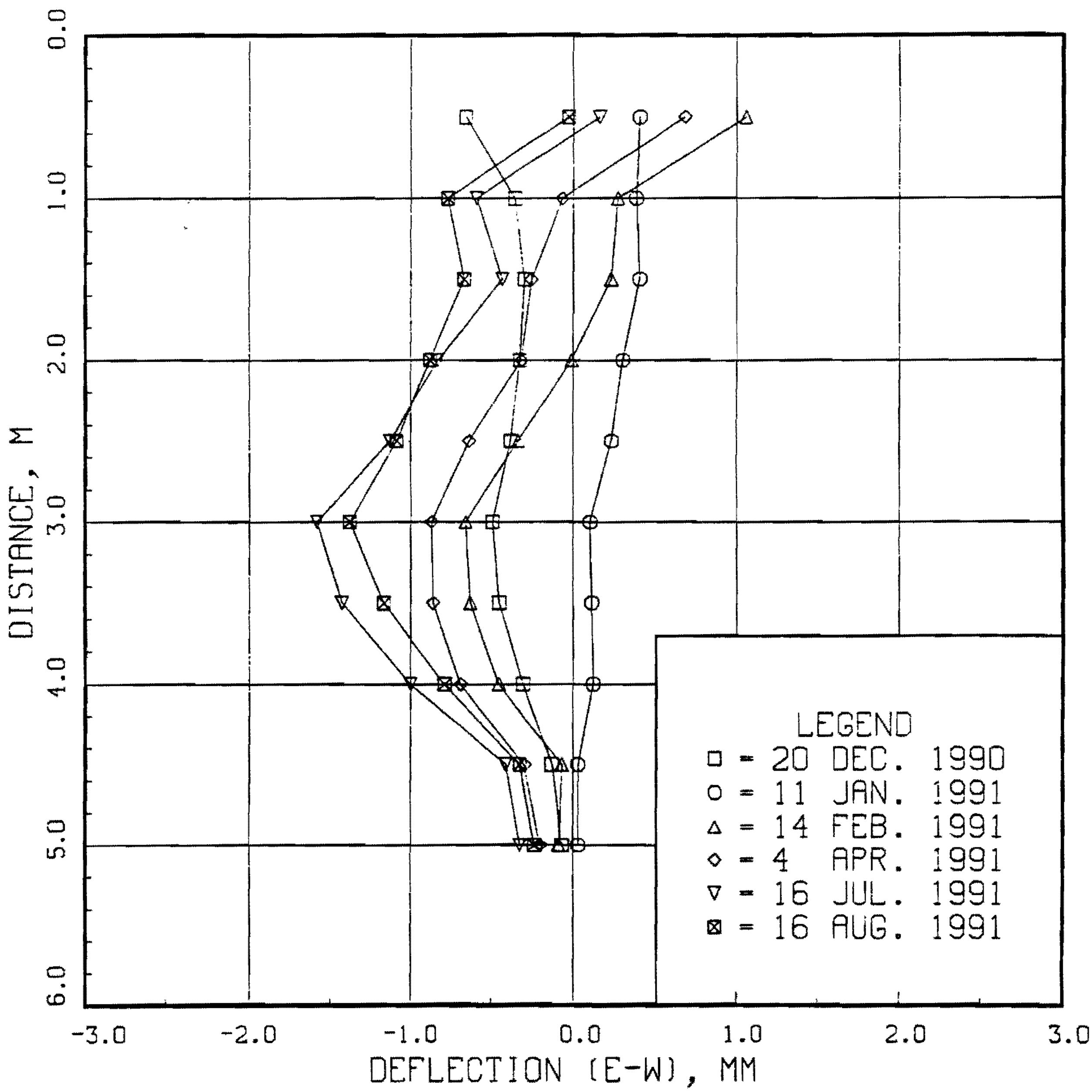


Figure 10.7 Vertical inclinometer deflection, perpendicular to wall 9

mm.(0.059 in.), which is the same as the maximum central deflection. It should be noted that the maximum central deflection occurs at approximately the same height as the top horizontal inclinometer. In addition, the deflection of the inclinometer parallel to the wall indicates that, after an initial southward deflection, the stabilized fill is being displaced northward, with the top meter of the fill having the greatest displacement (Figure 10.6).

(c) Earth Pressure Cells

Table 10.3 indicates the variation in stress with time for the various earth pressure cells. Initially the stresses recorded are largely as expected for gravity loading. However with the passage of time the stresses decrease and actually become tensile for some of the pressure cells. While the exact cause of this behavior is not known, a possible explanation for this could be that arching of the soil and stabilized fill is occuring around the pressure cells leading to stress concentration of the normal stress at the edges of the cells, instead of being uniformly distributed across it. Consequently, this could cause the pressure cells to underestimate the stresses. The magnitude of the underestimation would depend upon the ratio of the stiffness of the membrance to that of the stabilized fill and soil.

(d) Load Cells

The variation in the forces on the anchors with time for the monitored anchors is given in Table 10.4. From this table, it is clear that the forces are increasing with time. As expected, the forces on load cells 2 and 3, which roughly correspond to the location of the maximum deflection of the wall, are higher than that in load cell 1 at the base of the wall.

Table 10.3 Pressure cell table

Date	Pressure Cell # 6733 (psi)	Pressure Cell # 6734 (psi)	Pressure Cell # 6735 (psi)	Pressure Cell # 6736 (psi)
1/11/91	17.55	17.10	11.70	10.35
2/14/91	17.55	7.20	4.95	9.90
4/4/91	17.75	-6.80	4.05	11.70
7/16/91	2.25	-30.60	1.35	4.50
8/16/91	1.35	-29.70	4.05	1.35

Date	Pressure Cell # 6737 (psi)	Pressure Cell # 6738 (psi)	Pressure Cell # 6739 (psi)	Pressure Cell # 6740 (psi)
1/11/91	22.50	16.20	22.05	9.00
2/14/91	12.60	16.65	21.15	0.45
4/4/91	4.95	14.85	16.20	4.05
7/16/91	-15.30	-9.45	-7.65	-5.85
8/16/91	-16.65	-14.40	-12.60	-5.85

Table 10.4 Load cell table - wall 9

Date	Load Cell #1 (kg)	Load Cell #2 (kg)	Load Cell #3 (kg)
12/11/90	20.53	159.49	47.22
1/11/91	60.49	172.91	103.63
2/14/91	90.48	233.11	146.00
4/4/91	109.72	299.61	shorted
7/16/91	161.89	527.07	shorted
8/16/91	shorted	347.89	shorted

Table 10.5 Load cell table - wall 7

Date	Load Cell #4 (kg)
12/11/90	
1/11/91	
2/14/91	
4/4/91	13.71
7/16/91	25.69
8/16/91	40.14

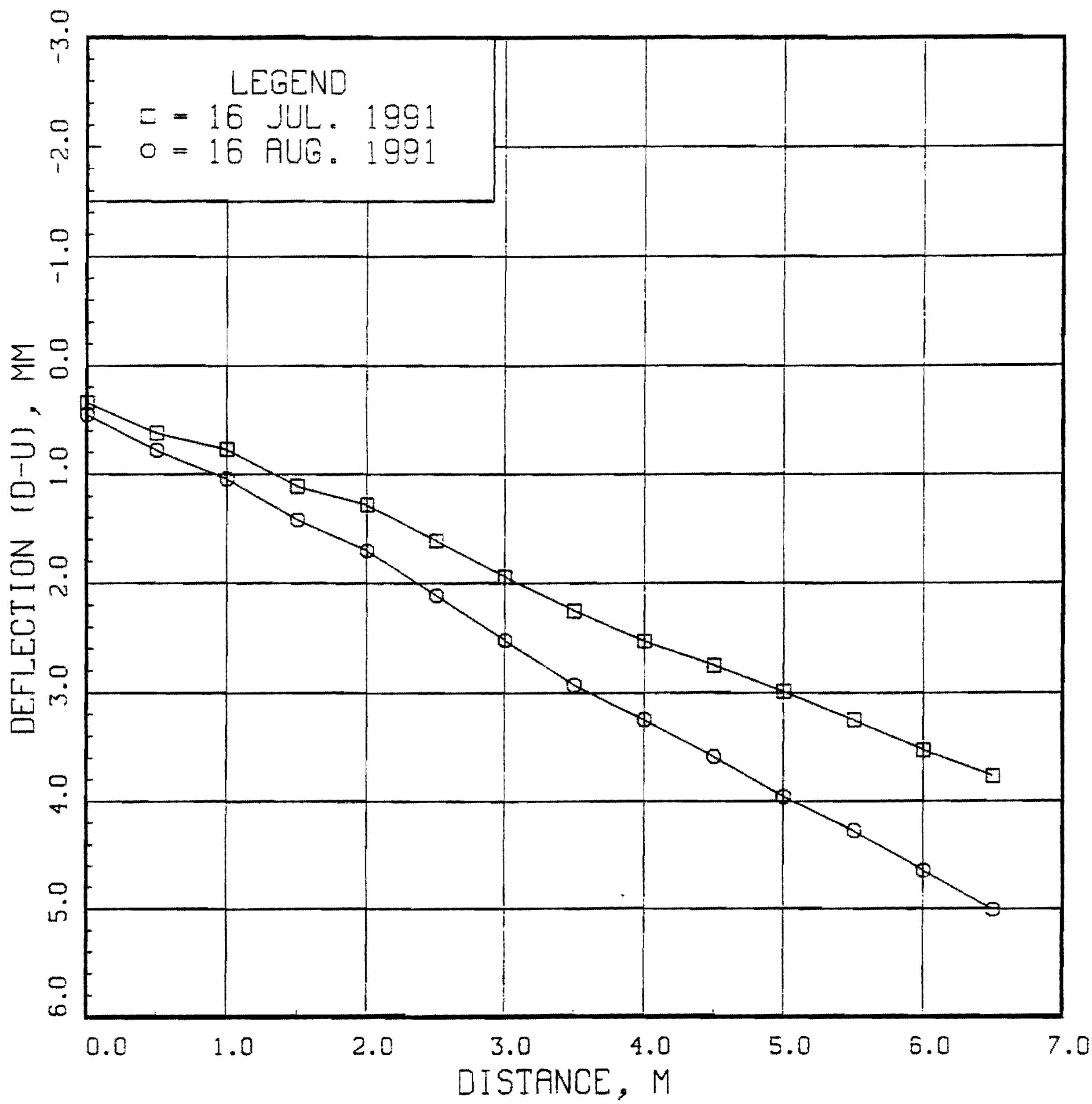


Figure 10.8 Relative deflection, horizontal inclinometer, wall 7

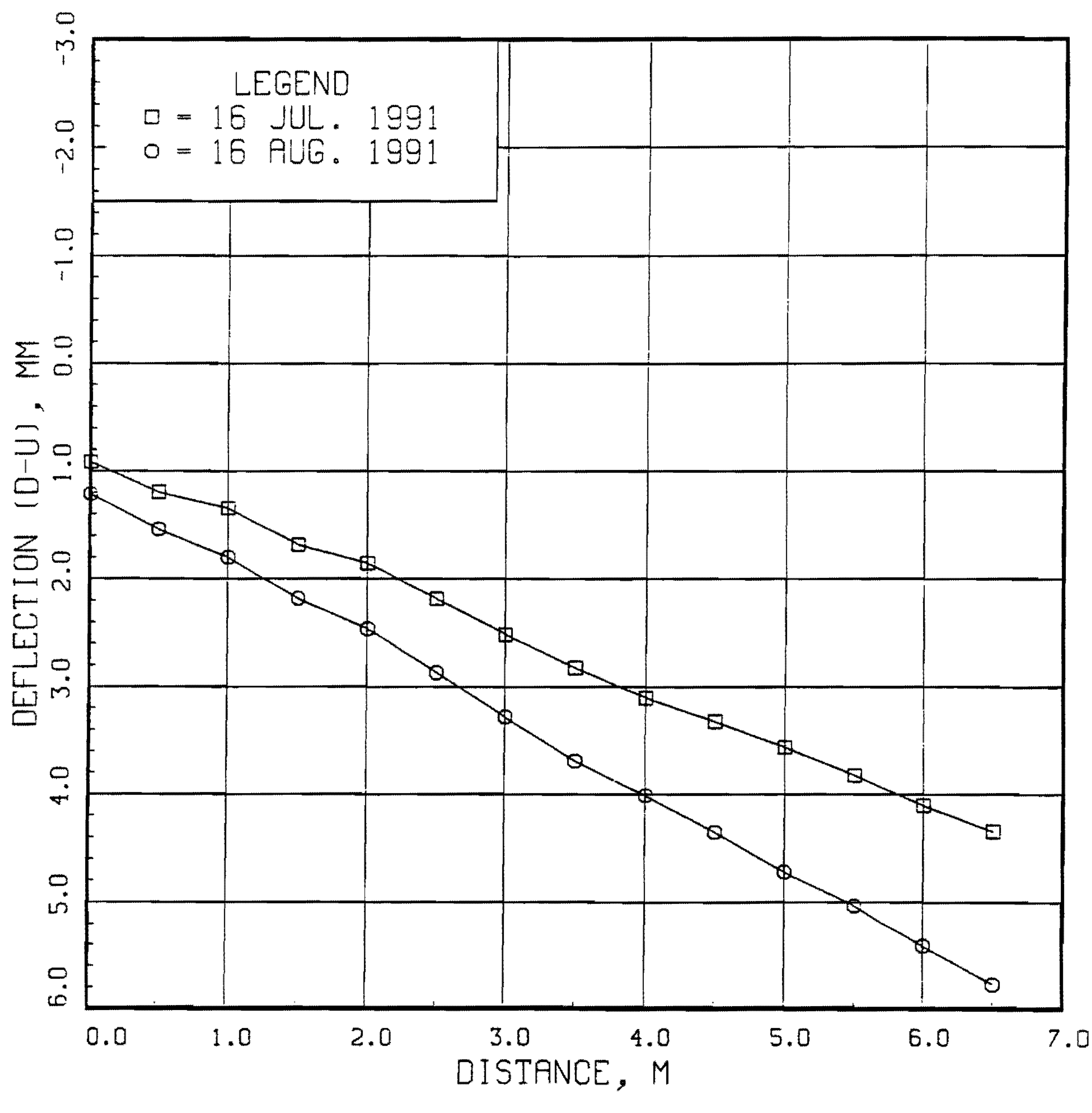


Figure 10.9 Absolute deflection, horizontal inclinometer, wall 7

10.5 Results - Wall 7

(a) Horizontal Inclinometer

The relative and absolute deflections of the horizontal inclinometer are shown in Figures 10.8 and 10.9. Figure 10.9 indicates that the settlement of the stabilized fill and soil increases approximately linearly with distance away from the wall face. The magnitude of the maximum settlement is 5.8 mm. (0.228 in.).

(b) Load Cell

Table 10.5 indicates the variation in anchor force with time. As noted above for the anchors in wall 9, the force in the anchor is increasing with time.

10.6 Summary

The results of the instrumentation readings indicate that the displacements in the stabilized fill and soil are small, on the order of 0.04 to 0.2 in. (1 to 5 mm.). The largest contribution to the displacements is from settlement, which is uniform for wall 9 and varies approximately linearly with distance away from the wall face for wall 7. The stresses in the stabilized fill and soil and the anchor forces are as expected so far.

11. CONCLUSIONS

Full-scale experimental retaining walls have now been constructed in Texas, using mechanical stabilization of the backfill material almost entirely by the addition of cement, rather than be reinforcing strips. Prevailing practice has been to incorporate significant amounts of cement in fill material anyway, largely as a way of preventing erosion problems during heavy rainfall and to guarantee compaction densities. Only nominal anchors are required to retain the facing panels, which essentially just provide a finished appearance and may be either segmental or one-piece.

Conventional stability analysis of this design is in principle very simple, as the cross-section can be treated as a monolithic gravity retaining wall, provided that sufficiently high soil-cement strengths are achieved. The problem of what intact strenth is necessary is still a difficult one. Finite element analysis has indicated that a factor of safety of at least 3 should be provided against crushing, in order to allow for the stress concentration that normally occurs at the front foot of the wall. In practice, a factor of safety of at least 5 has so far been provided against crushing.

The first full-scale wall constructed outside Houston had significant practical problems. The most dramatic of these was associated with considerable sensitivity to differential settlement. Compressible foundation soils caused a localized failure during construction by major cracking and separation of the facing panels in one location.

Although new specifications were written for this design, the contractors had a tendency to ignore several key provisions, in some cases causing distress, which had to be expensively repaired.

The second wall was constructed after considerable attention to detail, and has performed satisfactorily to a finished height of 28 ft, as can be seen in a recent photograph (Sept. 91) in Figure 11.1. Field instrumentation was installed to verify behavior, and the final cost of around $20 per square foot of wall promises to be competitive.

Other possible advantages of this concept are that it may also be quite easy to incorporate variable wall cross-sections into the design. These cross-sections may be variable both in elevation and in plan (which is to some extent done already), so as to optimize the use of material.

In general, the overall conclusions of this study are that cement stabilized soil retaining walls will work, with thorough engineering. However, significant differences exist in construction practice, as compared to conventional reinforced earth walls (even though they seem very similar). Contractors must be fully appraised of these differences, even though they may be very experienced in mechanically stabilized construction. Care is also required in using this design in soft foundation conditions, as it is not as ductile as mechanically stabilized designs.

12. ACKNOWLEDGEMENTS

In addition to the help and assistance of the Bridge Division of the Texas Department of Transportation (technical contact - Mark P. McClelland, Austin), the help of the following individuals is gratefully acknowledged:-

Mr. Mike Ho of the District 12 soils laboratory in Houston, Mr. Ed Suchicki of D-12, Steve Simmons, Marion Noski and the other D-12 field personnel at Cypress-Fairbanks, and Don Harley and Andy Munoz Jr. of the FHWA.

13. REFERENCES

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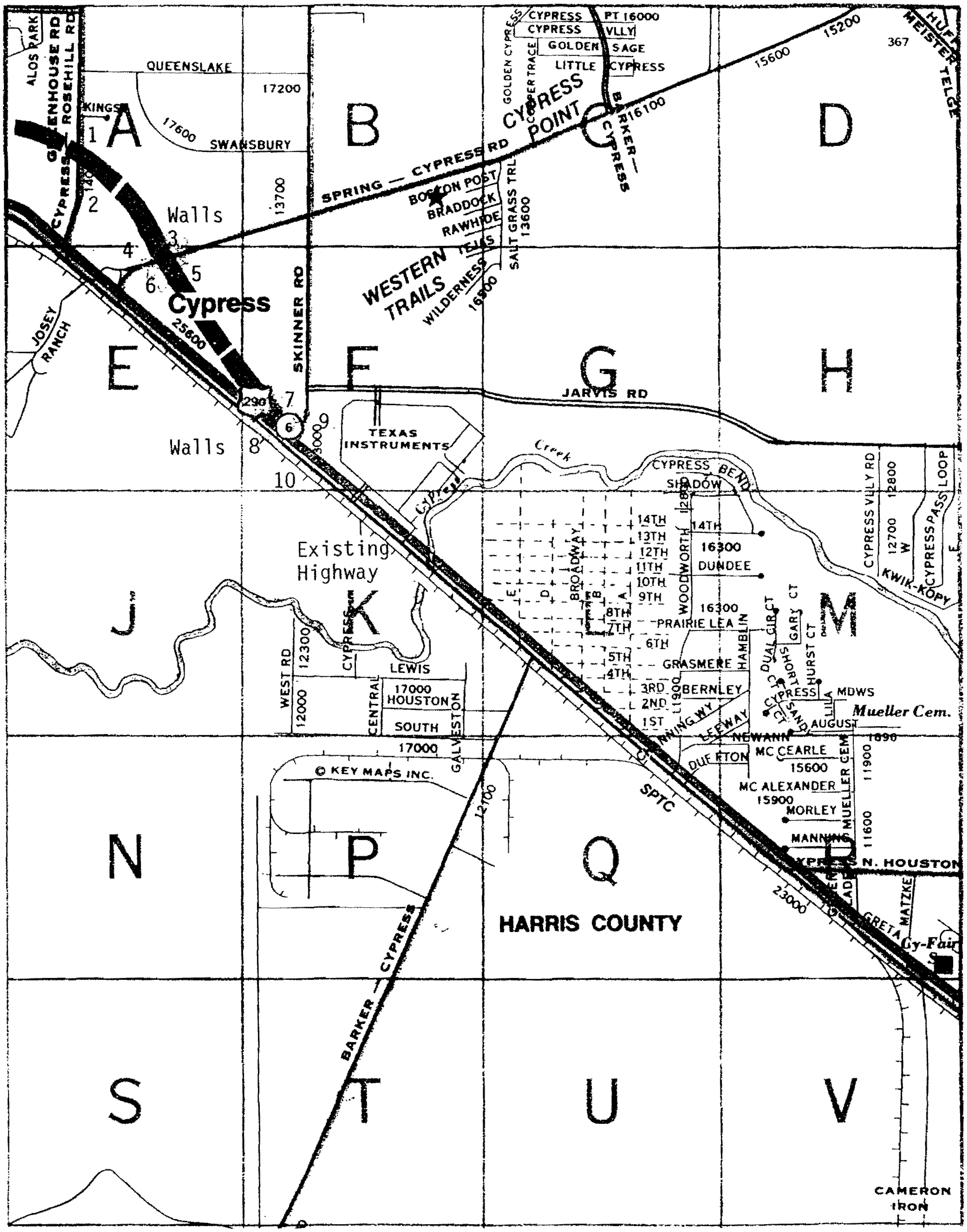
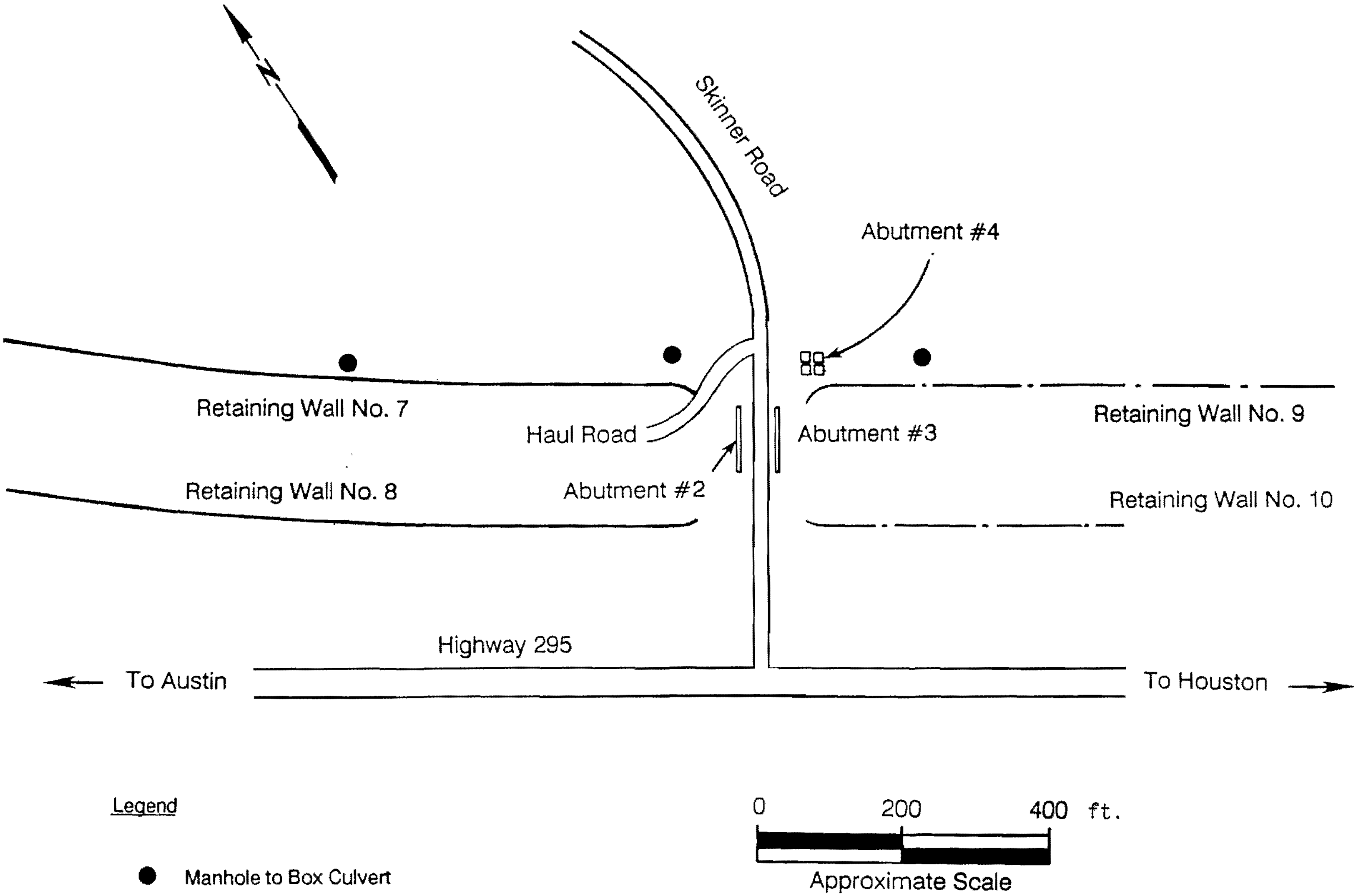
APPENDIX A

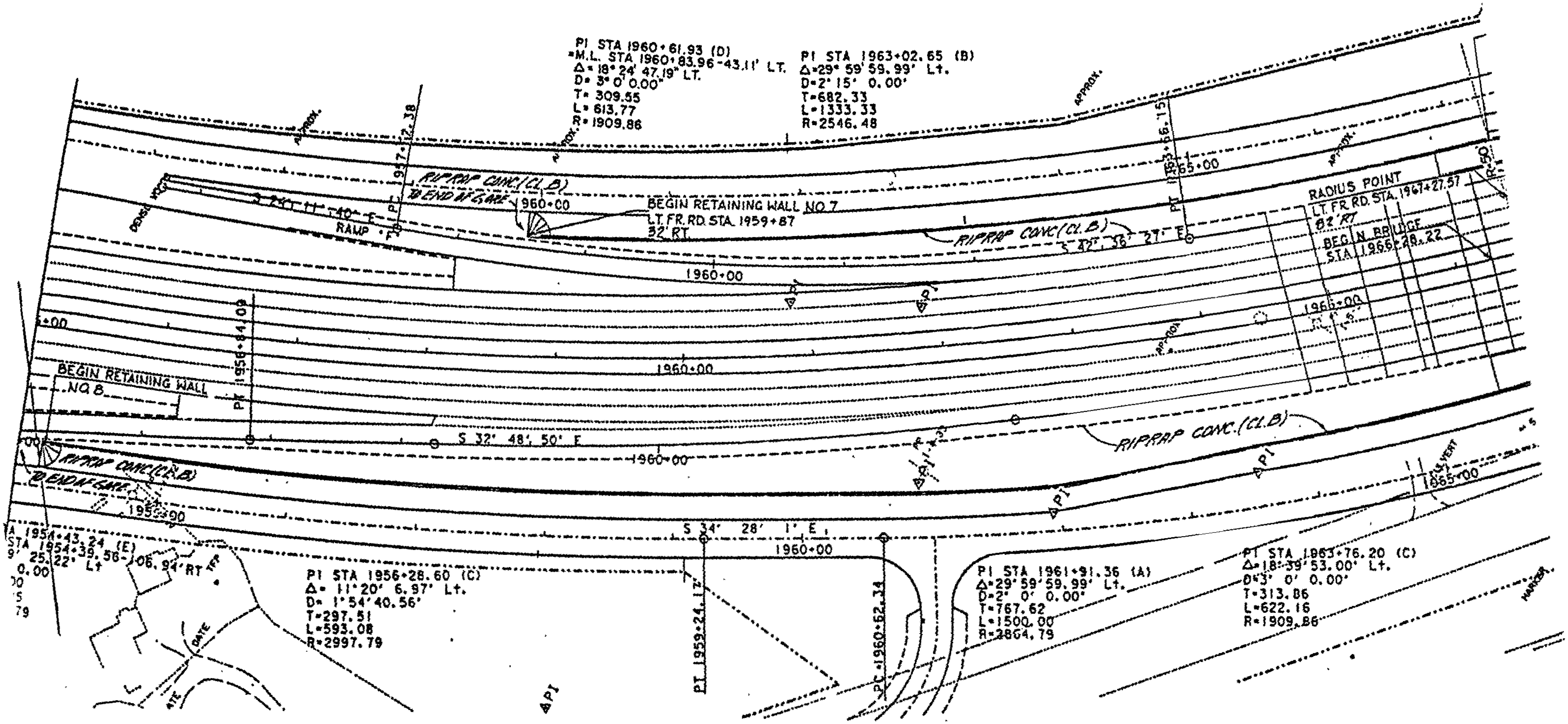
LAYOUT OF FIELD PROJECT

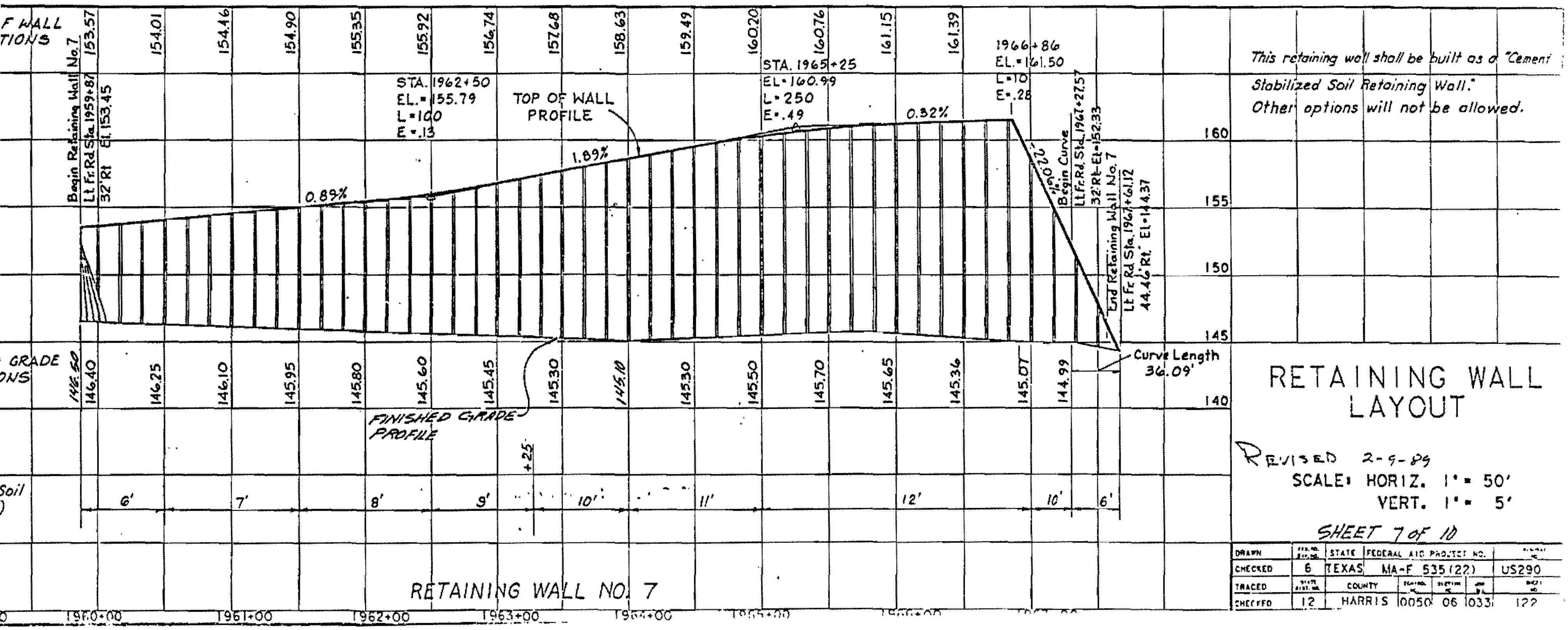


Location map of field construction

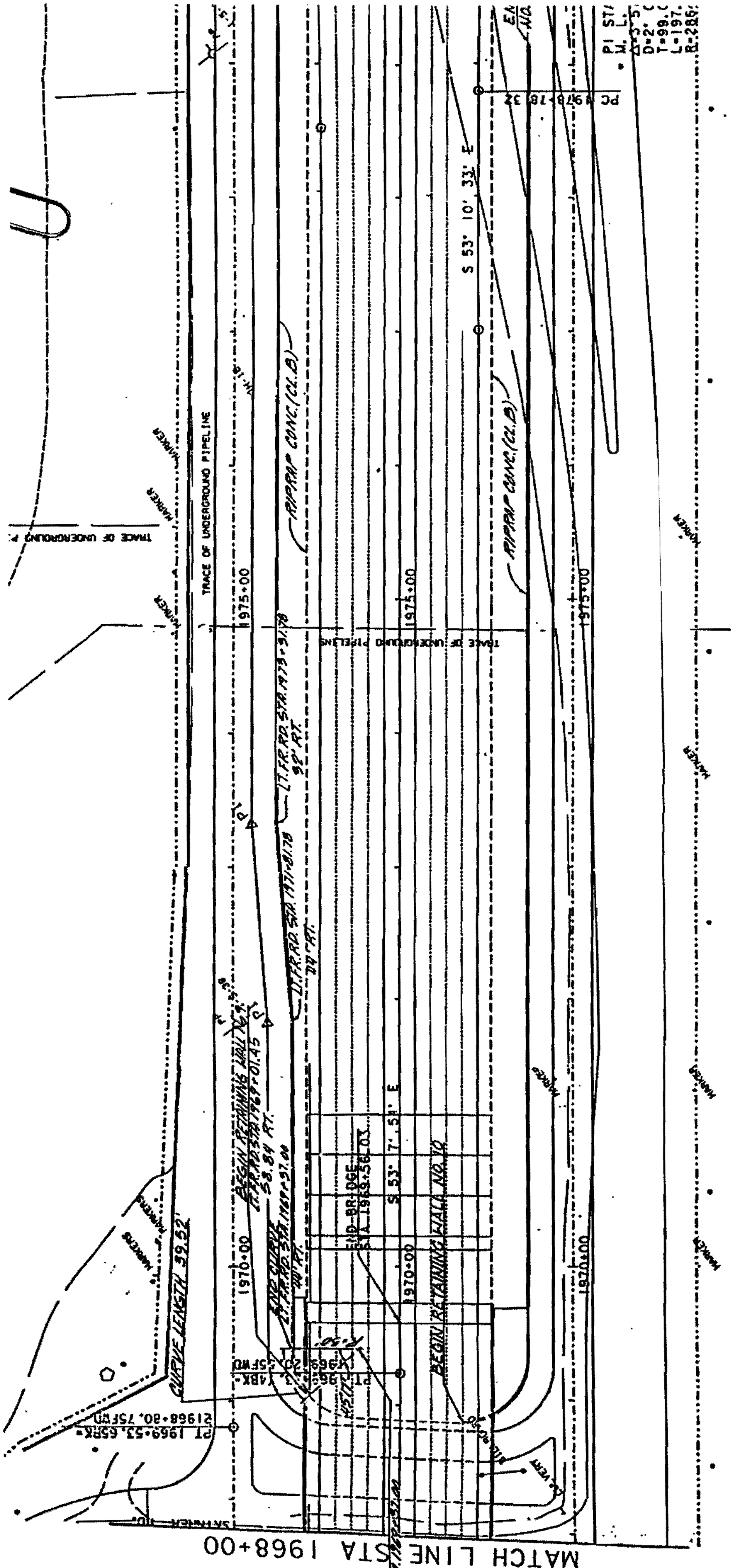

Skinner Road
Abutment #4
Retaining Wall No. 7
Haul Road
Abutment #2
Abutment #3
Retaining Wall No. 9
Retaining Wall No. 8
Retaining Wall No. 10
Highway 295
To Austin
To Houston
Legend
Manhole to Box Culvert
0
200
400 ft.
Approximate Scale

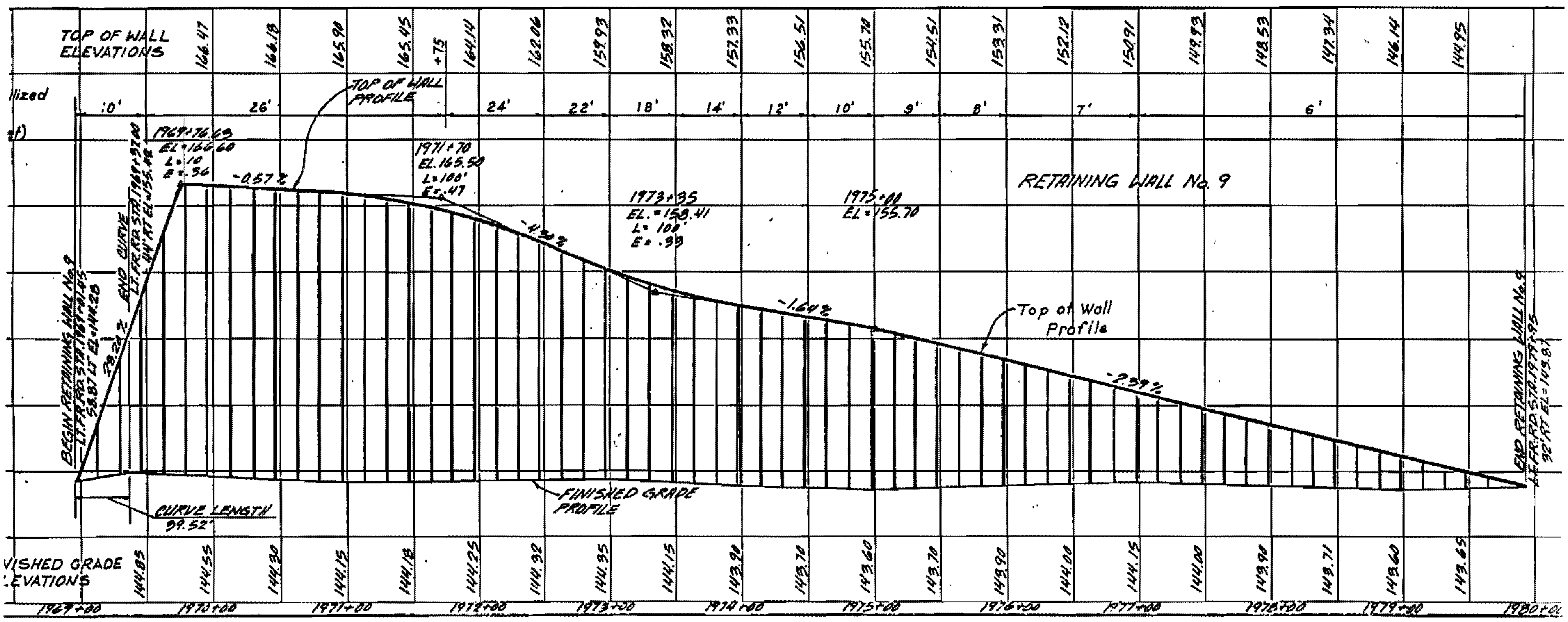
Site layout


PI STA 1960+61.93 (D)
=M.L. STA 1960+83.96-43.11' LT.
Δ= 18°24' 47.19" LT.
D= 3° 0' 0.00"
T= 309.55
L= 613.77
R= 1909.86
PI STA 1963+02.65 (B)
Δ=29° 59' 59.99' Lt.
D=2° 15' 0.00'
T=682.33
L=1333.33
R=2546.48
RIPRAP CONC.(CL.B)
TO END OF GABE
BEGIN RETAINING WALL NO.7
LT. FR. RD. STA. 1959+87
32' RT.
RAMP
1960+00
RIPRAP CONC.(CL.B)
RADIUS POINT
LT. FR. RD. STA. 1967+27.57
82' RT
BEGIN BRIDGE
STA. 1966+28.22
1965+00
1960+00
BEGIN RETAINING WALL
NO.8
PT 1956+84.09
S 32° 48' 50" E
1960+00
RIPRAP CONC.(CL.B)
RIPRAP CONC.(CL.B)
TO END OF GABE
S 34° 28' 1" E
1960+00
1965+00
PI STA 1956+28.60 (C)
Δ= 11°20' 6.97' Lt.
D= 1°54' 40.56'
T=297.51
L=593.08
R=2997.79
PT 1959+24.17
PC 1960+62.34
PI STA 1961+91.36 (A)
Δ=29° 59' 59.99' Lt.
D=2° 0' 0.00'
T=767.62
L=1500.00
R=2864.79
PI STA 1963+76.20 (C)
Δ=18° 39' 53.00' Lt.
D=3° 0' 0.00'
T=313.86
L=622.16
R=1909.86
APPROX.
GATE

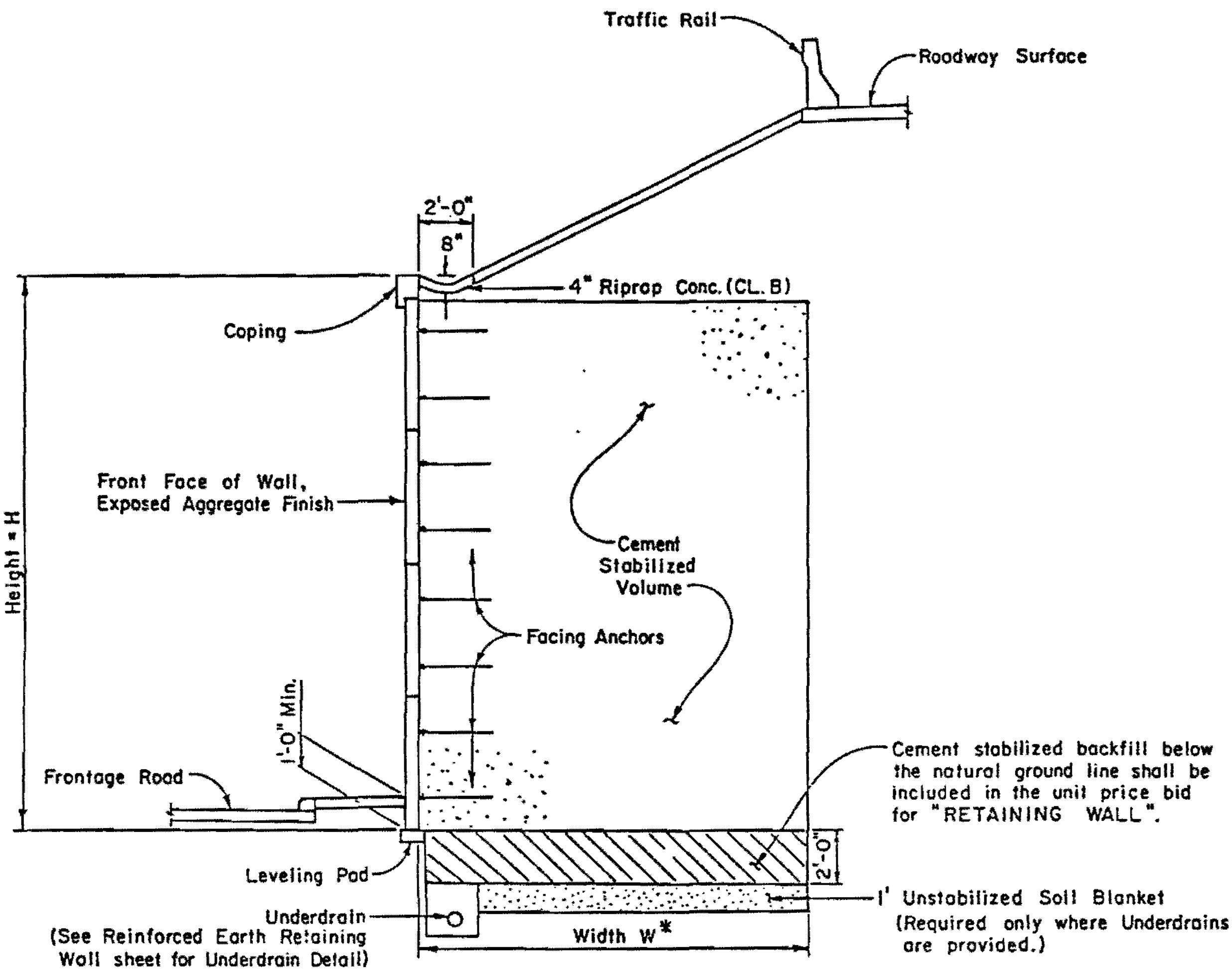


F WALL
TIONS
Begin Retaining Wall No. 7
Lt. Fr. Rd. Sta. 1959+87
32' Rt
El. 153.45
153.57
154.01
154.46
154.90
155.35
155.92
156.74
157.68
158.63
159.49
160.20
160.76
161.15
161.39
STA. 1962+50
EL.= 155.79
L = 100
E = .13
TOP OF WALL
PROFILE
STA. 1965+25
EL = 160.99
L = 250
E = .49
1966+86
EL. = 161.50
L = 10
E = .28
0.89%
1.89%
0.32%
22.06%
Begin Curve
Lt. Fr. Rd. Sta. 1967+27.57
32' Rt - El = 152.33
End Retaining Wall No. 7
Lt. Fr. Rd. Sta. 1967+61.12
44.46' Rt. El = 144.37
160
155
150
145
140
Curve Length
36.09'
GRADE
ONS
146.50
146.40
146.25
146.10
145.95
145.80
145.60
145.45
145.30
145.10
145.30
145.50
145.70
145.65
145.36
145.07
144.99
FINISHED GRADE
PROFILE
+25
Soil
6'
7'
8'
9'
10'
11'
12'
10'
6'
RETAINING WALL NO. 7
1960+00
1961+00
1962+00
1963+00
1964+00
1965+00
1966+00
This retaining wall shall be built as a "Cement Stabilized Soil Retaining Wall." Other options will not be allowed.
RETAINING WALL
LAYOUT
REVISED 2-9-89
SCALE: HORIZ. 1" = 50'
VERT. 1" = 5'
SHEET 7 of 10
DRAWN
CHECKED
TRACED
CHECKED
STATE
FEDERAL AID PROJECT NO.
6
TEXAS
MA-F 535 (22)
US290
COUNTY
12
HARRIS
0050
06
033
122


MATCH LINE STA 1968+00
TRACE OF UNDERGROUND PIPELINE
1975+00
1970+00
RIPRAP CONC.(CL.B)
S 53° 10' 33" E
S 53° 7' 53" E
END BRIDGE STA 1969+56.03
BEGIN RETAINING WALL NO. 10
CURVE LENGTH 39.52'
MARKER

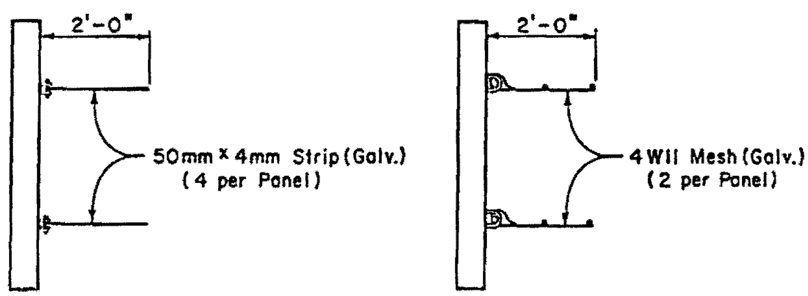


TOP OF WALL ELEVATIONS
166.47
166.18
165.90
165.45
+75
164.14
162.06
159.93
158.32
157.33
156.51
155.70
154.51
153.31
152.12
150.91
149.93
148.53
147.34
146.14
144.95
lized
10'
26'
24'
22'
18'
14'
12'
10'
9'
8'
7'
6'
TOP OF WALL PROFILE
1969+76.63
EL=166.60
L=10
E=.36
-0.57%
1971+70
EL.165.50
L=100'
E=.47
-4.30%
1973+35
EL.=158.41
L=100'
E=.33
1975+00
EL=155.70
-1.64%
RETAINING WALL No. 9
Top of Wall Profile
-2.39%
BEGIN RETAINING WALL No. 9
LT.FR.RD.STA.1969+01.95
58.87' LT EL=144.28
28.22%
END CURVE
LT.FR.RD.STA.1969+37.00
44' RT EL=155.48
END RETAINING WALL No. 9
LT.FR.RD.STA.1979+95
32' RT EL=143.87
CURVE LENGTH
39.52'
FINISHED GRADE PROFILE
NISHED GRADE ELEVATIONS
144.85
144.55
144.30
144.15
144.18
144.25
144.32
144.35
144.15
143.90
143.70
143.60
143.70
143.90
144.00
144.15
144.00
143.90
143.71
143.60
143.65
1969+00
1970+00
1971+00
1972+00
1973+00
1974+00
1975+00
1976+00
1977+00
1978+00
1979+00
1980+00



*See Retaining Wall Layout for width of Cement Stabilized Soil Block.

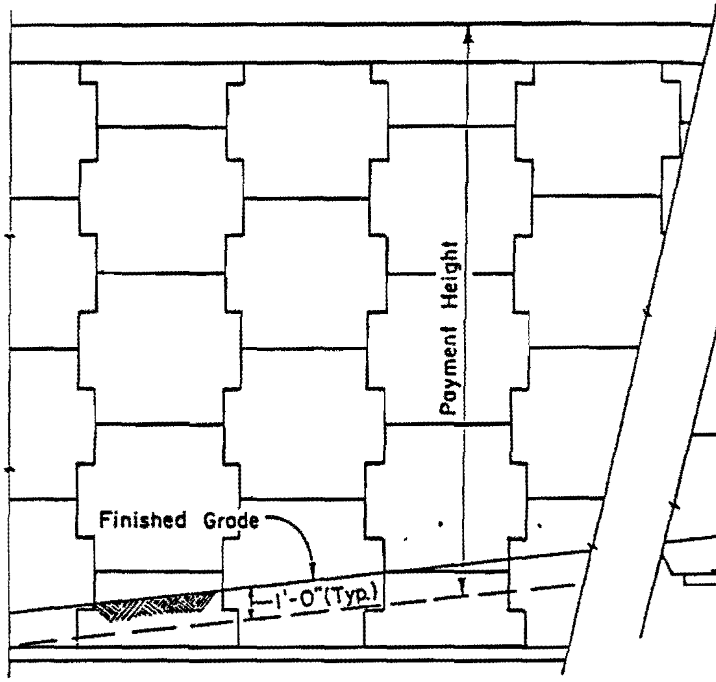
TYPICAL SECTION
(Wall at bottom of slope)



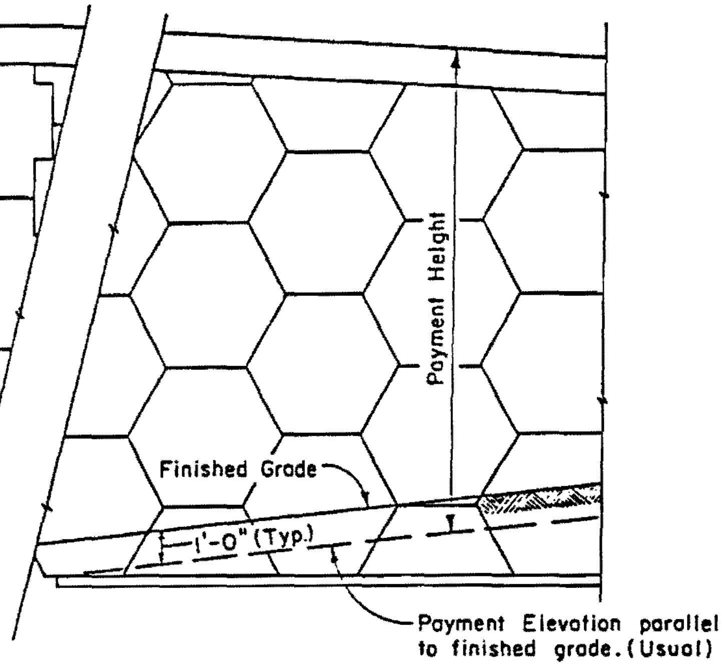
Reinforced Earth Panel

Retained Earth Panel

FACING ANCHOR DETAILS



Using Reinforced Earth Panels



Using Retained Earth Panels

ELEVATION

NOTES

Underdrains, when required, shall be provided at locations shown in the plans.

Cost of furnishing and installing Underdrains and Unstabilized Sand Blanket shall be included in the unit price bid for "Retaining Wall".

Payment Height shown in retaining wall layouts is considered the minimum height to be furnished. Additional wall furnished below payment line due to detailing or fabricator design requirements shall not be paid for directly but shall be considered incidental.

Exterior face of precast wall panels shall have an exposed aggregate finish unless otherwise noted.

Designed in accordance with Texas SDHPT Research Project 2-5-88-1178 "Design of Cement Stabilized Soil Retaining Walls with Concrete Panel Facing".

Retaining Walls No. 7 and 9 shall be constructed in accordance with these details. This wall type will not be an allowable option for the remaining walls on this project.

STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

CEMENT STABILIZED SOIL RETAINING WALL

(RETAINING WALLS NO. 7 & 9)

REVISED 2-9-89

ORIGINAL DRAWING DATE: FEBRUARY 1989	STATE DISTRICT	FEDERAL REGION	FEDERAL AID PROJECT	SHEET
REVISIONS	12	6	MA-F-535(22)	[illegible]

APPENDIX B

MATERIAL TEST RESULTS

NOV 15 '89 14:12 DS BRIDGE 845-6156 P.2

FAX 409-845-9536 - 9286

CONTROL 50-6-33

COUNTY Harris HIGHWAY US 290

RES. ENGR. Simmons SAMPLER Terry Vogt

STATION N/A DEPTH N/A

LOCATION House Hall Road, Cypress, Tx.

PRODUCER Hallet Materials

SPEC ITEM 4854 USE CSS

TYPE OF MATERIALS Sand + 7% cmt.

DATE COMP. 11-9-89 DATE REPTD 11-9-89

DATE INFO PROVIDED TO R.E. 11-9-89

LAB. NO. 1289J-376

DATE SAMPLED 11-3-89

COMP. EFFORT 50 Blow 13.26 ft./cu.

PI = 4.0 Max. 6

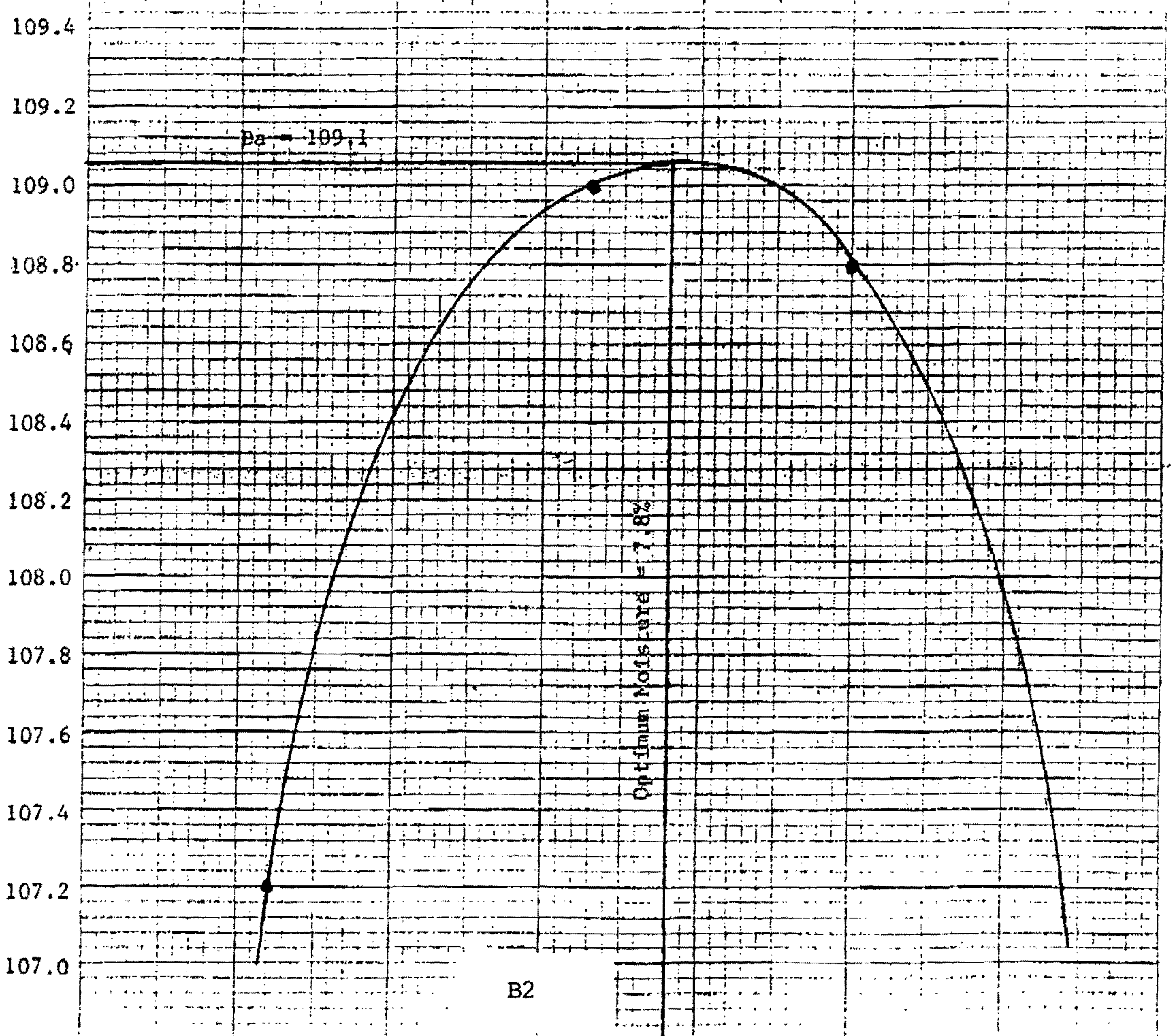
PH = 5.9 5.0 - 9.0

Resistivity = 8,004 ohm-cm Min. 1,5

Sieve	% Retained
4" =	0 (0)
3" =	0 (0 - 25)
200 =	98.5 (85 - 100)

Da = 109.1

Optimum Moisture = 7.8%



B2

LAW ENGINEERING

5500 Guhn Road Houston, Texas 77040 (713) 939-7161

SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

cement placed mostly by Oct 16th
(cut failure on Oct 31st)

LAB NO.	STA. NO.	DEPTH BELOW GRADE	AVERAGE COMP. STRENGTH	CEMENT CONTENT (SK/TON) — 4.7%	MOISTURE CONTENT	AGE (DAYS)
5705	1963+70	?	119	1.28 6%	6.6	?
5722	1967+00	6-8"	142	1.11 5.2	10.1	?
5723	1966+50	6-8"	128	1.36 6.4	9.4	?
5724	1966+00	4"	NA	1.15 5.4	10.4	?
5725	1966+00	14-16"	187	1.13 5.3	9.6	?
5726	1965+35	4"	NA	1.28 6.0	8.7	?
5727	1965+35	12"	127	1.51 7.1	11.3	?
5728	1965+35	16"	NA	1.22 5.7	10.6	?
5729	1965-25	12-14"	142	1.26 5.9	11.0	?
5730	1965+25	30-32"	NA	1.23 5.8	10.1	?
5731	1965+00	16-18"	137	1.17 5.5	9.5	?
5732	1964+50	6-8"	178	1.47 6.9	11.0	?
5733	1963+50	6-8"	165	1.06 5.0	9.8	?
5734	1963+00	6-8"	125	1.19 5.6	9.3	2
5735	1962+00	TOP	NA	1.45 6.8	6.6	1

200 psi required 6% required

ATOMIC ABSORPTION

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser

Steve Kaiser, CET
Materials Lab Supervisor
SK/LAB/cubecss.5705-35

Michael O'Grady
Construction Services

LAW ENGINEERING

5500 Guhn Road Houston, Texas 77040 (713) 939-7161

SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5705
DATE SAMPLED:	11/02/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1963+70
DEPTH BELOW GRADE:	?
SAMPLED BY:	MB / SDHPT
AVERAGE MOISTURE:	6.6 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.28

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.05X2.01	4.121	11/05/89	?	483	117
2	1.88X2.00	3.760	11/05/89	?	452	120
3	2.12X2.20	4.644	11/05/89	?	559	120

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 119 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser
Steve Kaiser, CET
Materials Lab Supervisor
SK/LAB/cubecss.5705

Michael O'Grady
Construction Services

LAW ENGINEERING

5500 Guhn Road Houston, Texas 77040 (713) 939-7161

SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5722
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1967+00
DEPTH BELOW GRADE:	6-8"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	10.1 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.11

COMPRESSIVE STRENGTH TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	1.92X1.88	3.610	11/05/89	?	532	147
2	2.05X2.00	4.100	11/05/89	?	580	141
3	1.98X2.05	4.059	11/05/89	?	559	138

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 142 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser
Steve Kaiser, CET
Materials Lab Supervisor
SK/LAB/cubecss.5722

Michael O'Grady
Construction Services

LAW ENGINEERING

5500 Guhn Road Houston, Texas 77040 (713) 939-7161

SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5723
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1966+50
DEPTH BELOW GRADE:	?
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	9.4 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.36

COMPRESSIVE STRENGTH TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.05X2.07	4.244	11/05/89	?	550	130
2	2.15X2.22	4.773	11/05/89	?	587	123
3	2.13X2.02	4.303	11/05/89	?	559	130

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 128 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser
Steve Kaiser, CET
Materials Lab Supervisor
SK/LAB/cubecss.5723

Michael O'Grady
Construction Services

LAW ENGINEERING

5500 Guhn Road Houston, Texas 77040 (713) 939-7161

SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5725
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1966+00
DEPTH BELOW GRADE:	14-16"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	9.6 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.13

COMPRESSIVE STRENGTH TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.10X2.17	4.557	11/05/89	?	894	196
2	2.06X2.10	4.326	11/05/89	?	801	185
3	1.85X1.69	3.127	11/05/89	?	561	180

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 187 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser

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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5727
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1935+35
DEPTH BELOW GRADE:	12"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	11.3 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.51

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.10X2.08	4.368	11/05/89	?	541	124
2	2.12X2.01	4.261	11/05/89	?	504	118
3	2.10X2.01	4.221	11/05/89	?	582	138

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 127 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser

Steve Kaiser, CET
Materials Lab Supervisor
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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5729
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1965+25
DEPTH BELOW GRADE:	14"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	11.0 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.26

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.30X2.12	4.876	11/05/89	?	719	148
2	2.28X2.08	4.742	11/05/89	?	619	130
3	2.02X2.04	4.121	11/05/89	?	606	147

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 142 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Ste Kaiser

Steve Kaiser, CET
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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5731
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1965+00
DEPTH BELOW GRADE:	16-18"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	9.5 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.17

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.20X2.08	4.576	11/05/89	?	597	130
2	2.15X2.28	4.902	11/05/89	?	745	152
3	2.10X2.15	4.515	11/05/89	?	580	128

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 137 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

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Materials Lab Supervisor
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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5732
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1964+50
DEPTH BELOW GRADE:	6-8"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	11.0 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.47

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.15X1.90	4.085	11/05/89	?	745	183
2	2.42X2.35	5.687	11/05/89	?	976	172
3	2.00X2.05	4.305	11/05/89	?	768	178

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 178 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

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Steve Kaiser, CET
Materials Lab Supervisor
SK/LAB/cubecss.5732

Michael O'Grady
Construction Services

LAW ENGINEERING

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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5733
DATE SAMPLED:	11/03/89
DATE PLACED:	?
SAMLPE LOCATION:	STA. 1963+50
DEPTH BELOW GRADE:	6-8"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	9.8 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.06

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.00X2.05	4.100	11/05/89	?	671	164
2	2.15X2.17	4.665	11/05/89	?	712	153
3	2.10X2.12	4.452	11/05/89	?	792	178

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 165 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

Steve Kaiser

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Michael O'Grady
Construction Services

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SAND -CEMENT TEST REPORT

CLIENT: SDHPT (Simmons Res.) DATE: 11/07/89

PROJECT: 0050-06-033 JOB NO.: HT-2543
HWY 290 @ SKINNER ROAD TASK 14

LAB NO.:	5734
DATE SAMPLED:	11/03/89
DATE PLACED:	11/01/89
SAMLPE LOCATION:	STA. 1963+00
DEPTH BELOW GRADE:	6-8"
SAMPLED BY:	SK / LAW ENGINEERING
AVERAGE MOISTURE:	9.3 %
SUPPLIER:	HALLETT MATERIALS (JOSEY RANCH)
CEMENT CONTENT (Sack/Ton):	1.19

COMPRESSIVE STRENGTH
TEST RESULTS

SPECIMEN NUMBER	DIMENSION (INCH)	AREA (SQ.IN)	DATE TESTED	TEST AGE (DAYS)	MAX.LOAD (lbs)	COMPRESSIVE STR.(PSI)
1	2.11X2.38	5.022	11/05/89	2	597	119
2	2.20X2.52	5.544	11/05/89	2	699	126
3	? 10X2.10	4.410	11/05/89	2	567	129

SPECIFIED COMPRESSIVE STRENGTH: 100 PSI @ 48 HRS
AVERAGE OF (3) SPECIMENS = 125 PSI

UNLESS OTHERWISE INDICATED ALL TESTS WERE PERFORMED IN GENERAL ACCORDANCE WITH ASTM D-1633.

* CUBES CUT FROM SAMPLES TAKEN IN FIELD

Respectfully submitted,

LAW ENGINEERING, INC. Reviewed by:

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SK/LAB/cubecss.5734

Michael O'Grady
Construction Services

APPENDIX C

GEOLOGY OF THE SITE

1 INTRODUCTION

This discussion summarizes interpretations made of the study site geology and physical characteristics of the foundation materials in the vicinity of retention walls 7, 8, 9, and 10. These interpretations were formulated from analysis of data collected by the Texas State Department of Highways and Public Transportation during the site investigation phase of the project and from "mini-cone" penetration data collected for Texas A&M University, by McClelland Consultants Inc. in March 1990.

2 GENERAL GEOLOGIC SETTING

The study site is located adjacent to the town of Cypress, in Northwestern Harris County, Tx. The physiography of the region, typical of northern reaches of the Texas gulf coastal province, is comprised of forested uplands and upland savannas (Figure 1). The landforms and sedimentary materials in the region result from fluvial-deltaic activity which dominated landforming processes in this portion of the Gulf Coast.

The sedimentary units at the study site were laid down during the Pleistocene Epoch of the early Quaternary Period (approx. 2 my. b.p.). Two natural Pleistocene sedimentary systems - an alluvial fan system and a fluvial-deltaic system dominated the depositional history of the study region. The alluvial fan system occurs as wedge-shaped belt of tree covered sands and gravels (Unit III Figure 2). Interspersed within the coarse grained alluvial fan deposits are isolated prairie areas composed of overbank mud deposits.

The Pleistocene fluvial-deltaic system is further subdivided into two basic units - fluvial sands and delta-plain muds. The fluvial sand deposits occur as a continuous blanket of meanderbelt sand deposits across the northwestern parts of Harris Co. and include the study site. Pleistocene delta-plain deposits of the fluvial-deltaic system occur in the southern parts of the area as a flat mud and clay substrate covered with prairie grasses. These fine-grained clayey sediments bound isolated narrow elongate distributary sands trending in a north-south direction (Figure 3).

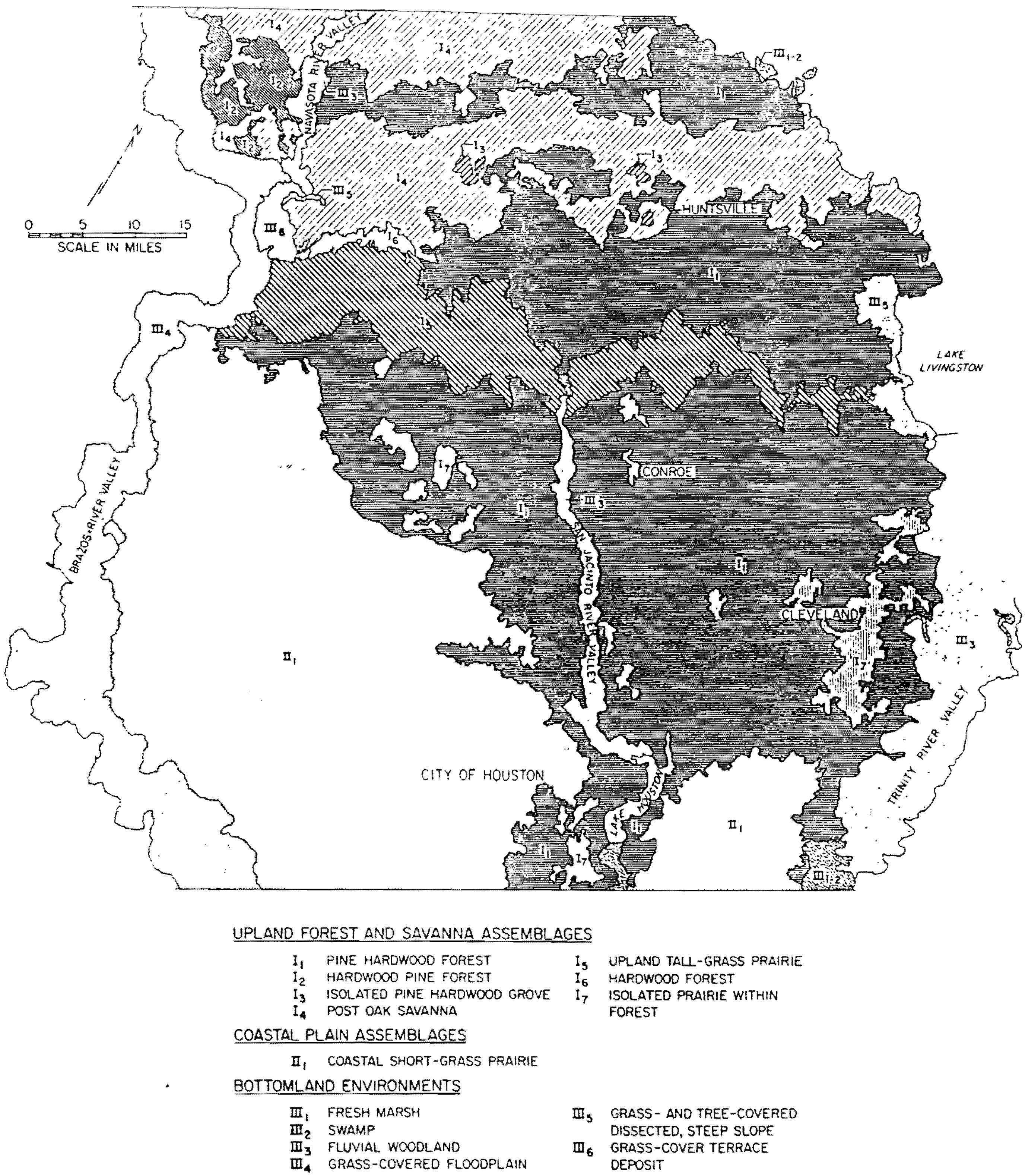


Figure 1. Biologic Assemblages defined in the Greater Houston area.

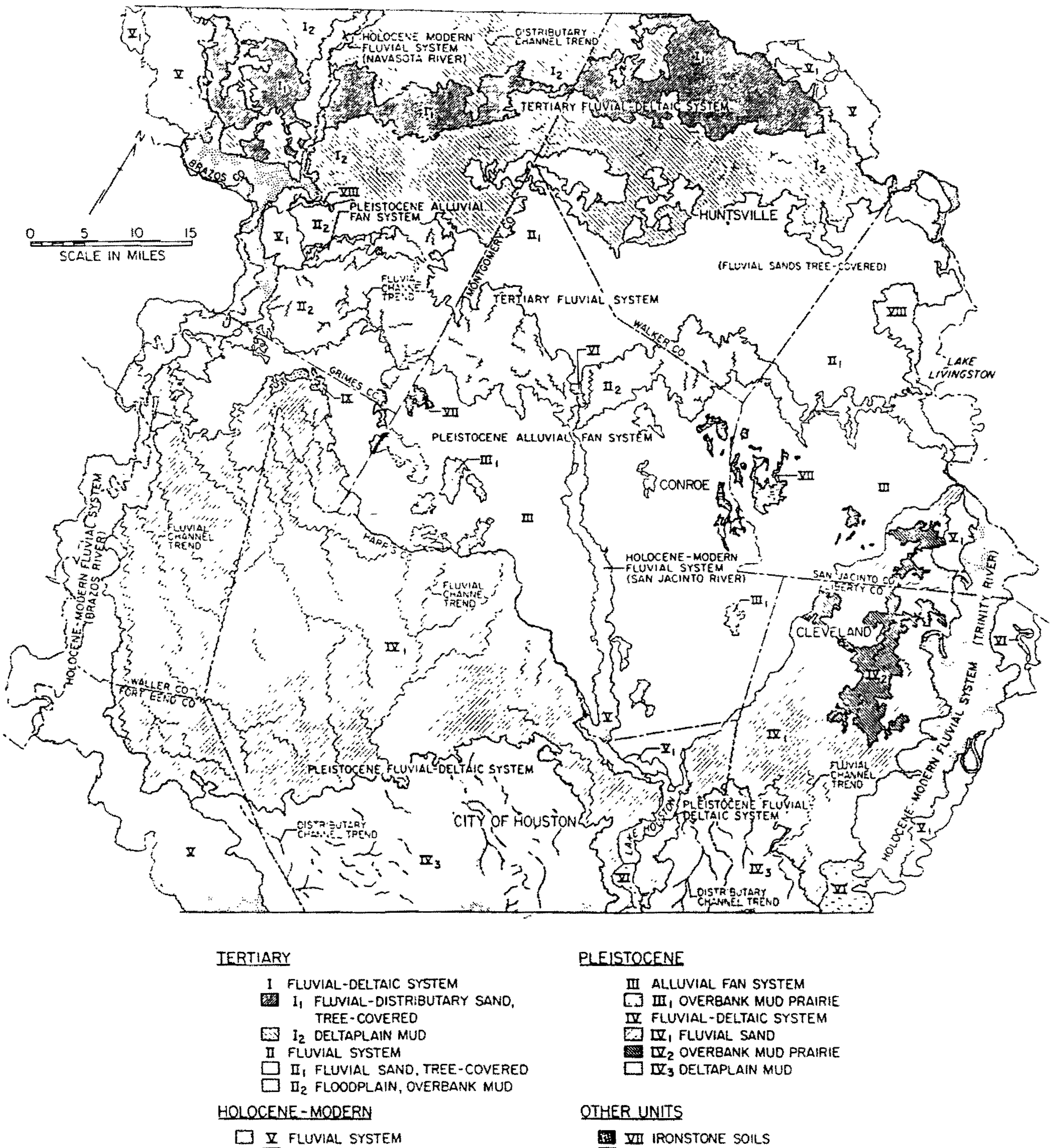


Figure 2. Natural systems defined by environmental geologic mapping in the Greater Houston area.

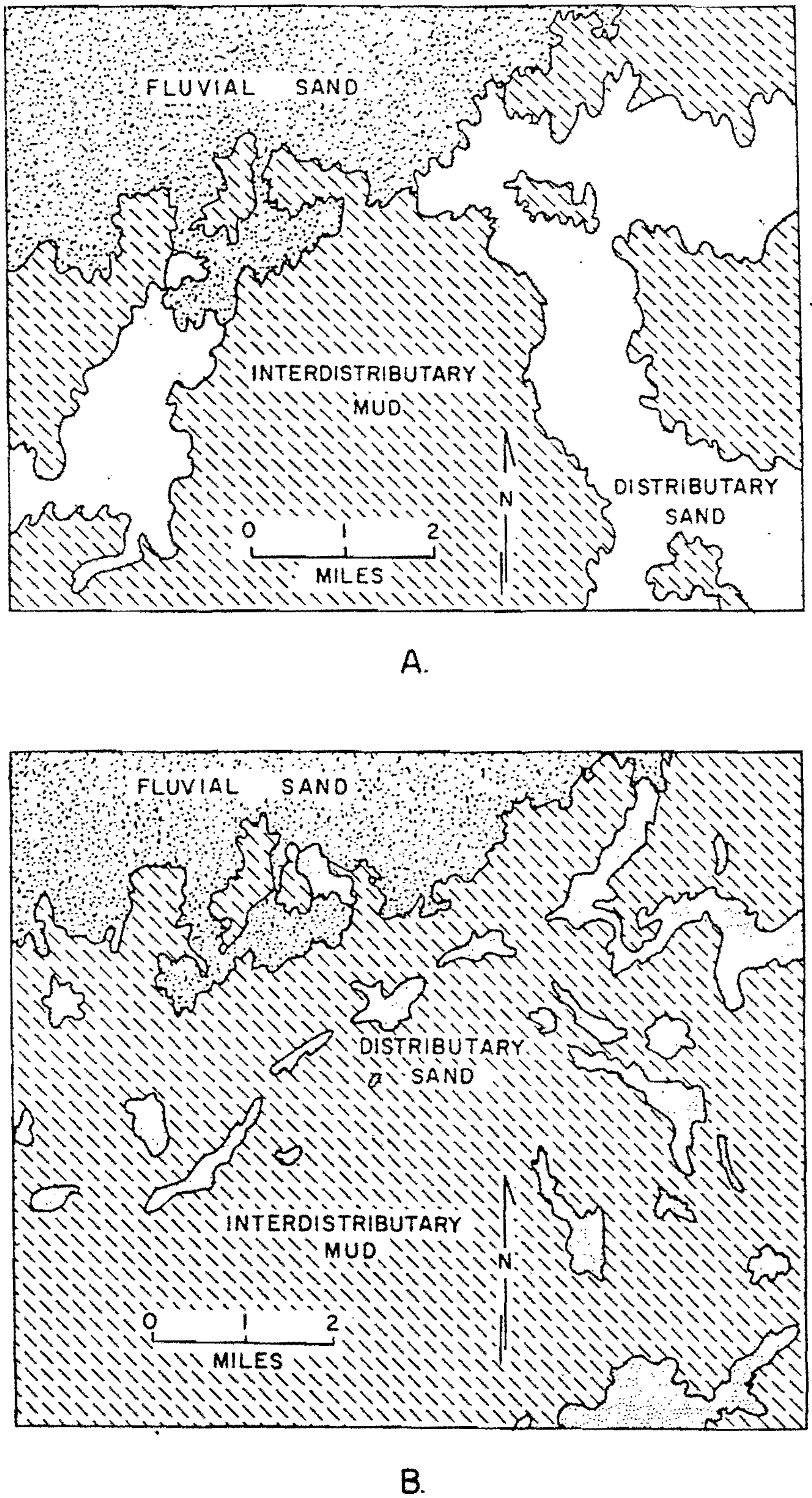


Figure 3. Sand trends in a high-mud deltaic system in the Greater Houston area.

Figure 4 illustrates the process by which meandering distributary channels grade down-dip, coastward into elongate belts of sand and silt. This material was deposited principally during periods of delta building (i.e. progradation). As these channels shift laterally with time, laterally continuous belts of sand and silt are formed. It was by these processes that the sediments underlying the project were deposited.

3 SITE SPECIFIC CHARACTERISTICS

3.1 Site Surface Soils

Based on a United States Department of Agriculture, Soil Conservation Service survey of the area in 1974, the soils at the site may be broadly classified by two principal assemblages. These are the Clodine-Addicks-Gessner Association and the Katy-Aris Association. Both associations are essentially loamy prairie soils. The soils are poorly drained and are moderately to very slowly permeable units. These characteristics require special consideration so as avoid foundation problems such as settlement and bearing failure (Plate 1).

3.2 Site Geology

3.2.1 Literature Review

A review of the literature concerning the geologic history of the site indicates that the project is located upon the Pleistocene age, Montgomery Formation (Plate 1). Previously this unit has been mapped and referred to as the Upper Lissie Formation. A general listing of the Montgomery Formation's characteristics is as follows:

> Clay, silt, and very minor siliceous gravel of granule and small pebble size, gravel more abundant northwestward, locally calcareous, concretions of calcium carbonate, iron oxide, and iron manganese oxides common in zone of weathering; Fluviatile; Surface fairly flat and featureless except for numerous rounded shallow depressions and pimple mounds; + / - 100 ft thick.

3.2.2 Soil Boring Interpretation

The Texas State Department of Highways and Public Transportation (TSDHPT) conducted the bridge foundation and soil test boring program along the

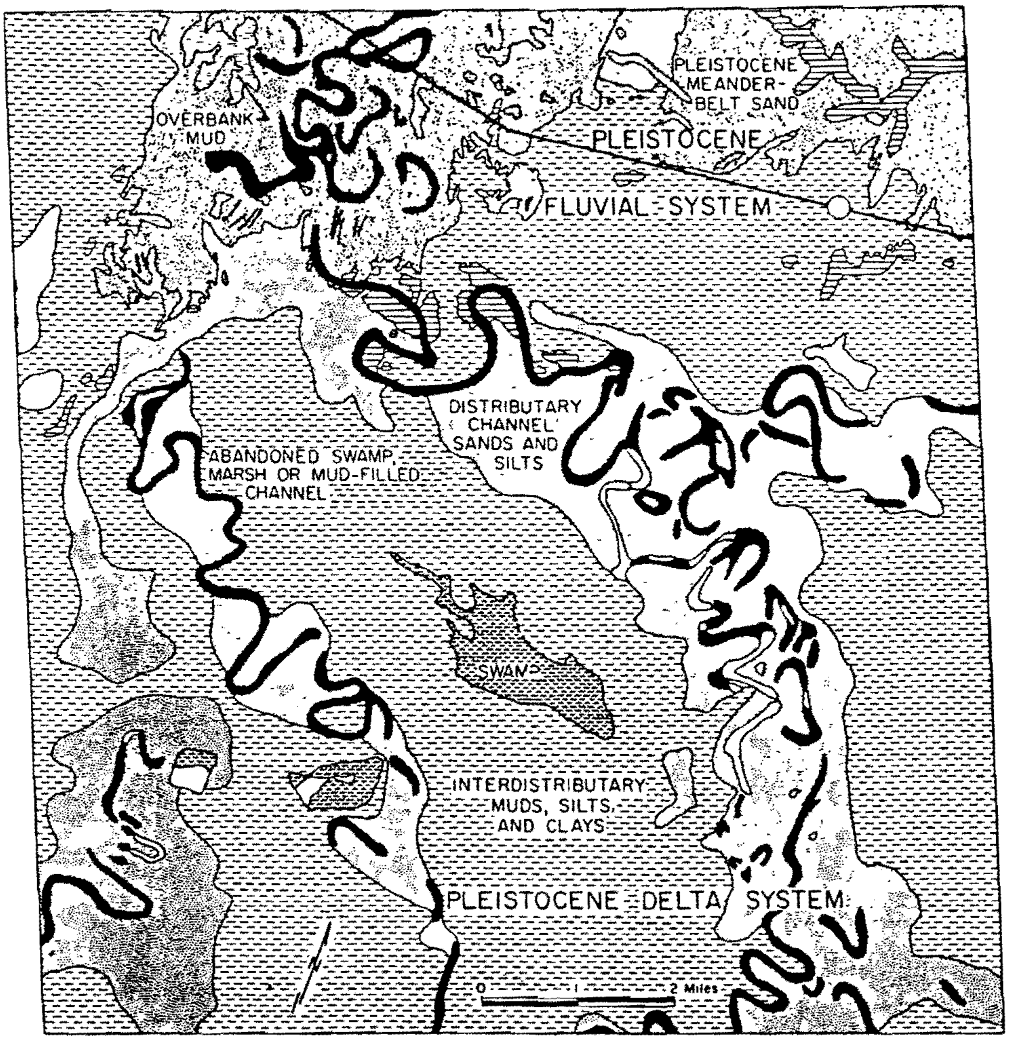


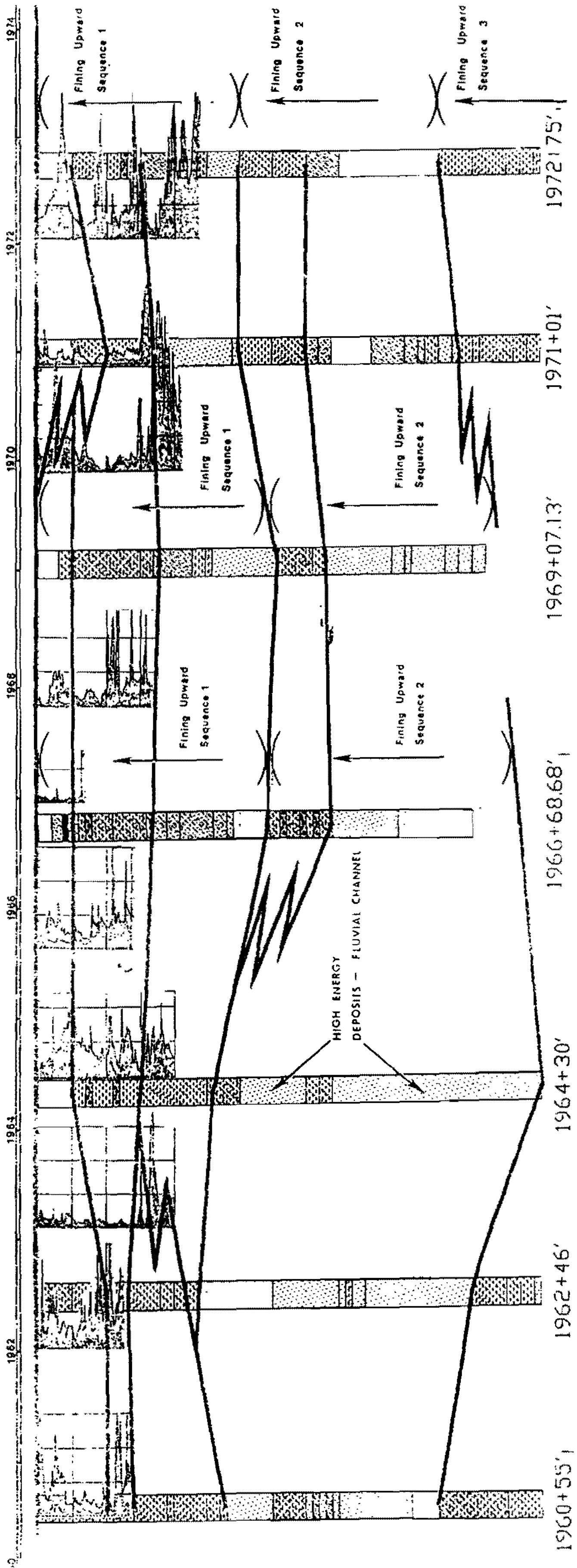
Figure 4. Pleistocene fluvial-deltaic facies, coastal uplands in vicinity of Devers, Beaumont-Port Arthur area. Meanderbelt sands appear to grade coastward into elongate belts of sand and silt that were deposited principally during delta building. After Fisher and others, 1973.

approximate centerline of the project. This data was made available to Texas A&M University and has been reproduced in Appendix A. Portions of this data were used herein to interpret the soil stratigraphy underlying the project site. Specifically seven borings spaced from station 1960+55' to 1972+76' were employed to generate a subsurface cross-section between walls 7, 8, 9, and 10 (Plate 1).

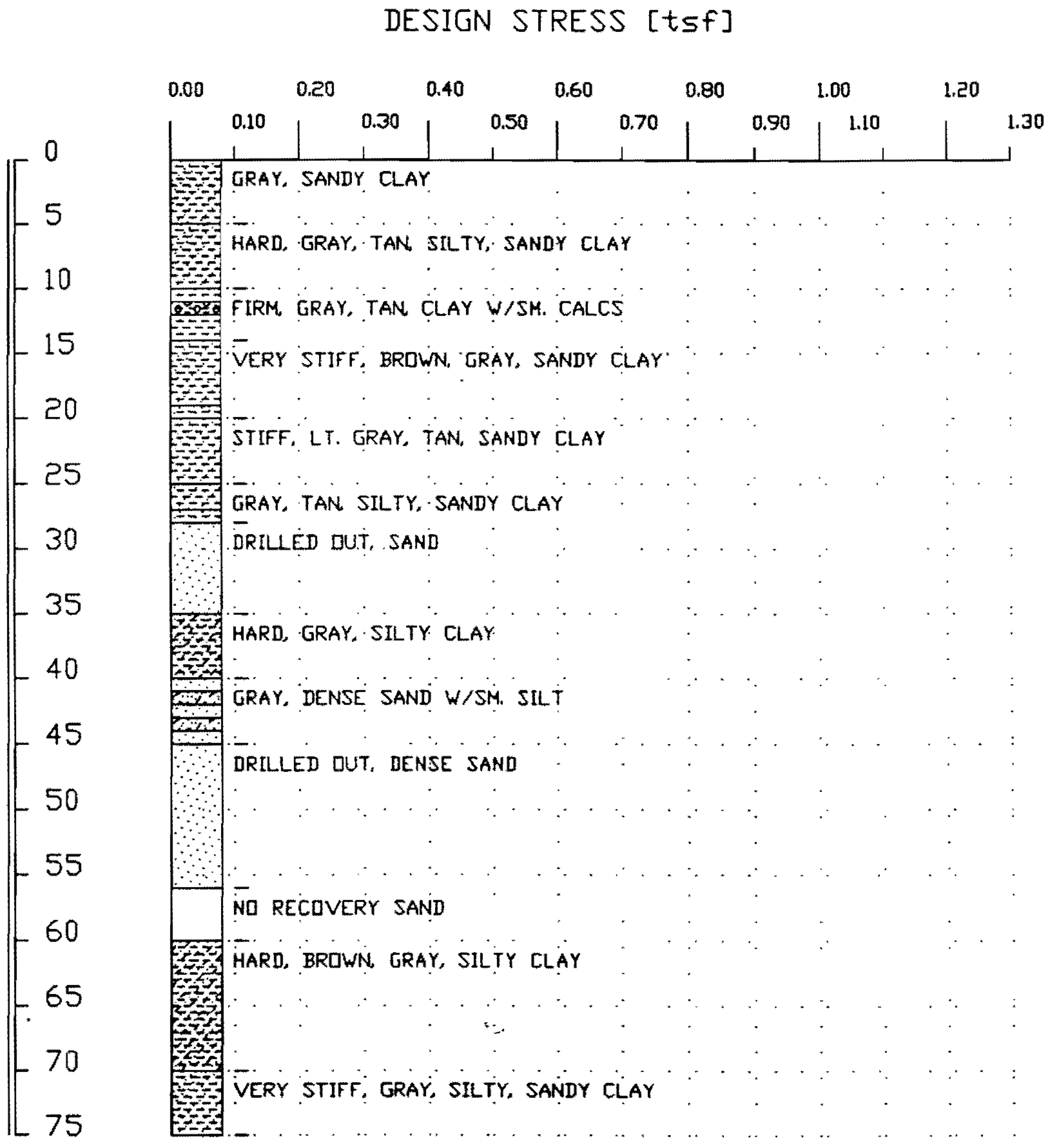
As can be seen from the cross section, the geometry of the stratigraphy is typical of the Texas gulf coast in general and fluvial-deltaic complexes in particular. Abundant variation in sedimentary materials, lateral facies changes over relatively short distances and gradation trends are all indicative of the fluvial-deltaic nature of these deposits. The environment of deposition of the large sand bodies is one of relatively high energy. It is likely that these units were formed from ancient distributary channels during deltaic progradation. On the cross section are pulses or sequences of deposition indicated as "fining upward." These are another indicator of a fluvial environment of deposition for the soils at the site. The coarser sediments within these sequences are generally higher energy deposits. As the energy of the system decreases due either to channel abandonment or decreased flow following flood events the finer grained sediments are deposited.

General trends across the site are:

1. Except for the N.W. end of walls 7 and 8, initial sands are found at a depth of approx. 15 to 20 ft.
2. The units tend to become coarser overall with depth up to the data limit of 75 ft.
3. The shear strength of the soil although quite variable tends to increase with depth.
4. Maximum values of shear strength occur at a depth of approximately 50 ft.
5. The soils at a depth of 5 to 15 ft. tend to be stiff.
6. Toward the S.W. (walls 9 and 10) units tend to become more complex leading to greater variations in physical properties.



Geologic cross-section through site



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GRAY, SANDY CLAY
HARD, GRAY, TAN, SILTY, SANDY CLAY
FIRM, GRAY, TAN, CLAY W/SM. CALCS
VERY STIFF, BROWN, GRAY, SANDY CLAY
STIFF, LT. GRAY, TAN, SANDY CLAY
GRAY, TAN, SILTY, SANDY CLAY
DRILLED OUT, SAND
HARD, GRAY, SILTY CLAY
GRAY, DENSE SAND W/SM. SILT
DRILLED OUT, DENSE SAND
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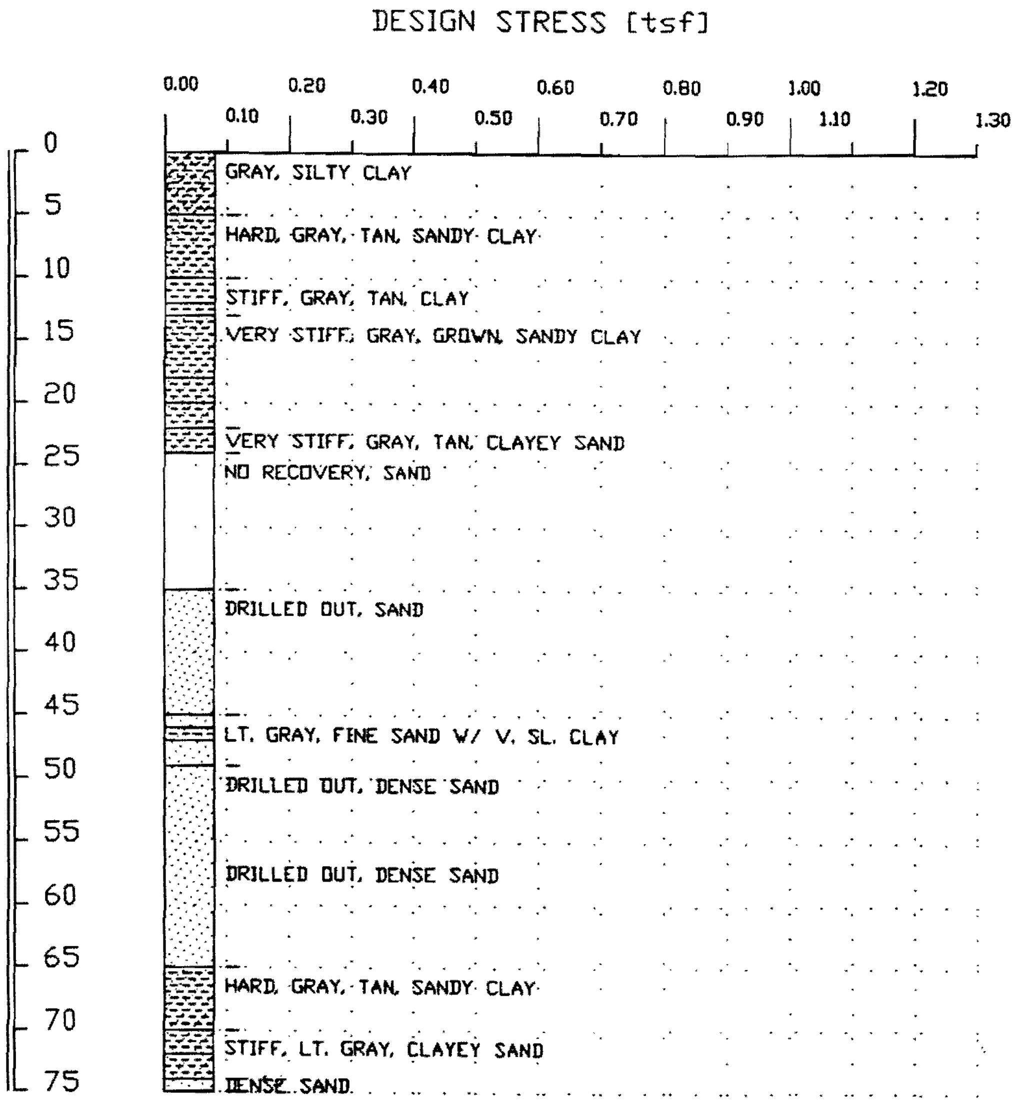
HARD, BROWN, GRAY, SILTY CLAY
VERY STIFF, GRAY, SILTY, SANDY CLAY

1960+55′



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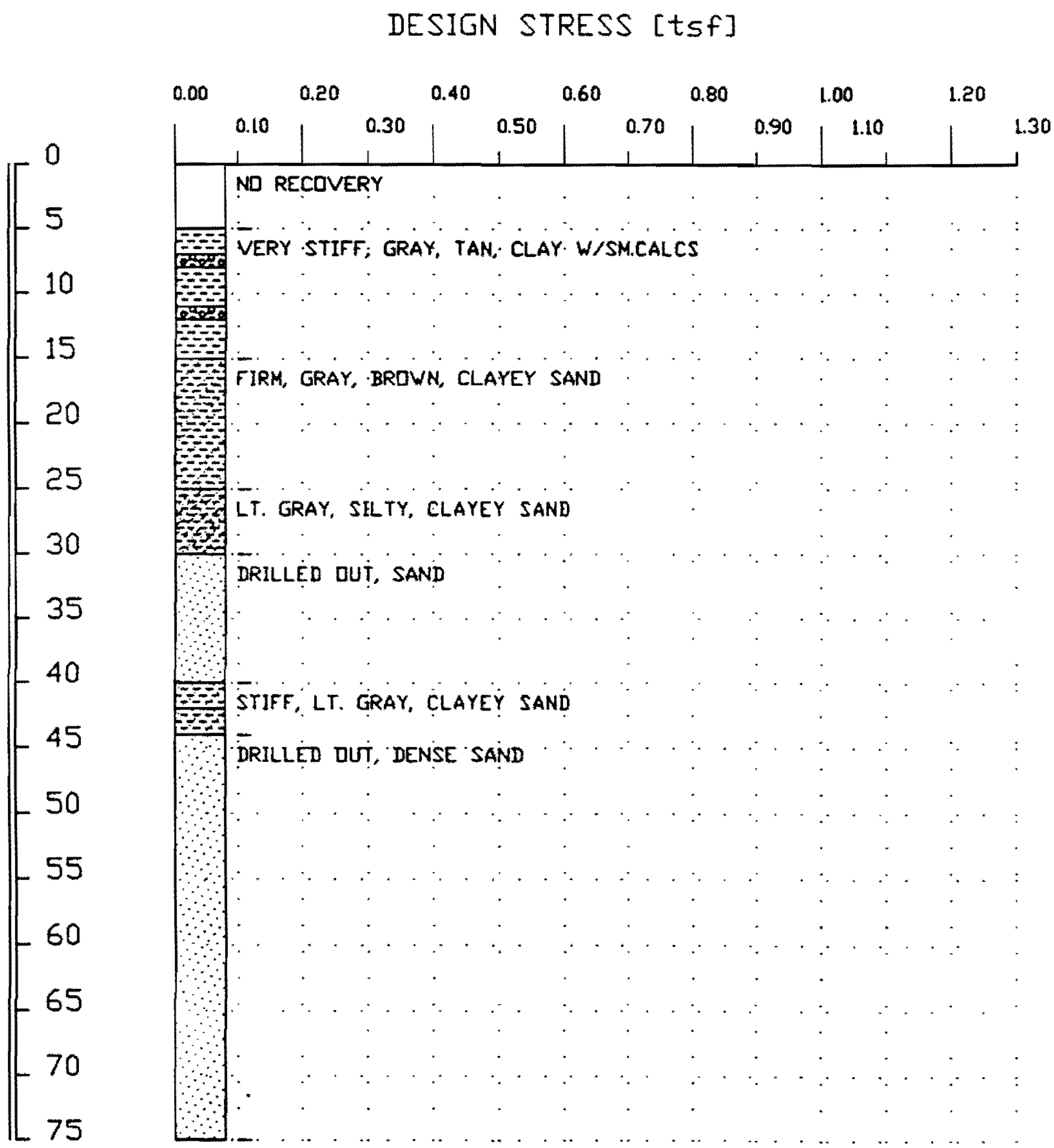
HARD, GRAY, TAN, SANDY CLAY
STIFF, GRAY, TAN, CLAY
VERY STIFF, GRAY, GROWN, SANDY CLAY
VERY STIFF, GRAY, TAN, CLAYEY SAND
NO RECOVERY, SAND
DRILLED OUT, SAND
LT. GRAY, FINE SAND W/ V. SL. CLAY
DRILLED OUT, DENSE SAND
DRILLED OUT, DENSE SAND
HARD, GRAY, TAN, SANDY CLAY
STIFF, LT. GRAY, CLAYEY SAND
DENSE SAND

1962+46'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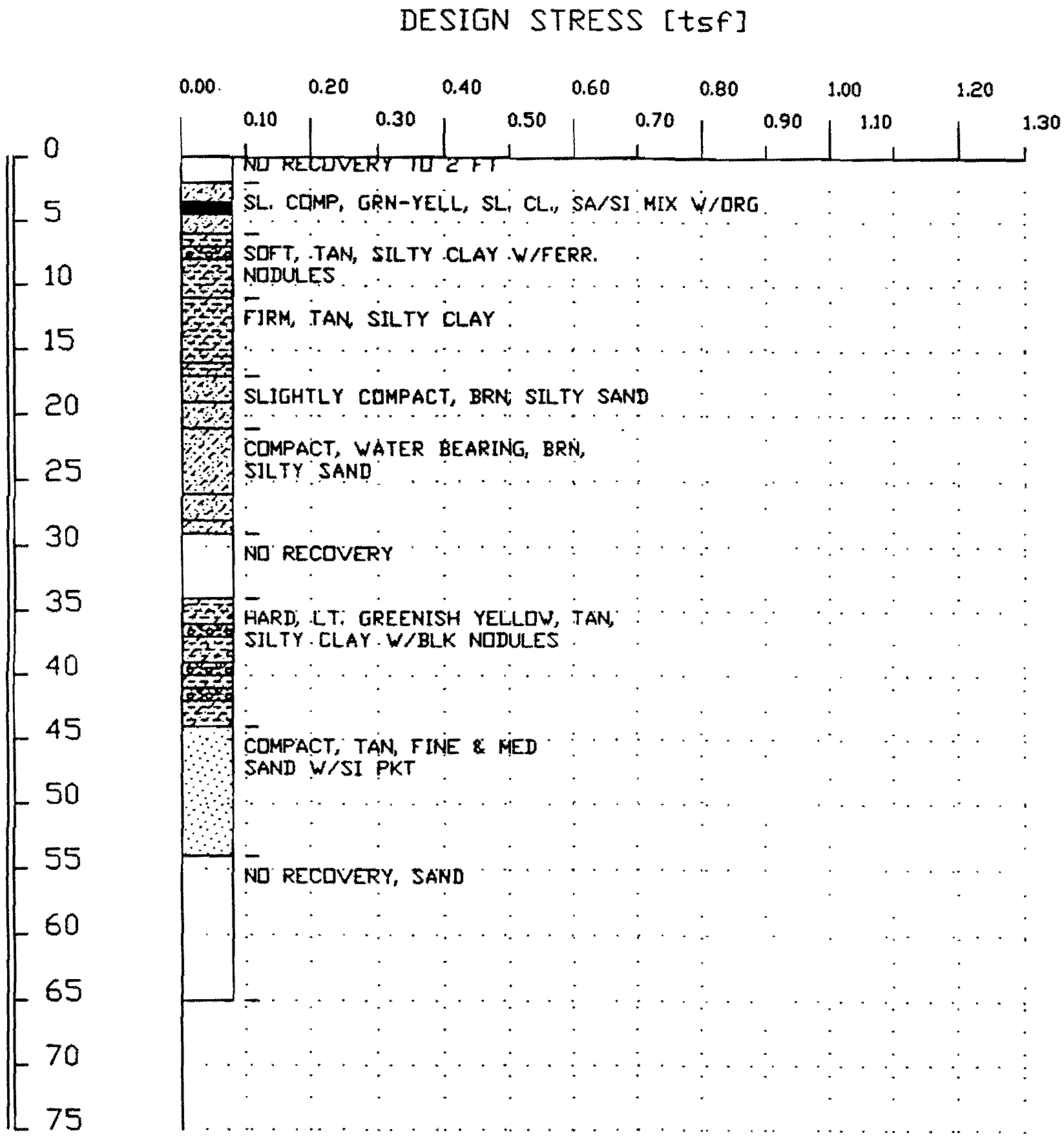
75
NO RECOVERY
VERY STIFF, GRAY, TAN, CLAY W/SM.CALCS
FIRM, GRAY, BROWN, CLAYEY SAND
LT. GRAY, SILTY, CLAYEY SAND
DRILLED OUT, SAND
STIFF, LT. GRAY, CLAYEY SAND
DRILLED OUT, DENSE SAND

1964+30′



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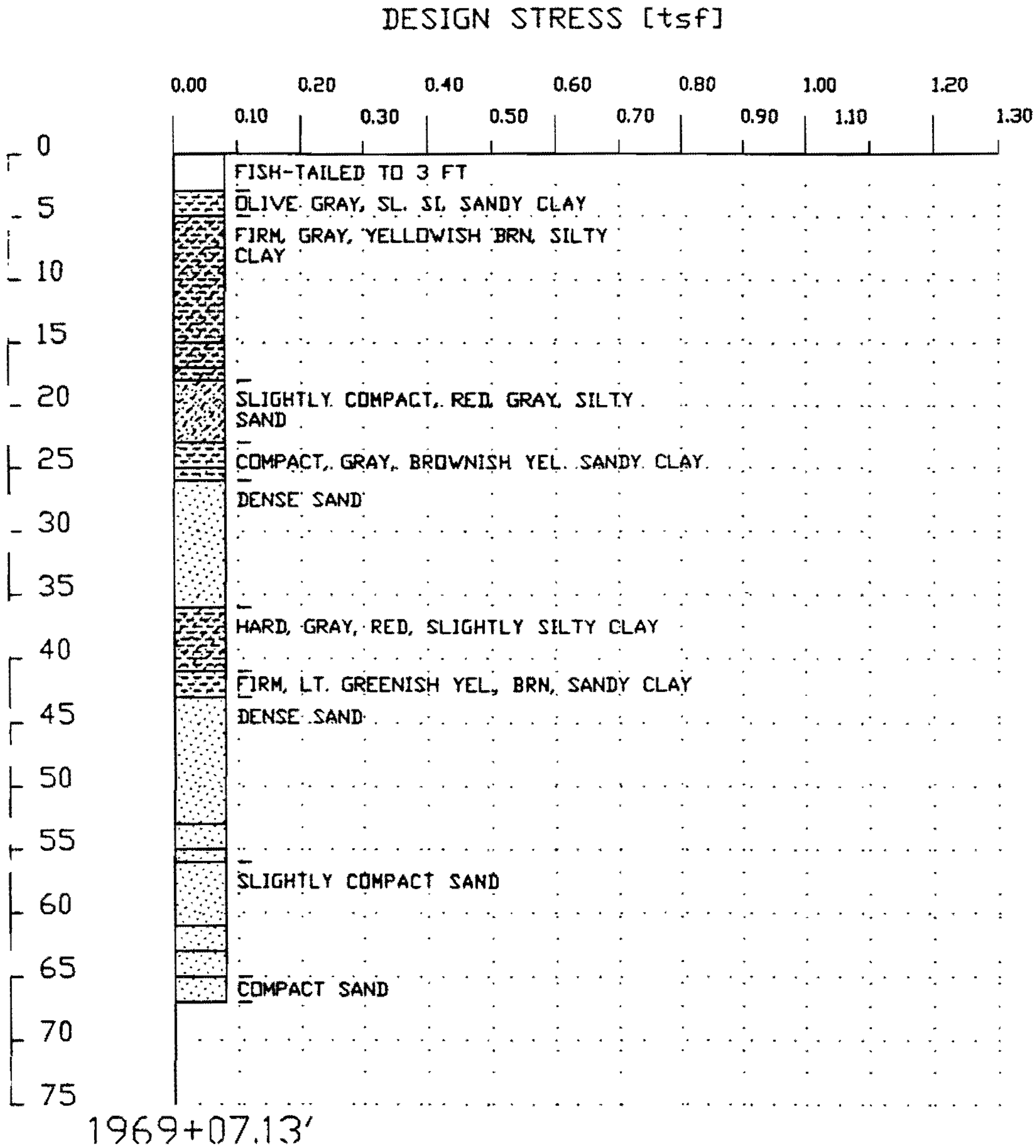
75
NO RECOVERY TO 2 FT
SL. COMP, GRN-YELL, SL. CL., SA/SI MIX W/ORG
SOFT, TAN, SILTY CLAY W/FERR. NODULES
FIRM, TAN, SILTY CLAY
SLIGHTLY COMPACT, BRN, SILTY SAND
COMPACT, WATER BEARING, BRN, SILTY SAND
NO RECOVERY
HARD, LT. GREENISH YELLOW, TAN, SILTY CLAY W/BLK NODULES
COMPACT, TAN, FINE & MED SAND W/SI PKT
NO RECOVERY, SAND

1966+68.68′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1.30
FISH-TAILED TO 3 FT
OLIVE GRAY, SL. SL SANDY CLAY
FIRM, GRAY, YELLOWISH BRN, SILTY CLAY
SLIGHTLY COMPACT, RED, GRAY, SILTY SAND
COMPACT, GRAY, BROWNISH YEL. SANDY CLAY
DENSE SAND
HARD, GRAY, RED, SLIGHTLY SILTY CLAY
FIRM, LT. GREENISH YEL., BRN, SANDY CLAY
DENSE SAND
SLIGHTLY COMPACT SAND
COMPACT SAND
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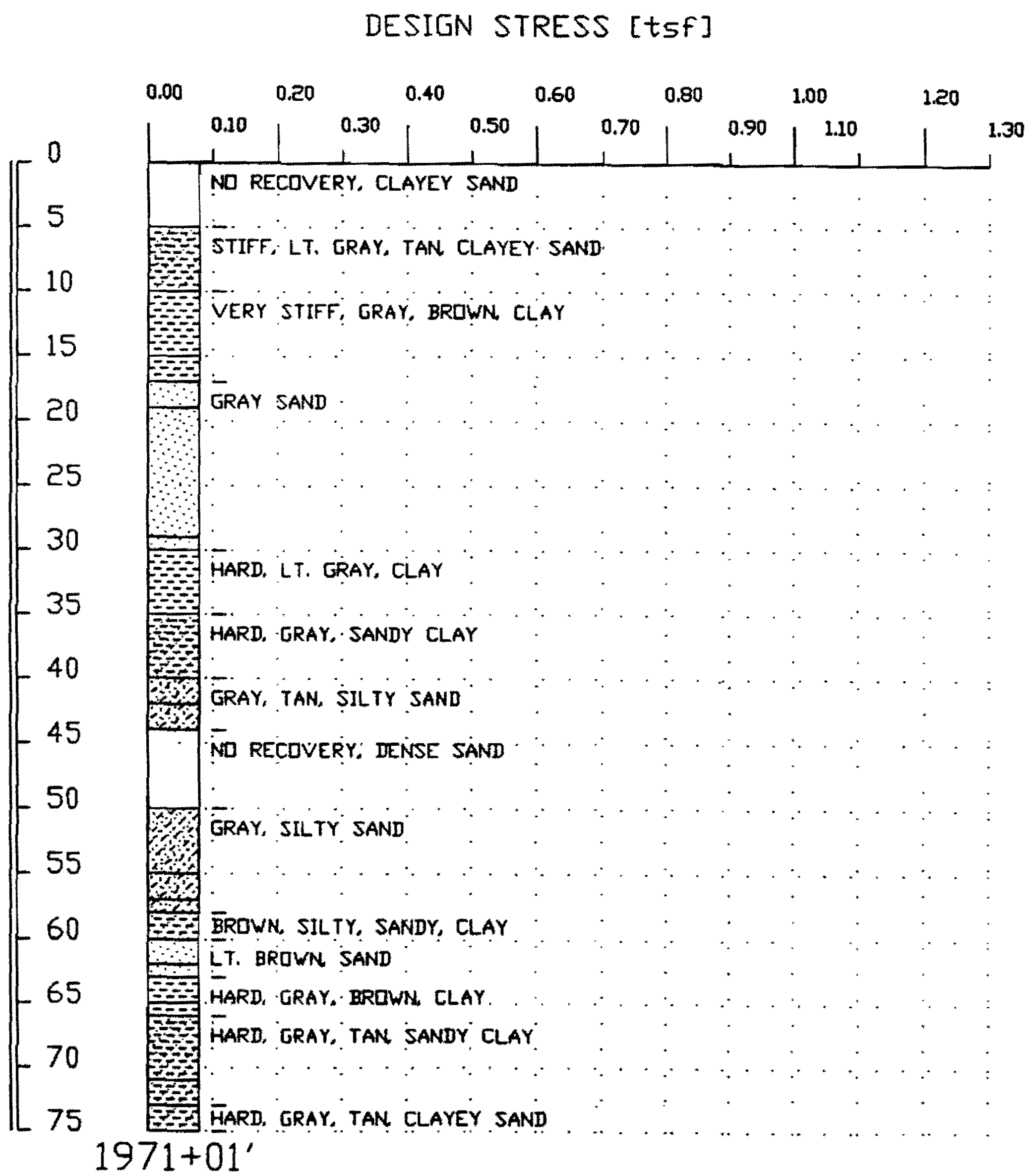
70
75
1969+07.13'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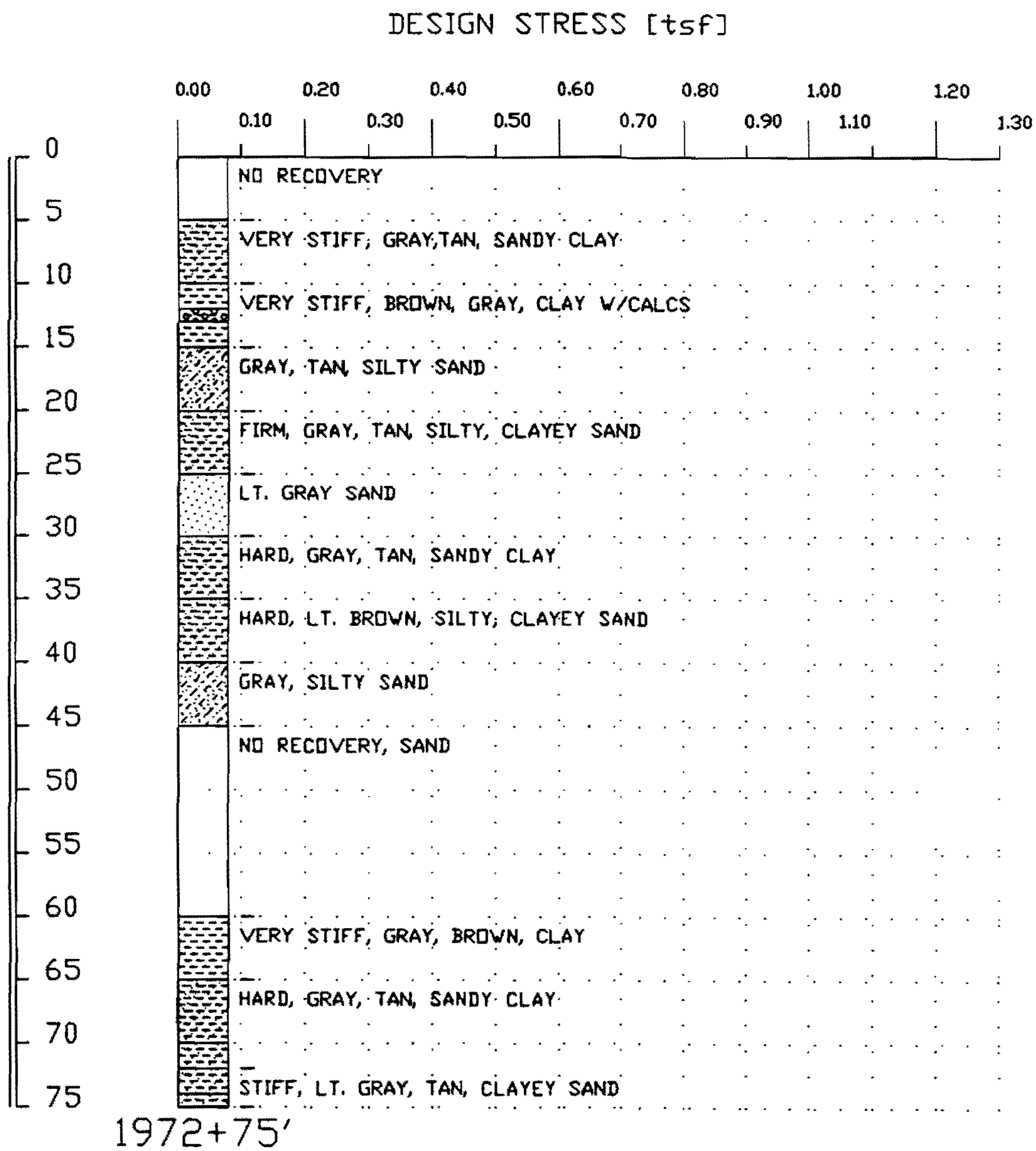
NO RECOVERY, CLAYEY SAND
STIFF, LT. GRAY, TAN, CLAYEY SAND
VERY STIFF, GRAY, BROWN, CLAY
GRAY SAND
HARD, LT. GRAY, CLAY
HARD, GRAY, SANDY CLAY
GRAY, TAN, SILTY SAND
NO RECOVERY, DENSE SAND
GRAY, SILTY SAND
BROWN, SILTY, SANDY, CLAY
LT. BROWN, SAND
HARD, GRAY, BROWN, CLAY
HARD, GRAY, TAN, SANDY CLAY
HARD, GRAY, TAN, CLAYEY SAND
1971+01'


DESIGN STRESS [tsf]
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
NO RECOVERY
VERY STIFF, GRAY,TAN, SANDY CLAY
VERY STIFF, BROWN, GRAY, CLAY W/CALCS
GRAY, TAN, SILTY SAND
FIRM, GRAY, TAN, SILTY, CLAYEY SAND
LT. GRAY SAND
HARD, GRAY, TAN, SANDY CLAY
HARD, LT. BROWN, SILTY, CLAYEY SAND
GRAY, SILTY SAND
NO RECOVERY, SAND
VERY STIFF, GRAY, BROWN, CLAY
HARD, GRAY, TAN, SANDY CLAY
STIFF, LT. GRAY, TAN, CLAYEY SAND
1972+75'

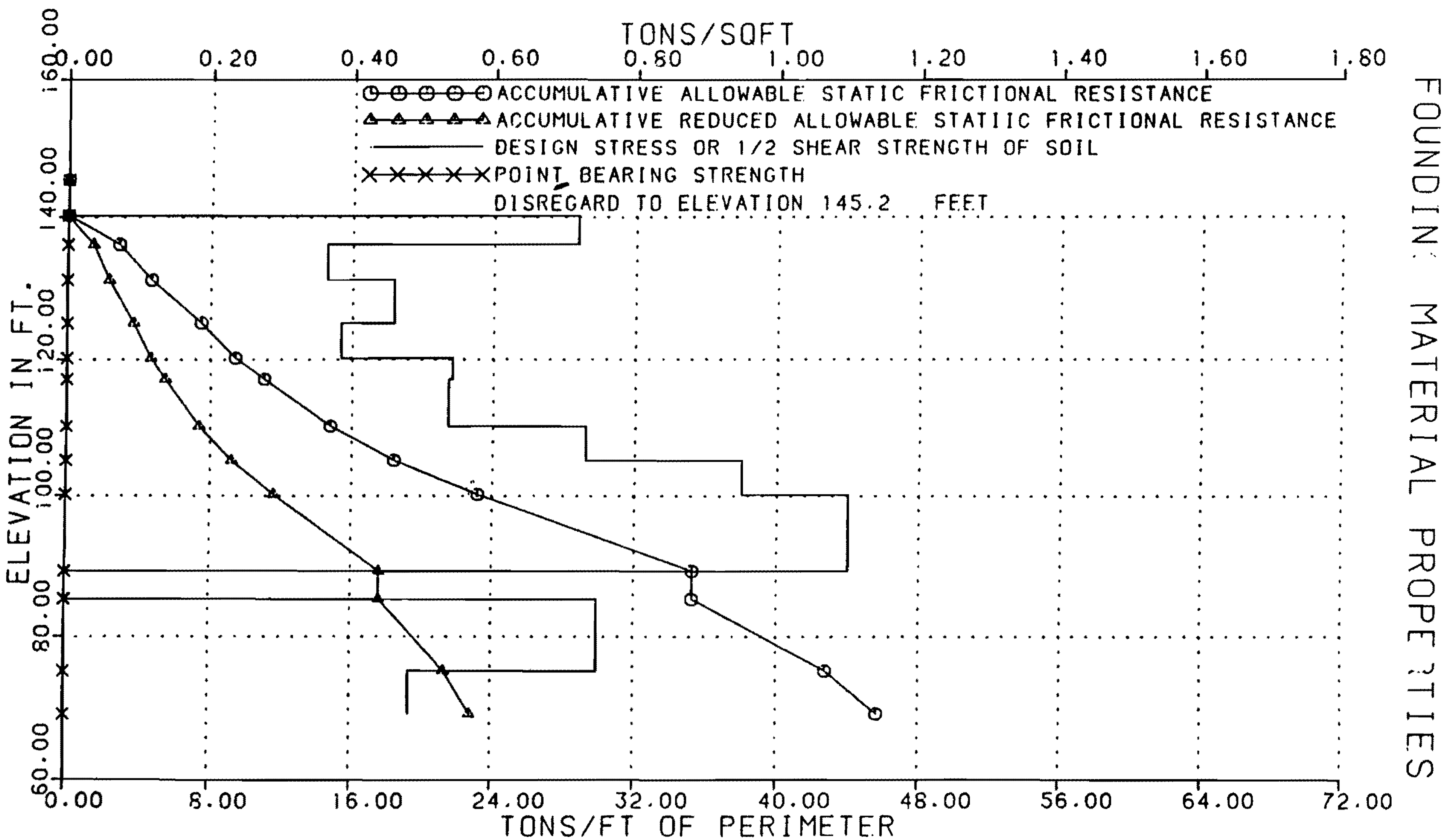
DRILLING LOG

COUNTY HARRIS | STRUCTURE BRIDGE * SKINNER ROAD | THD DIST 12
HIGHWAY NO US 290 | HOLE NO SK7 | DATE 05/01/85
CONTROL 0050-05-033 | STATION 1960+55 | GRD. ELEV. 145.2
IPE | LOCATION 31.0 FT RT | GRD. WATER ELEV. 145.2

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
145.2 0				GRAY, SANDY CLAY	
		5 (6.0")	7 (6.0")		
140.2				HARD, GRAY, TAN, SILTY, SANDY CLAY	
135.2 10		3 (6.0")	5 (6.0")	FIRM, GRAY, TAN CLAY W/SM. CALCS.	
131.2		5 (6.0")	8 (6.0")	VERY STIFF, BROWN, GRAY, SANDY CLAY	
125.2 20		12 (6.0")	19 (6.0")		
				STIFF, LIGHT GRAY, TAN, SANDY CLAY	
		17 (6.0")	21 (6.0")		
120.2				GRAY, TAN, SILTY, CLAYEY SAND	
117.2					
30		25 (6.0")	18 (6.0")	DRILLED-OUT, SAND	
		17 (6.0")	20 (6.0")		
110.2				HARD, GRAY, SILTY CLAY	
105.2 40		32 (6.0")	44 (6.0")		
				GRAY, DENSE SAND W/SM. SILT	
		32 (6.0")	37 (6.0")		
100.2				DRILLED-OUT, DENSE SAND	
50		46 (6.0")	52 (5.7")		
		43 (6.0")	50 (6.0")		
89.2				NO RECOVERY, SAND	
85.2 60		10 (6.0")	13 (6.0")		
				HARD, BROWN, GRAY, SILTY CLAY	
		15 (6.0")	19 (6.0")		
75.2 70		18 (6.0")	20 (6.0")		
				VERY STIFF, GRAY, SILTY, SANDY CLAY	
		34 (6.0")	25 (6.0")		

*REMARKS:

DRILLER MIKE BAHM | LOGGER R. CURSON, JOHN HOES | TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE * SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK7	DATE	05/01/86
CONTROL	0050-06-033	STATION	1960+55	BY	SJH
IPE		LOCATION	31.0 FT RT		

DRILLING LOG

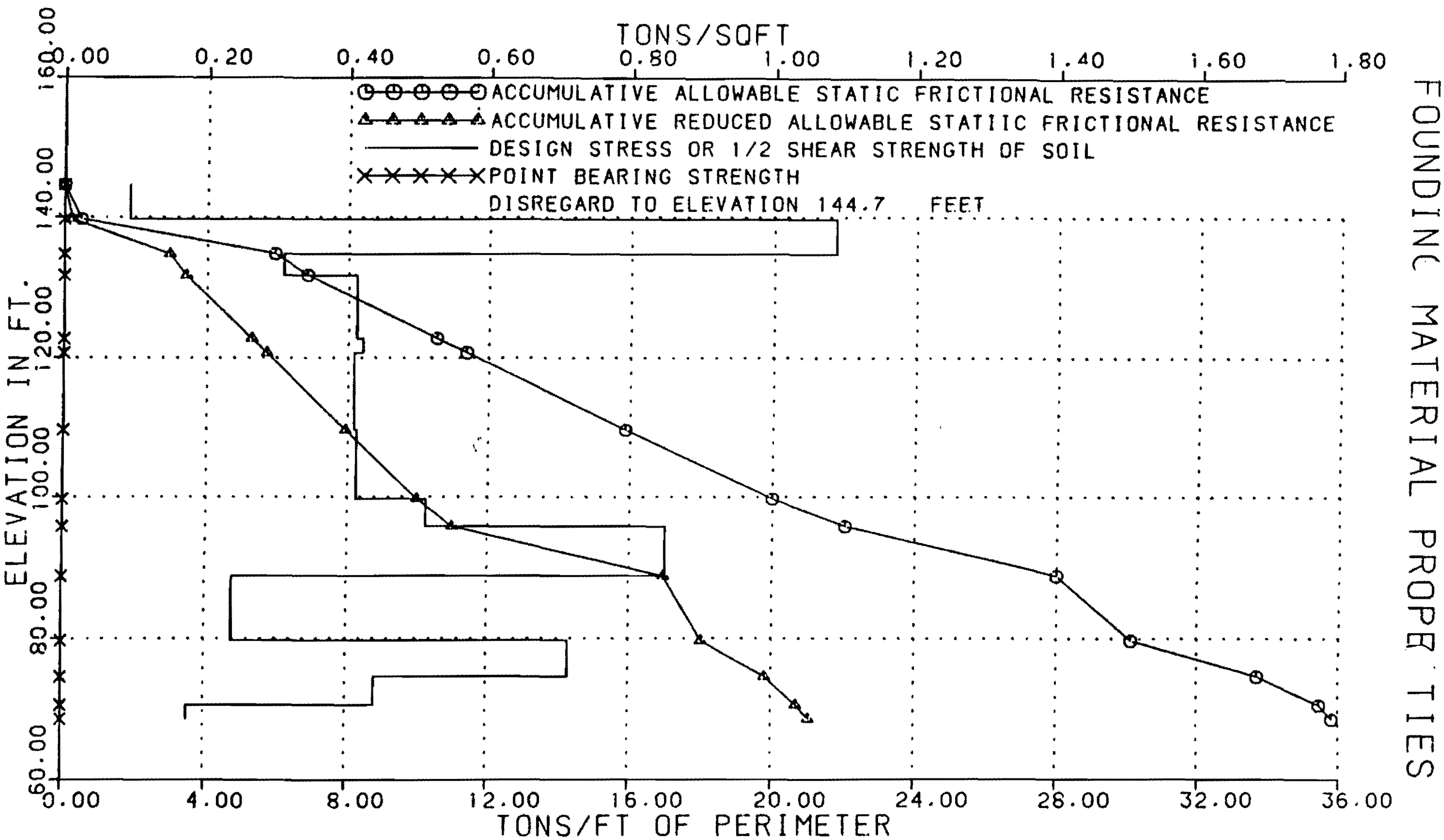
COUNTY HARRIS | STRUCTURE BRIDGE • SKINNER ROAD | THD DIST 12
HIGHWAY NO US 290 | HOLE NO SK5 | DATE 05/01/66
CONTROL 0050-06-033 | STATION 1962+46 | GRD. ELEV. 144.7
IPE | LOCATION 36.0 FT RT | GRD.WATER ELEV. 144.7

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
144.7	0				GRAY,SILTY CLAY	C	
5			8 (5.0")	10 (5.0")			
139.7					HARD,GRAY,TAN,SANDY CLAY		
5			8 (5.0")	8 (5.0")		10	
134.7	10				STIFF,GRAY,TAN CLAY		
3							
131.7			7 (5.0")	10 (6.0")	VERY STIFF,GRAY,BROWN,SANDY CLAY		
9							
	20		7 (5.0")	10 (6.0")		20	
122.7					VERY STIFF,GRAY,TAN,CLAYEY SAND		
120.7			12 (5.0")	25 (5.0")	NO-RECOVERY,SAND		
	30		40 (5.0")	27 (5.0")		30	
109.7			16 (6.0")	16 (5.0")	DRILLED-OUT,SAND		
	40		15 (5.0")	17 (5.0")		40	
99.7			15 (6.0")	25 (5.0")	LIGHT GRAY,FINE SAND W/V. SL.CLAY		
95.7	50		[illegible]	57 (5.0")	DRILLED-OUT,DENSE SAND	50	
			50 (4.5")	50 (3.5")			
86.7					DRILLED-OUT,SAND		
7	60		9 (5.0")	10 (5.0")		50	
79.7			12 (5.0")	12 (5.0")	HARD,GRAY,TAN,SANDY CLAY		
5							
74.7	70		21 (5.0")	21 (5.0")	STIFF,LIGHT GRAY,CLAYEY SAND	70	
4							
70.7			50 (4.[illegible])	50 (5.0")	DENSE SAND		

•REMARKS:

DRILLER MIKE BAHM LOGGER R. CURSON. JOHN HOES TITLE

C20



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK6	DATE	05/01/66
CONTROL	0050-05-033	STATION	1962+46	BY	SJH
IPE		LOCATION	36.0 FT RT		

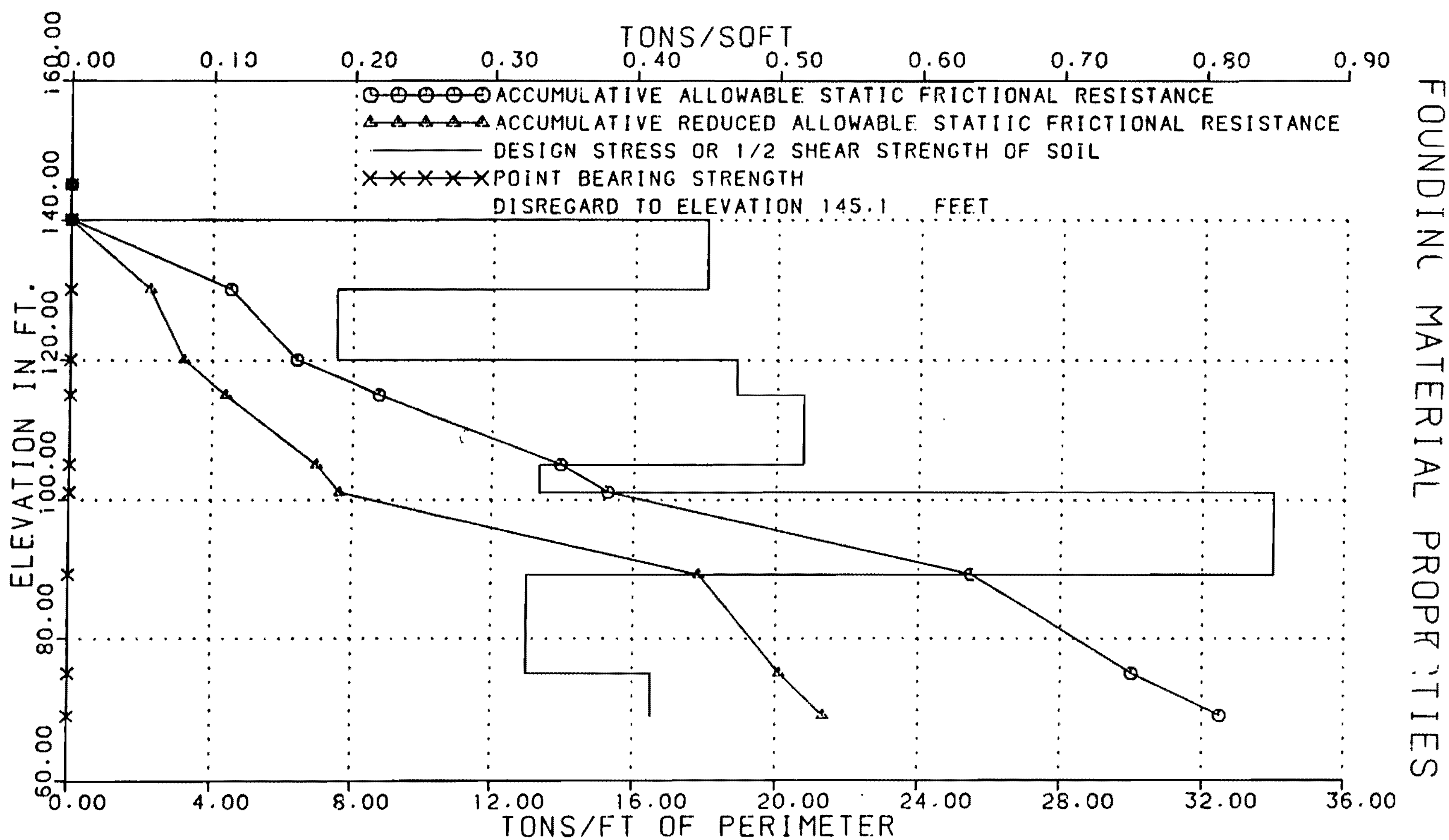
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK5	DATE	05/01/66
CONTROL	0050-06-033	STATION	1964+30	GRD. ELEV.	145.1
IPE		LOCATION	15.1 FT RT	GRD.WATER ELEV.	145.1

ELEV FT.	DEPTH	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
145.1	0				NO RECOVERY	
			5 (6.0")	3 (6.0")		
140.1					VERY STIFF, GRAY, TAN CLAY W/SM. CALCS	
	10		9 (6.0")	5 (6.0")		
			4 (6.0")	6 (6.0")		
130.1					FIRM, GRAY, BROWN, CLAYEY SAND	
	20		6 (6.0")	7 (6.0")		
			12 (6.0")	21 (6.0")		
120.1					LIGHT GRAY, SILTY, CLAYEY SAND	
115.1	30		16 (6.0")	36 (6.0")		
					DRILLED-OUT, SAND	
			13 (6.0")	16 (6.0")		
105.1	40		13 (6.0")	13 (6.0")		
					STIFF, LIGHT GRAY, CLAYEY SAND	
101.1			50 (5.5")	50 (5.0")	DRILLED-OUT, DENSE SAND	
	50		45 (6.0")	32 (6.0")		
			50 (3.5")	50 (3.0")		
89.1					DRILLED-OUT, SAND	
	60		18 (6.0")	14 (6.0")		
			8 (6.0")	12 (6.0")		
75.1	70		20 (6.0")	24 (6.0")		
					DRILLED-OUT, SAND	
			12 (6.0")	13 (6.0")		

*REMARKS: GWE OBSD. @ END OF DRLG.

DRILLER MIKE BAHM LOGGER R. CURSON, JOHN HOES TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK5	DATE	05/01/85
CONTROL	0050-06-033	STATION	1964+30	BY	SJH
IPE		LOCATION	16 I FT RT		

DRILLING LOG

COUNTY	· HARRIS	STRUCTURE	BEAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	2	DATE	04/01/65
CONTROL	0050-06-033	STATION	1966+56.56	GRD. ELEV.	144.4
IPE		LOCATION	10.5 FT. RT	GRD. WATER ELEV.	144.4

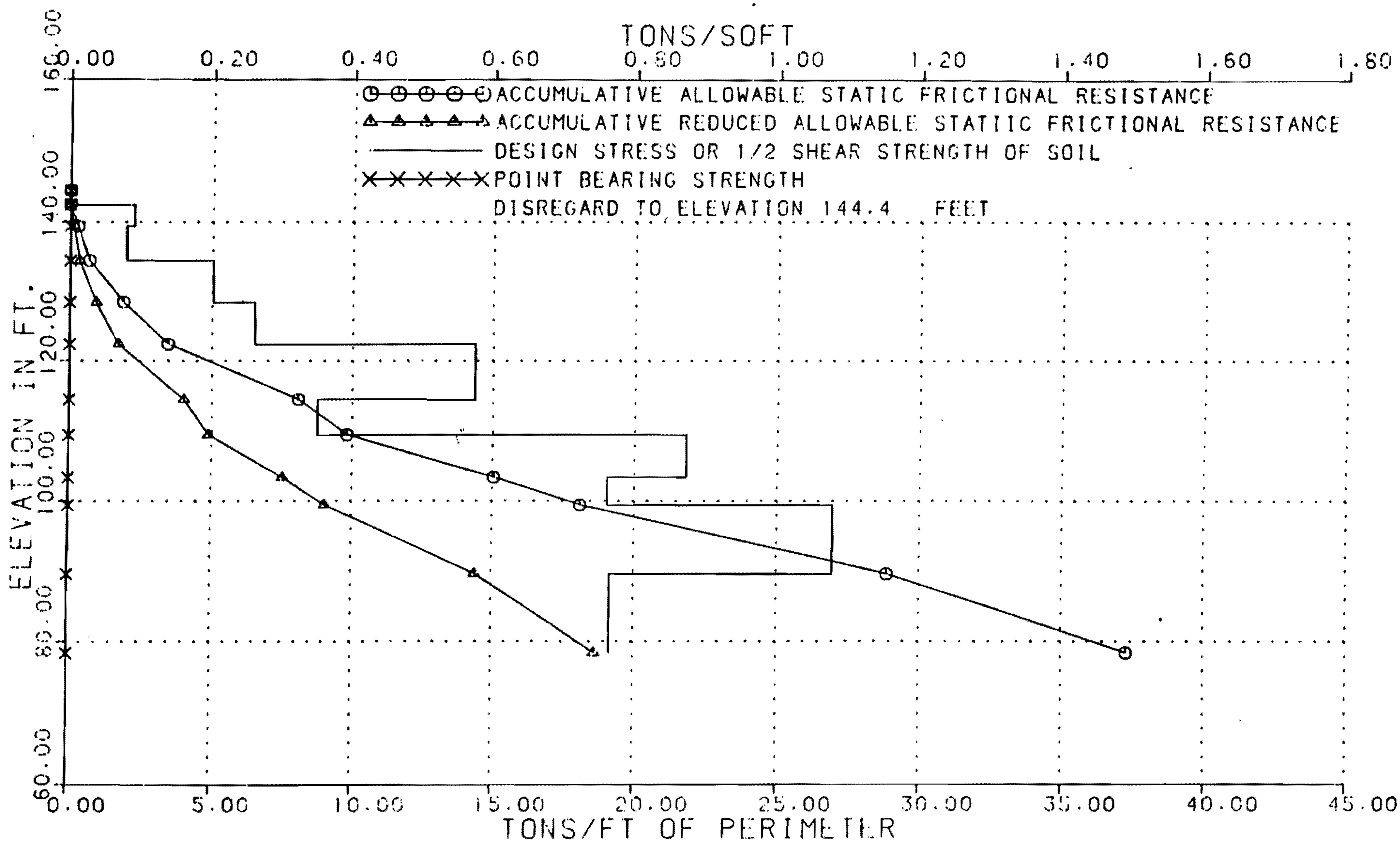
ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
144.4	0					
142.4					NO RECOVERY TO 2 FT	
139.4			5 (6.0")	5 (6.0")	SL. COMP..GY.SL.CL..SA-SI MIX W/ORG	
134.4	10		6 (6.0")	7 (6.0")	SOFT.TAN.SILTY CLAY W/FER. NOD.	
			10 (6.0")	10 (6.0")	FIRM.TAN.SILTY CLAY	
126.4						
	20		12 (6.0")	20 (6.0")	SLIGHT COMPACT.BROWN SILTY SAND	
122.4			20 (6.0")	26 (6.0")	COMPACT.WATER BEARING.BN.SILTY SAND	
114.4	30		12 (6.0")	16 (6.0")		
109.4			13 (6.0")	16 (6.0")	NO RECOVERY	
	40		15 (6.0")	16 (6.0")	HARD.LT.GY.TAN.SILTY CLAY W/BL. NOD	
103.4						
99.4			15 (6.0")	40 (6.0")	HARD.LT.GY.TAN.V.SL.SI.CL. W/BL.NOD	
	50		50 (4.0")	46 (6.0")	COMPACT.TAN.FINE & MD.SAND W/SI PKT	
83.4			47 (6.0")	46 (6.0")		
	60		40 (6.0")	26 (6.0")	NO RECOVERY	
			10 (6.0")	13 (6.0")		

•REMARKS:

DRILLER W. WILLMAN/M. BAHM LOGGER R. K. CURSON TITLE

C24

FOUNDING MATERIAL PROPERTIES



COUNTY	HARRIS	STRUCTURE	BEAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	2	DATE	04/01/85
CONTROL	0050-06-033	STATION	1066+68.68	BY	SFY
IC		LOCATION	10.5 FT. RT		

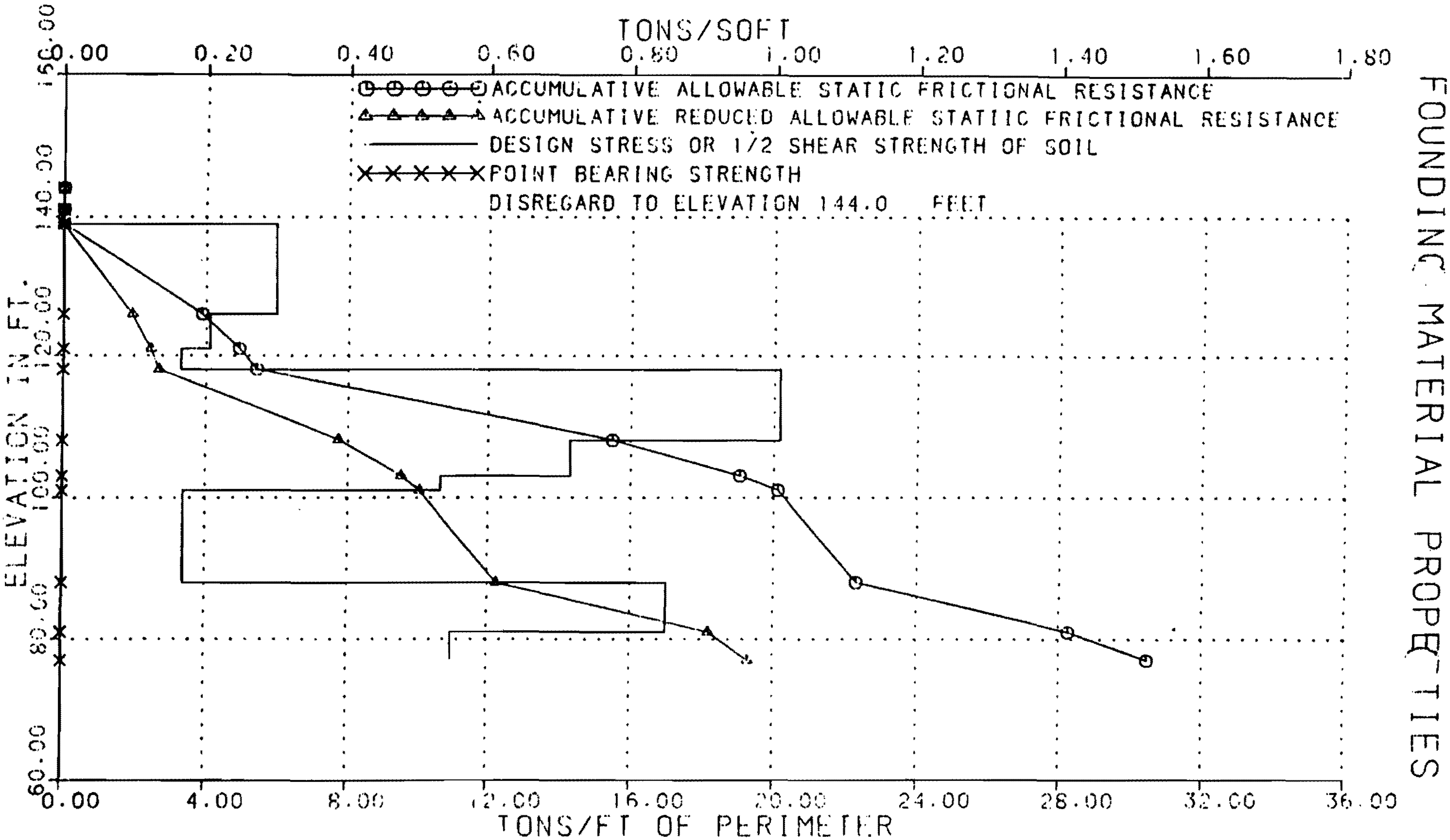
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	BEAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	1	DATE	04/01/65
CONTROL	0050-06-033	STATION	1969+07.13	GRD. ELEV.	144.0
IPE		LOCATION	3.28 FT. RT	GRD. WATER ELEV.	144.0

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
						0	
144.0	0				FISH-TAILED TO 3 FT.		
141.0					OLIVE GRAY, SL. SI., SANDY CLAY		
139.0			4 (6.0")	4 (6.0")	FIRM, GRAY, YELLOWISH BN. SILTY CLAY		
	10		7 (6.0")	6 (6.0")		10	
			10 (6.0")	11 (6.0")			
125.0					SLIGHTLY COMPACT, RD. GRAY, SILTY SAND		
	20		14 (6.0")	11 (6.0")		20	
121.0					COMPACT, GRAY, BRNISH YEL., SANDY CLAY		
			20 (6.0")	37 (6.0")			
116.0					DENSE SAND		
	30		50 (6.0")	50 (5.5")		30	
			15 (6.0")	17 (6.0")			
102.0					HARD, GRAY, RED, SLIGHT SILTY CLAY		
	40		15 (6.0")	17 (6.0")		40	
103.0					FIRM, LT. GY., BN., SANDY CLAY		
101.0					DENSE SAND		
			46 (6.0")	40 (6.0")			
	50		50 (3.0")	50 (2.2")		50	
			50 (3.2")	50 (2.7")			
8E.0					SLIGHTLY COMPACT SAND		
	60		12 (6.0")	12 (6.0")		60	
61.0					COMPACT SAND		
			21 (6.0")	23 (6.0")			

•REMARKS·

DRILLER W. WILLMAN/M. BAHM LOGGER R. K. CORSON TITLE



C26

COUNTY	HARRIS	STRUCTURE	BEAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	1	DATE	04/01/65
CONTROL	0050-06-033	STATION	1969+07.13	BY	SFY
IPE		LOCATION	3.28 FT. RT		

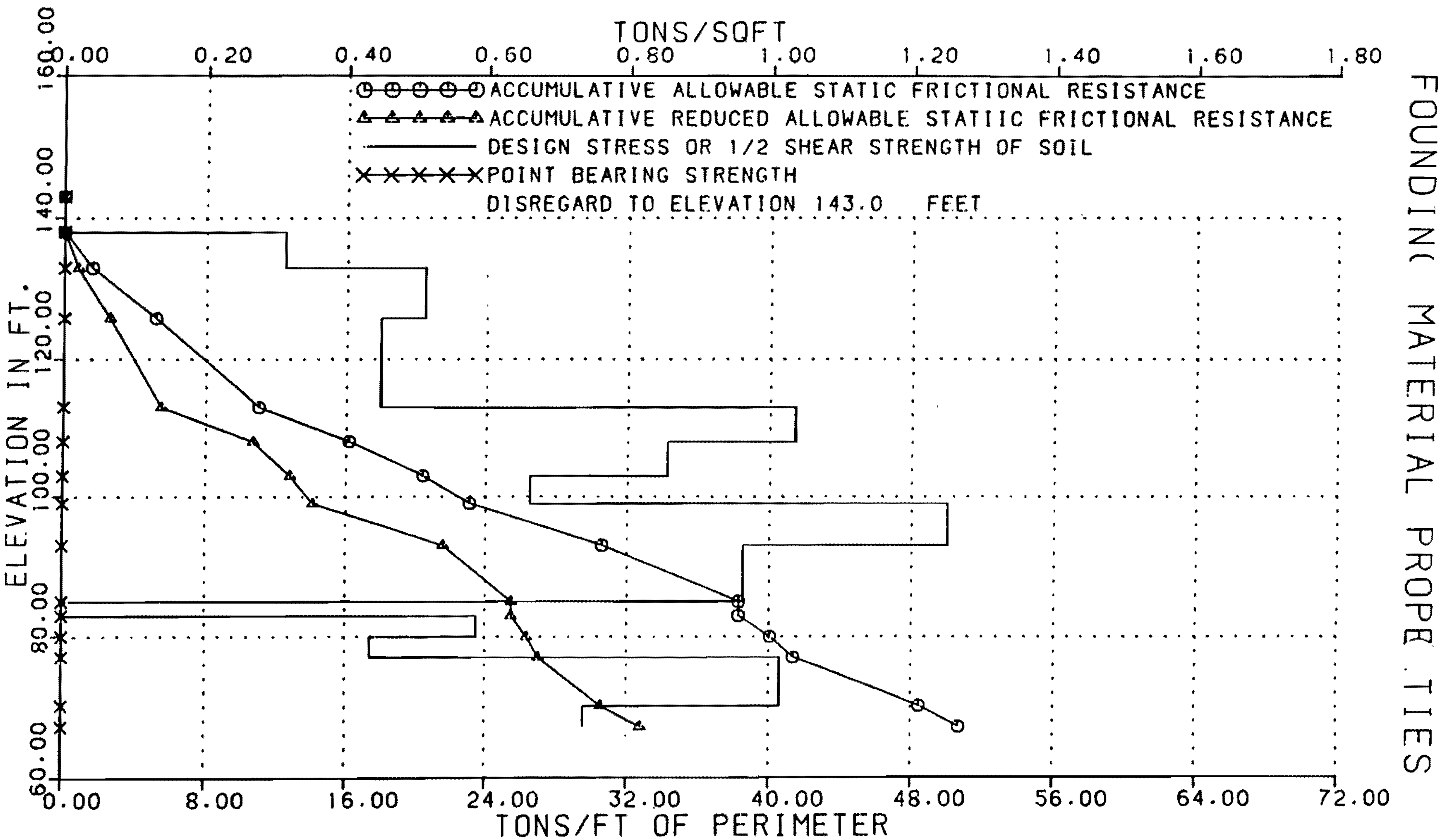
DRILLING LOG

COUNTY HARRIS | STRUCTURE BRIDGE @ SKINNER ROAD | THD DIST 12
HIGHWAY NO US 290 | HOLE NO SK3 | DATE 05/01/86
CONTROL 0050-05-033 | STATION 1971+01 | GRD. ELEV. 143.0
IPE | LOCATION 19.6 FT RT | GRD. WATER ELEV. 143.0

ELEV FT.	DEPTH	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
143.0	0				CLAYEY SAND, NO CORE	
			5 (6.0")	3 (6.0")		
136.0					STIFF, LIGHT GRAY, TAN, CLAYEY SAND	
133.0	10		3 (6.0")	3 (6.0")	VERY STIFF, GRAY, BROWN CLAY	
			6 (6.0")	7 (6.0")		
125.0					GRAY SAND	
	20		10 (6.0")	11 (6.0")		
			23 (6.0")	32 (6.0")		
113.0	30		50 (3.3")	29 (6.0")	HARD, LIGHT GRAY CLAY	
			15 (6.0")	16 (6.0")		
108.0					HARD, GRAY, SANDY CLAY	
103.0	40		20 (6.0")	33 (6.0")	GRAY, TAN, SILTY SAND	
99.0			50 (3.3")	50 (2.3")	NO RECOVERY, DENSE SAND	
93.0	50		50 (6.0")	50 (6.0")	GRAY, SILTY SAND	
			27 (6.0")	18 (6.0")		
85.0					BROWN, SILTY, SANDY CLAY	
83.0	60		29 (6.0")	16 (6.0")	LIGHT BROWN SAND	
80.0			12 (6.0")	14 (6.0")	HARD, GRAY, BROWN CLAY	
77.0					HARD, GRAY, TAN, SANDY CLAY	
	70		25 (6.0")	23 (6.0")		
70.0			46 (6.0")	54 (4.5")	HARD, GRAY, TAN, CLAYEY SAND	

*REMARKS: GWE OBSD. @ END OF DRLG.

DRILLER MIKE BAHM LOGGER R. CURSON, JOHN HOES TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK3	DATE	05/01/86
CONTROL	0050-05-033	STATION	1971+01	BY	SJH
		LOCATION	12.6 FT RT		

DRILLING LOG

COUNTY HARRIS | STRUCTURE BRIDGE @ SKINNER ROAD | THD DIST 12
HIGHWAY NO US 290 | HOLE NO SK4 | DATE 05/01/86
CONTROL 0050-06-033 | STATION 1972+75 | GRD. ELEV. 142.3
IPE | LOCATION 21.7 FT RT | GRD. WATER ELEV. 142.3

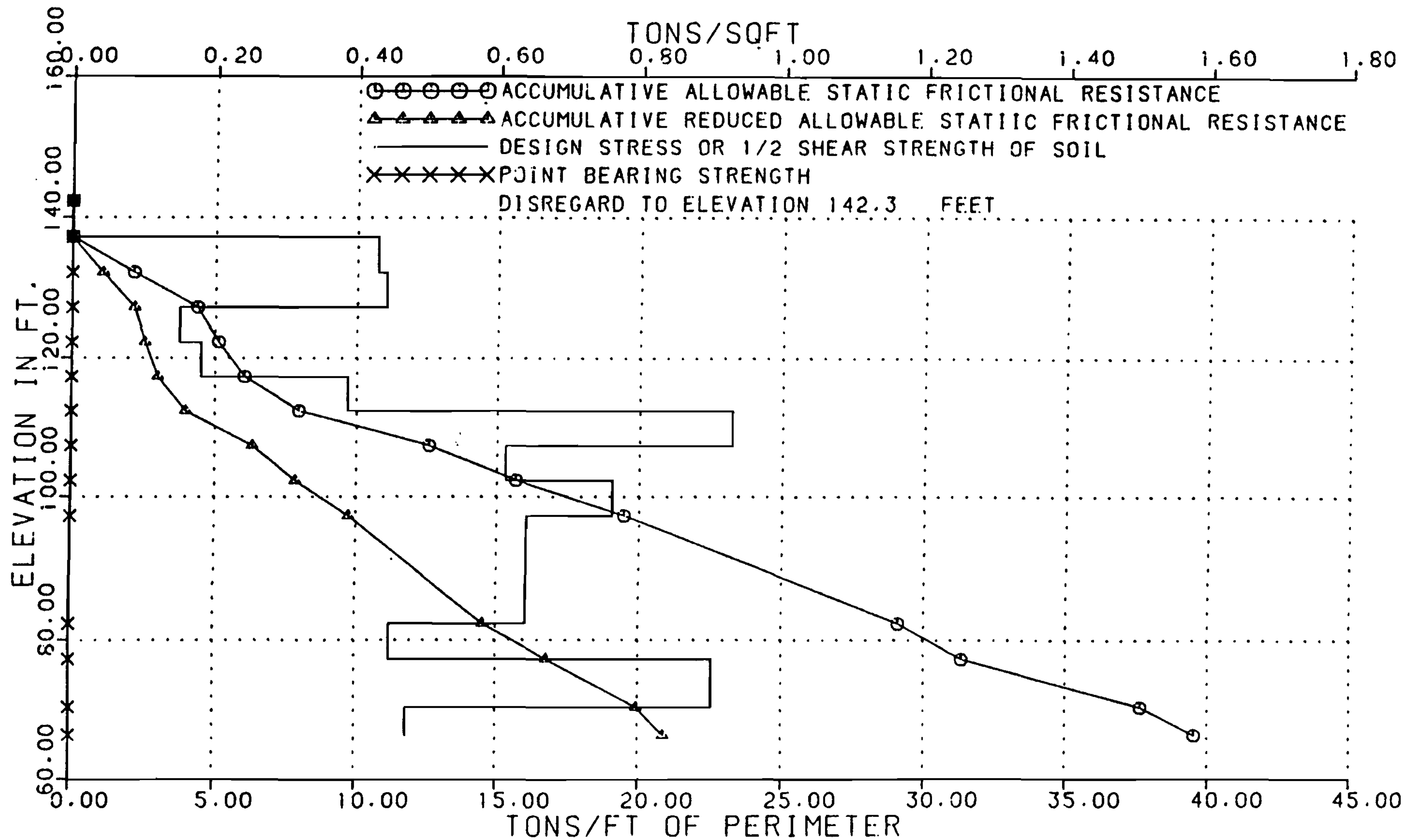
ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.3 0			5 (6.0")	4 (6.0")	NO RECOVERY	
137.3			3 (6.0")	3 (6.0")	VERY STIFF, GRAY, TAN, SANDY CLAY	
132.3 10			6 (6.0")	5 (6.0")	VERY STIFF, BROWN, GRAY CLAY W/CALCS.	
127.3			15 (6.0")	12 (6.0")	GRAY, TAN, SILTY SAND	
122.3 20			12 (6.0")	19 (6.0")	FIRM, GRAY, TAN, SILTY, CLAYEY SAND	
117.3			31 (6.0")	15 (6.0")	LIGHT GRAY SAND	
112.3 30			16 (6.0")	16 (6.0")	HARD, GRAY, TAN, SANDY CLAY	
107.3			35 (6.0")	25 (6.0")	HARD, LIGHT BROWN, SILTY, CLAYEY SAND	
102.3 40			27 (6.0")	31 (6.0")	GRAY, SILTY SAND	
97.3			21 (6.0")	19 (6.0")	NO RECOVERY, SAND	
50			33 (6.0")	23 (6.0")		
			50 (5.0")	45 (6.0")		
82.3 60			18 (6.0")	20 (6.0")	VERY STIFF, GRAY, BROWN CLAY	
77.3			26 (6.0")	30 (6.0")	HARD, GRAY, TAN, SANDY CLAY	
70						
70.3			44 (6.0")	56 (6.0")	STIFF, LIGHT GRAY, TAN, CLAYEY SAND	

*REMARKS: GWE OBSD. @ END OF DRLG.

DRILLER MIKE BAHM | LOGGER R. CURSON, JOHN HOES | TITLE

C30

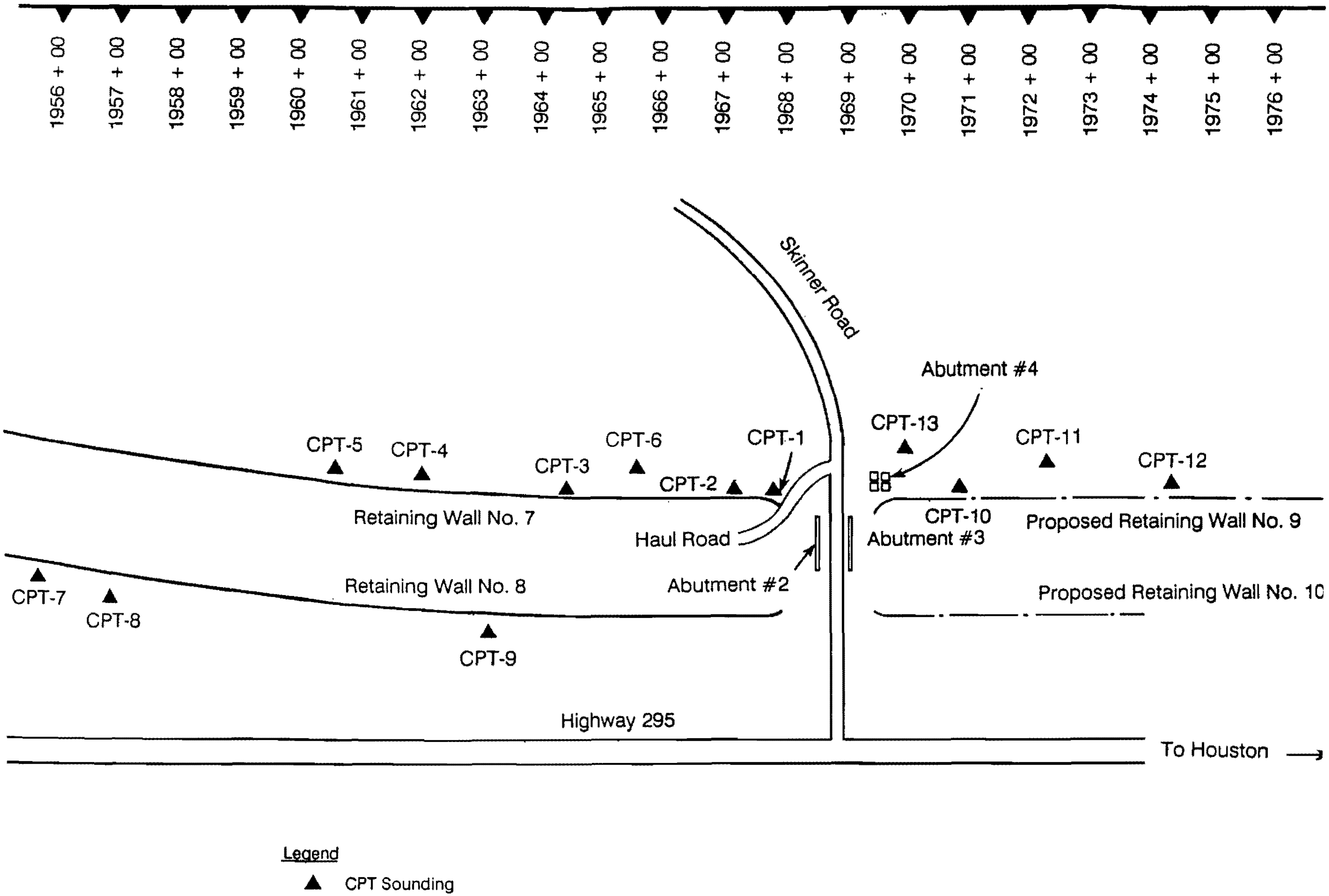
FOUNDINC MATERIAL PROPE TIES



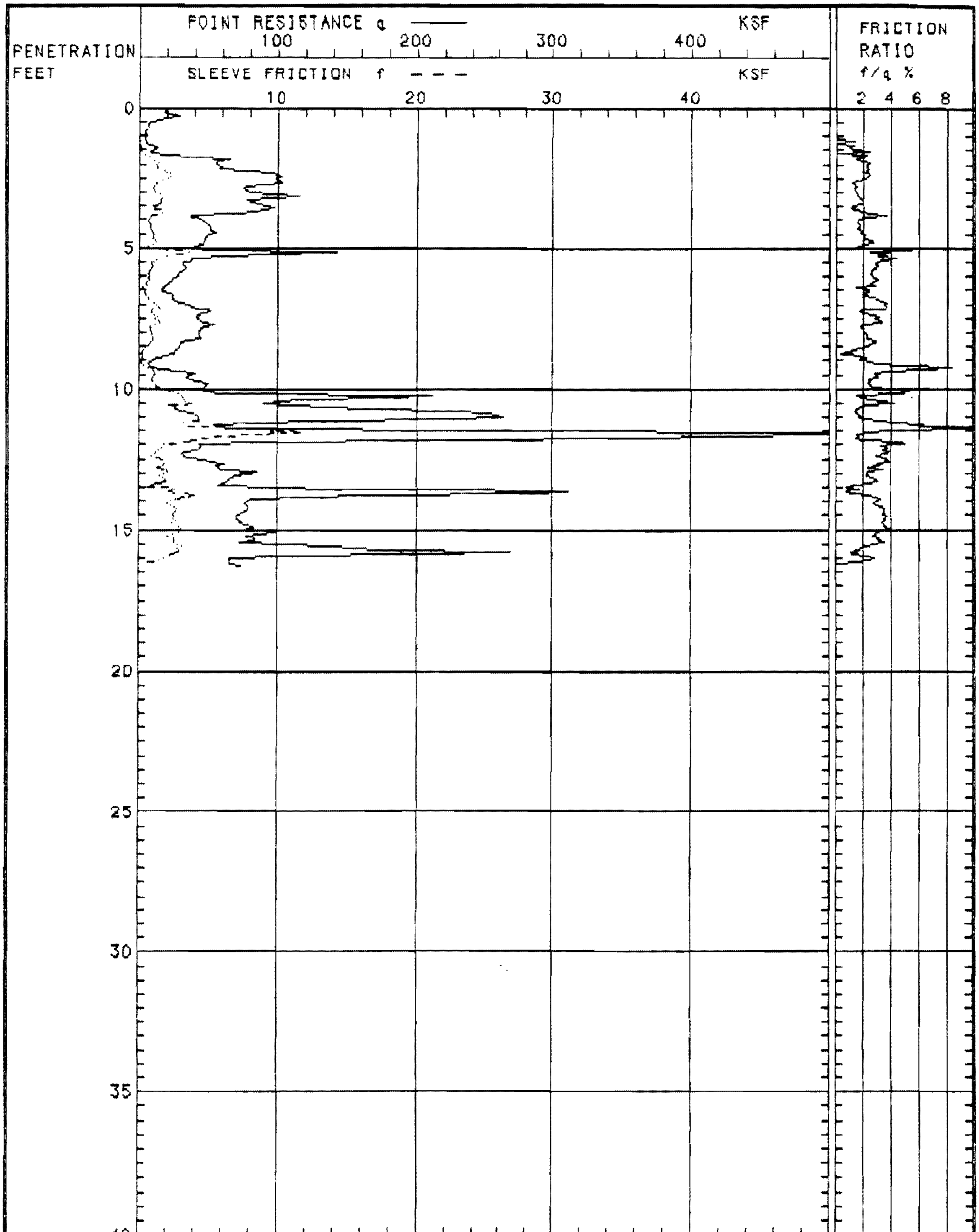
COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK4	DATE	05/01/86
CONTROL	0050-06-033	STATION	1972+75	BY	SJH
IPE		LOCATION	21.7 FT RT		

APPENDIX D

RESULTS OF CONE PENETRATION TESTING



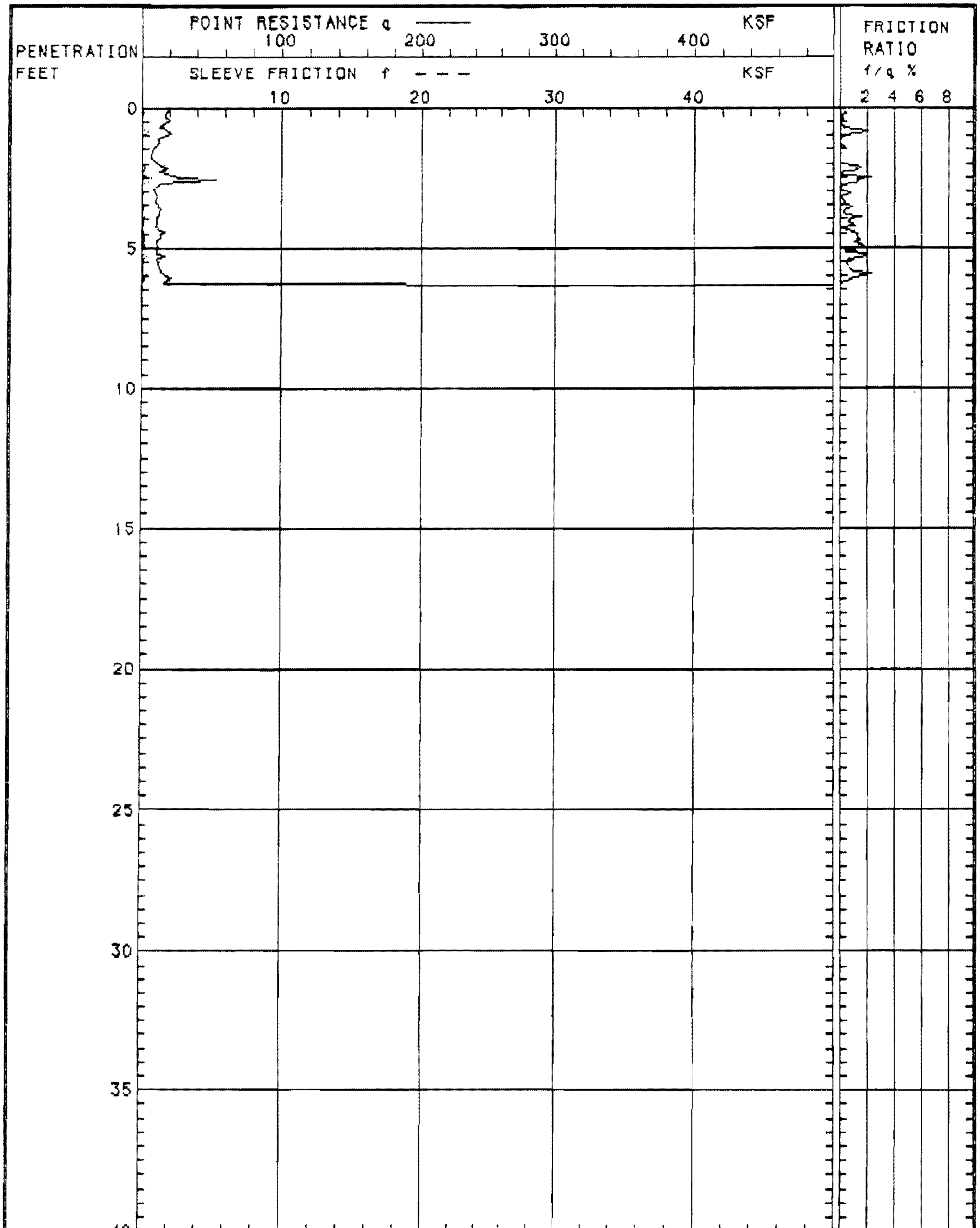
Location of cone penetrometer soundings



RESULT OF CONE PENETRATION TEST

CPT 1

JOB NUMBER: 0601-0312
GROUND SURFACE ELEV.: 144.4 FEET
CONE NUMBER: 10
DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 7 STA. 1987+60: 1'N
DATE: 03-07-1990



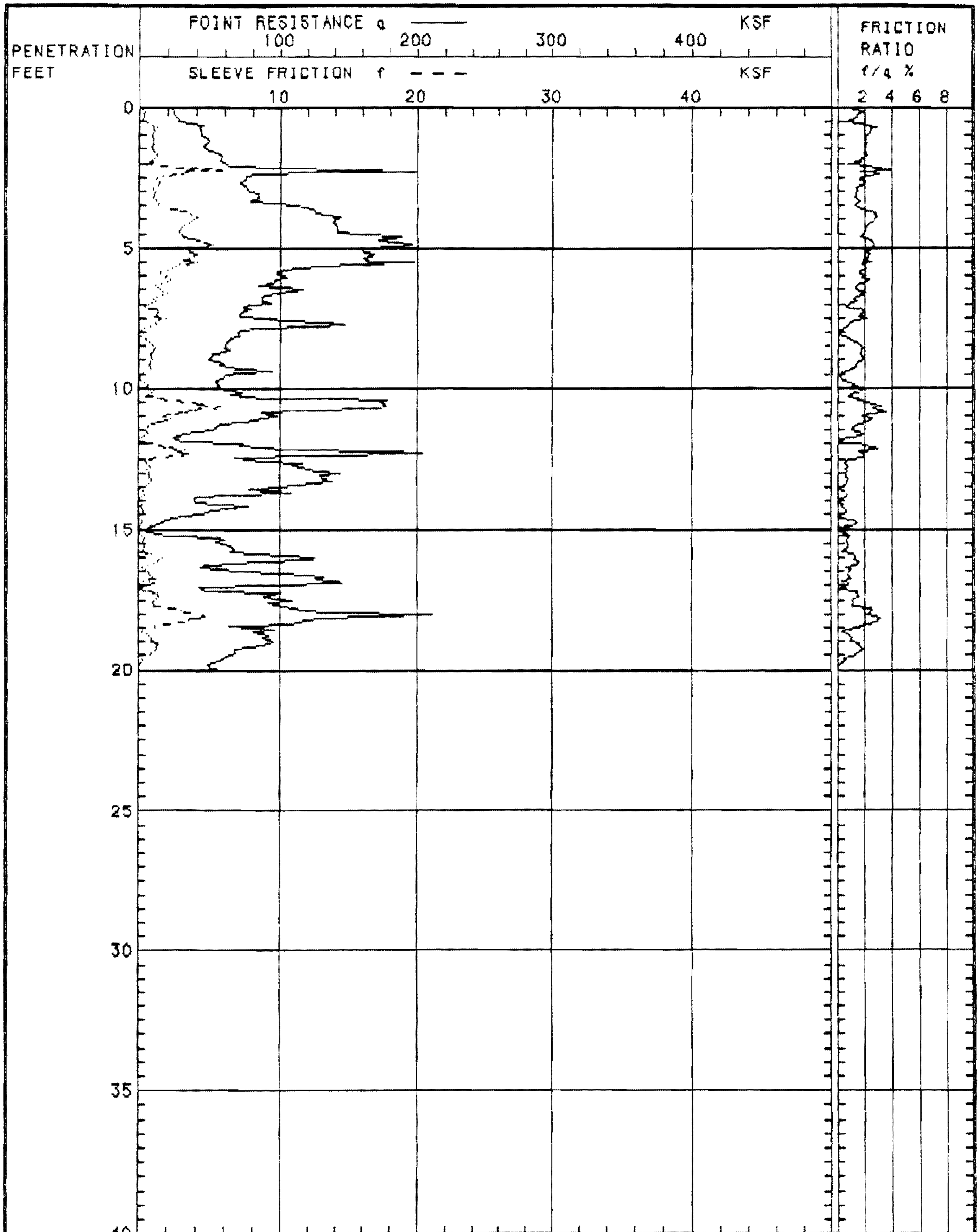
RESULT OF CONE PENETRATION TEST

CPT 2

JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 144.7 FEET

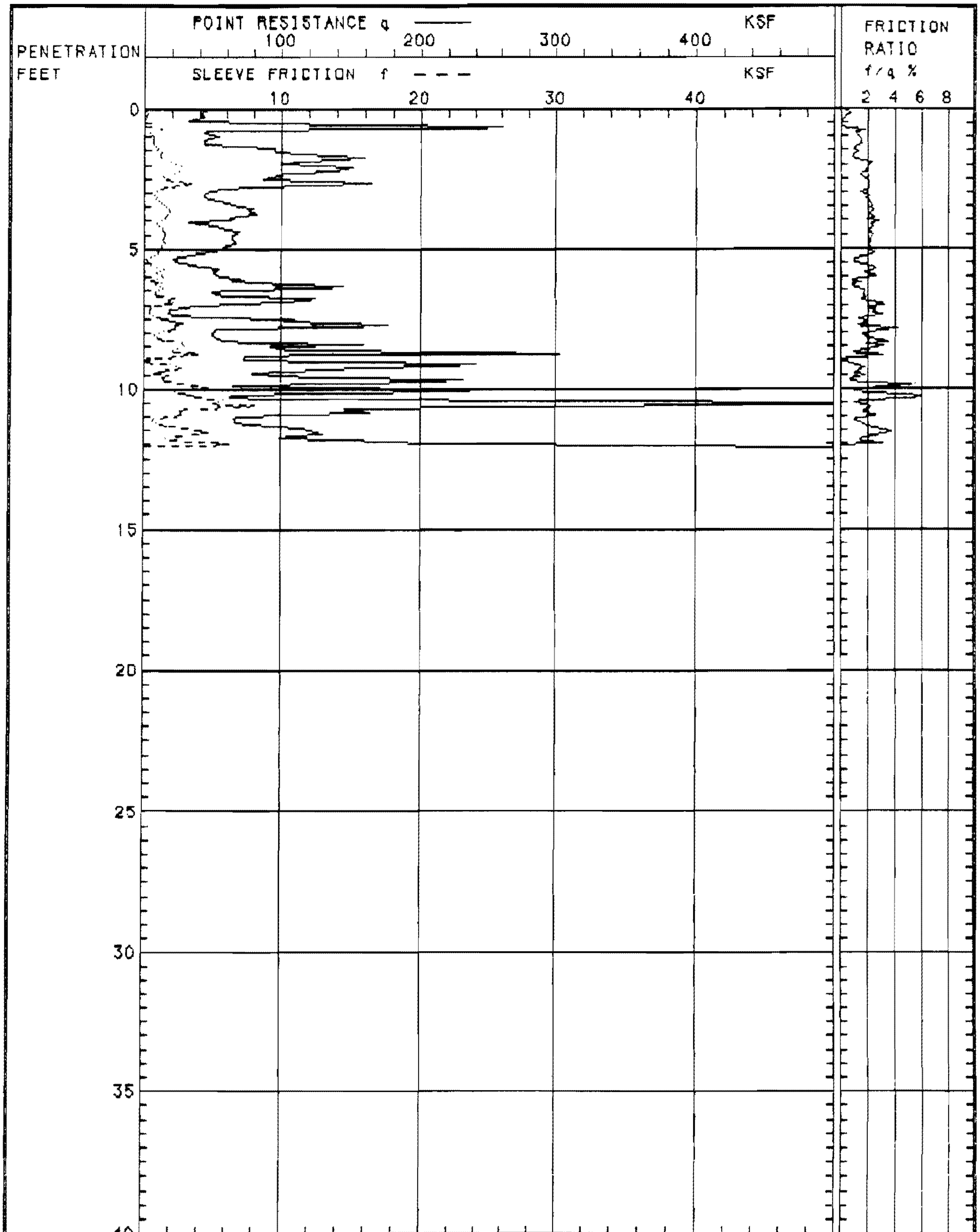
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET

LOCATION: CYPRESS BYPASS WALL 7 STA. 1987+00: 3'N DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST
CPT 3

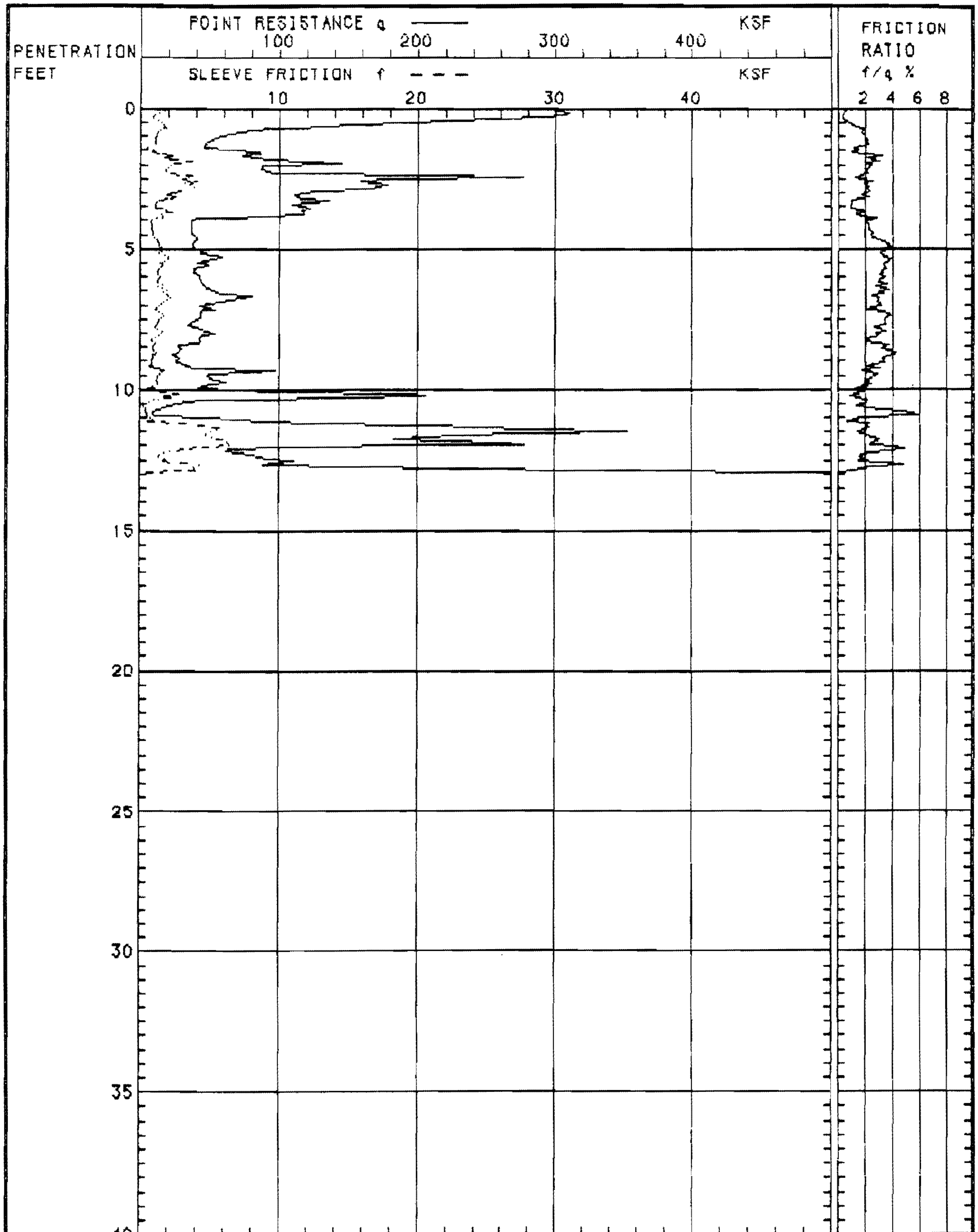
JOB NUMBER:	0601-0312	GROUND SURFACE ELEV.:	144.1 FEET
CONE NUMBER:	10	DEPTH TO GROUNDWATER:	0.0 FEET
LOCATION:	CYPRESS BYPASS WALL 7 STA. 1964+30: 30'N	DATE:	03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 4

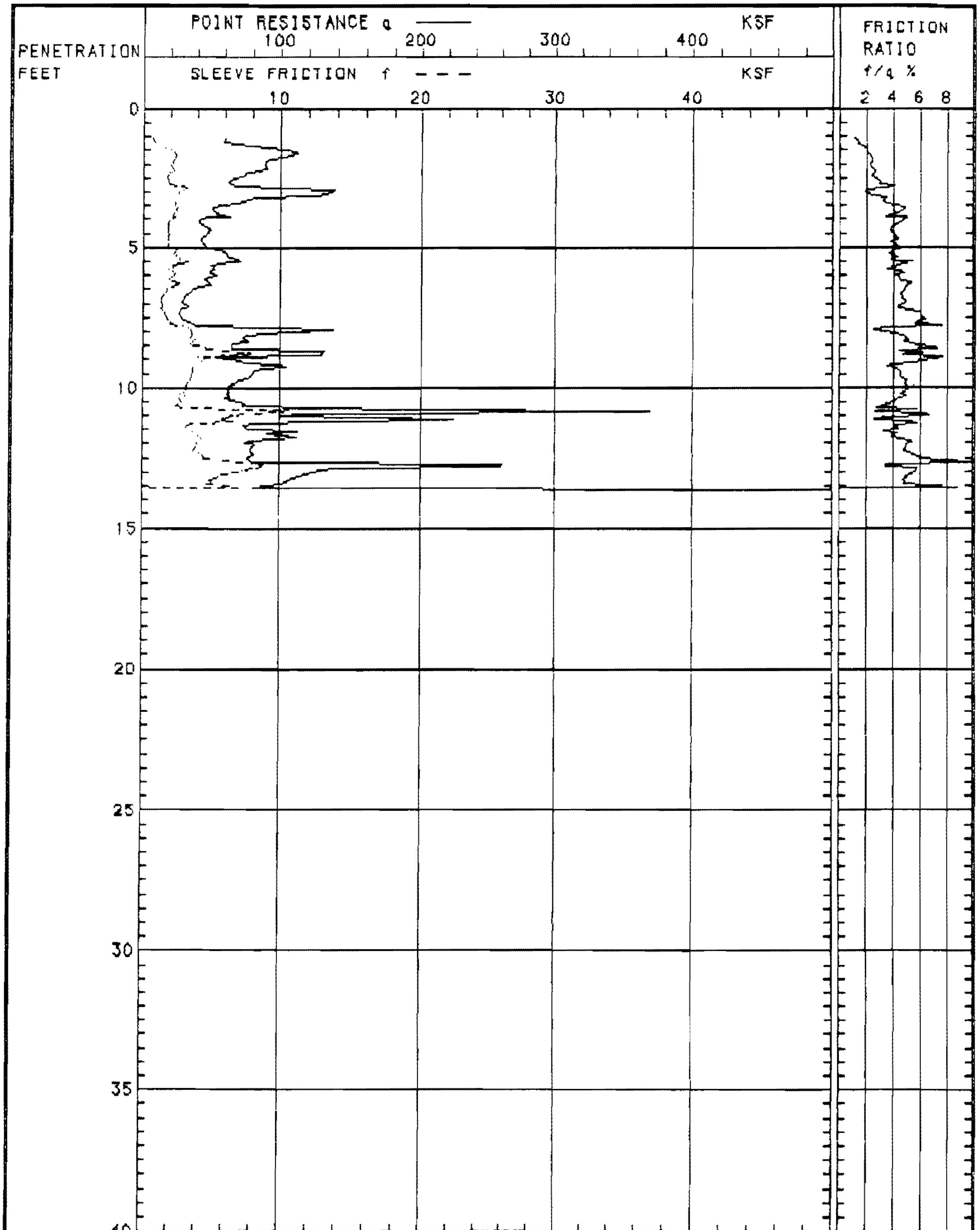
JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 144.7 FEET
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 7 STA. 1962+00: 17'N DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 5

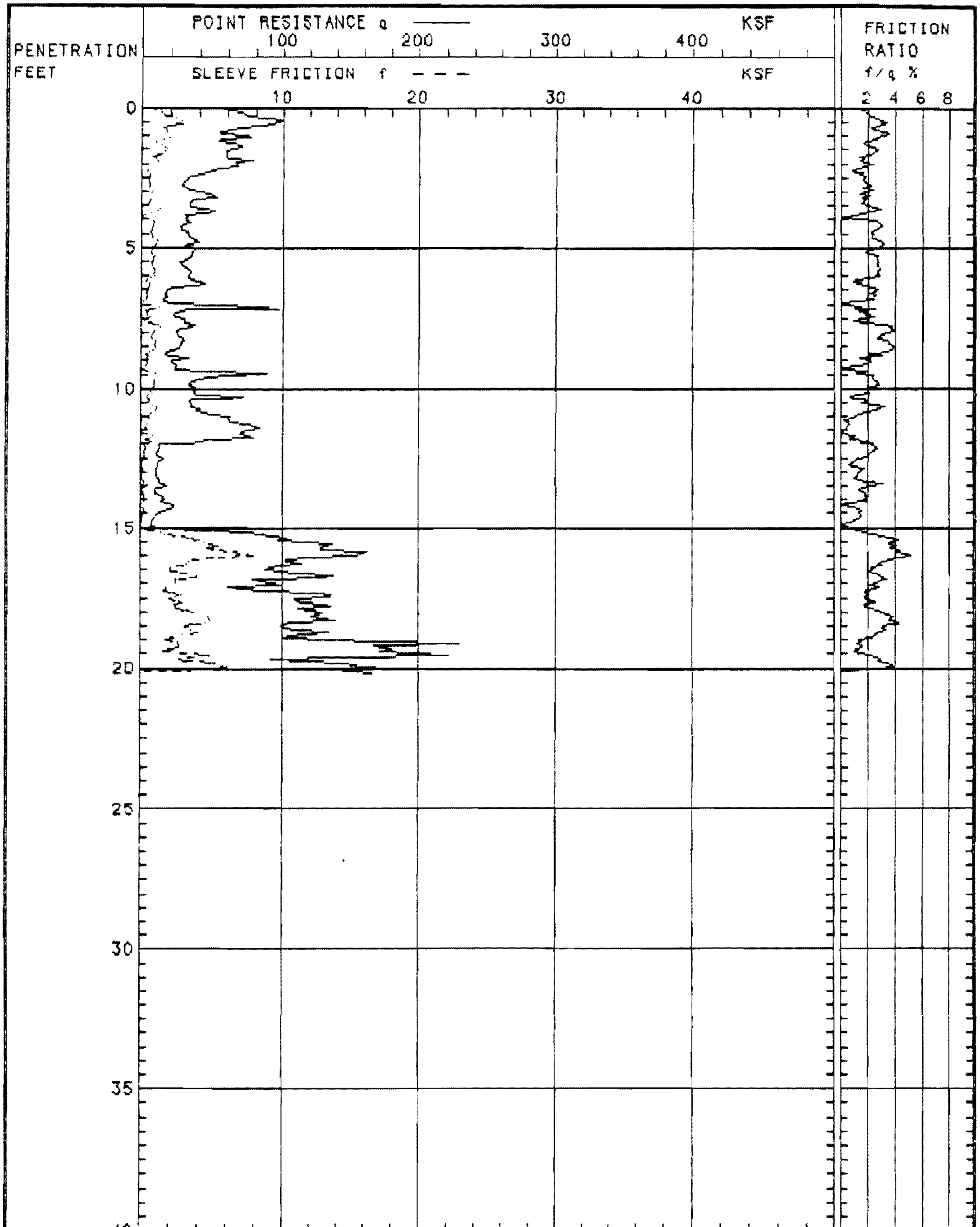
JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 146.1 FEET
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 7 STA. 1960+60: 10'N DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 6

JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 143.7 FEET
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 7 STA. 1985+60: 21'N DATE: 03-08-1990



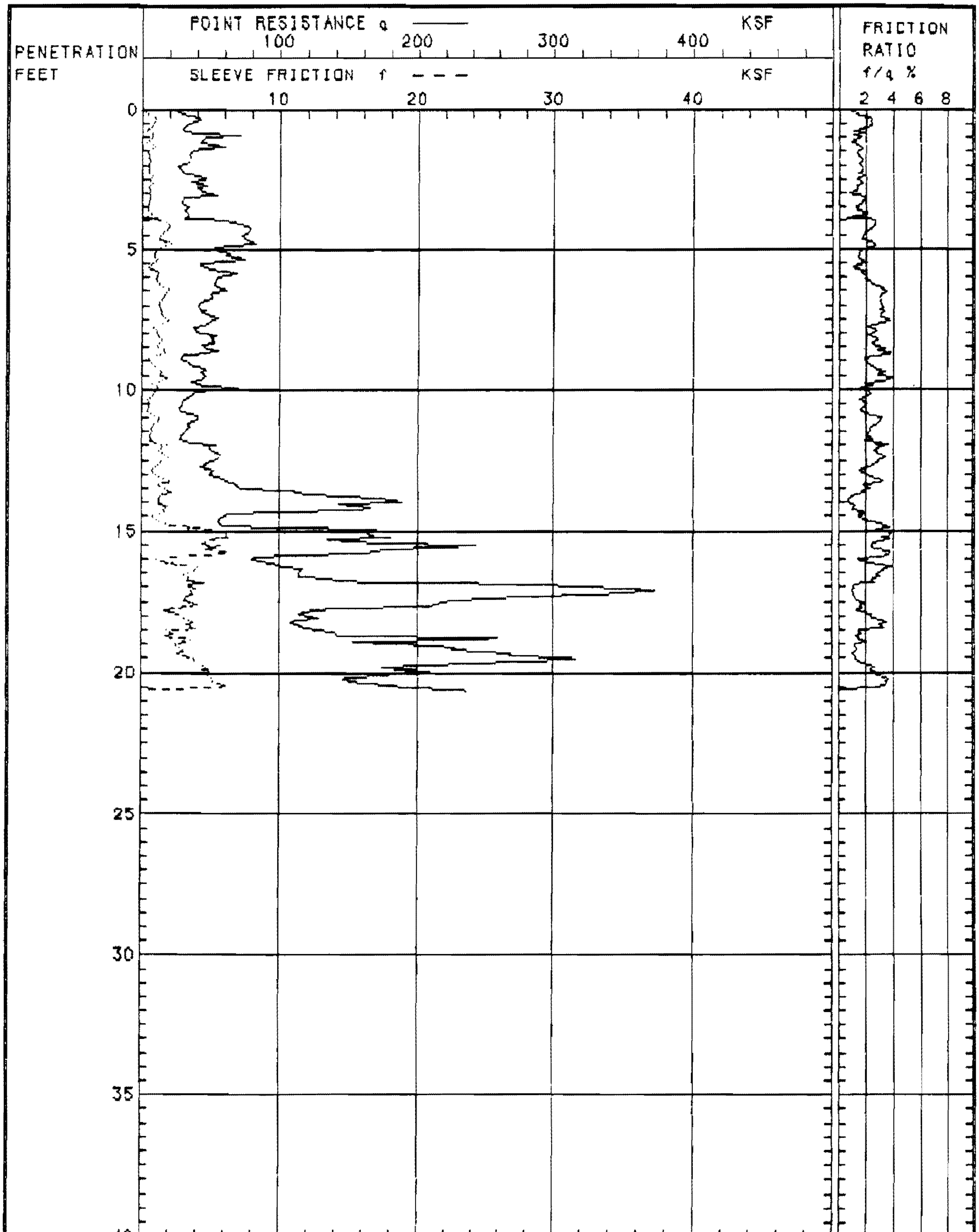
RESULT OF CONE PENETRATION TEST

CPT 7

JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 144.6 FEET

CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET

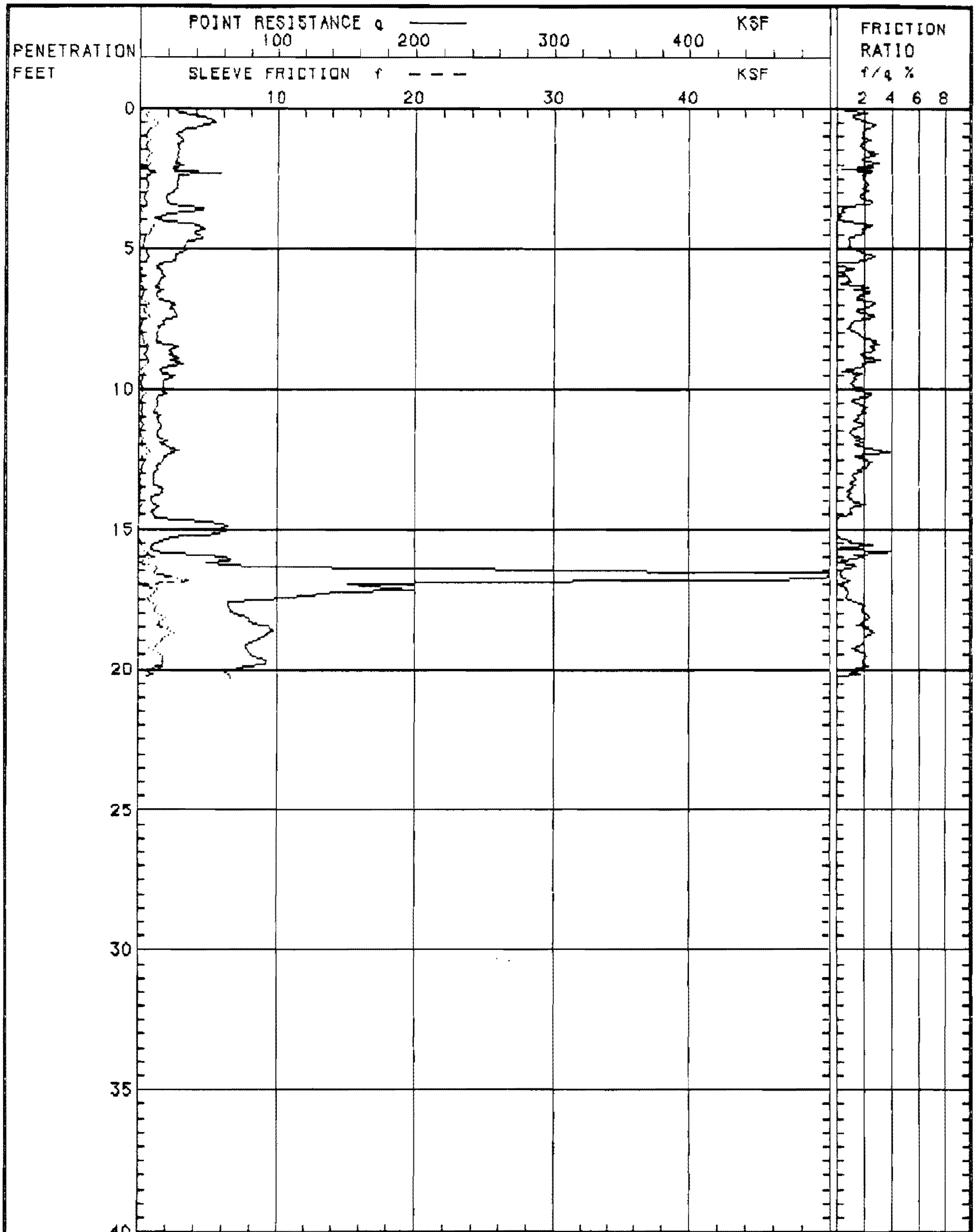
LOCATION: CYPRESS BYPASS WALL 8 STA. 1955+50: 15'S DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 8

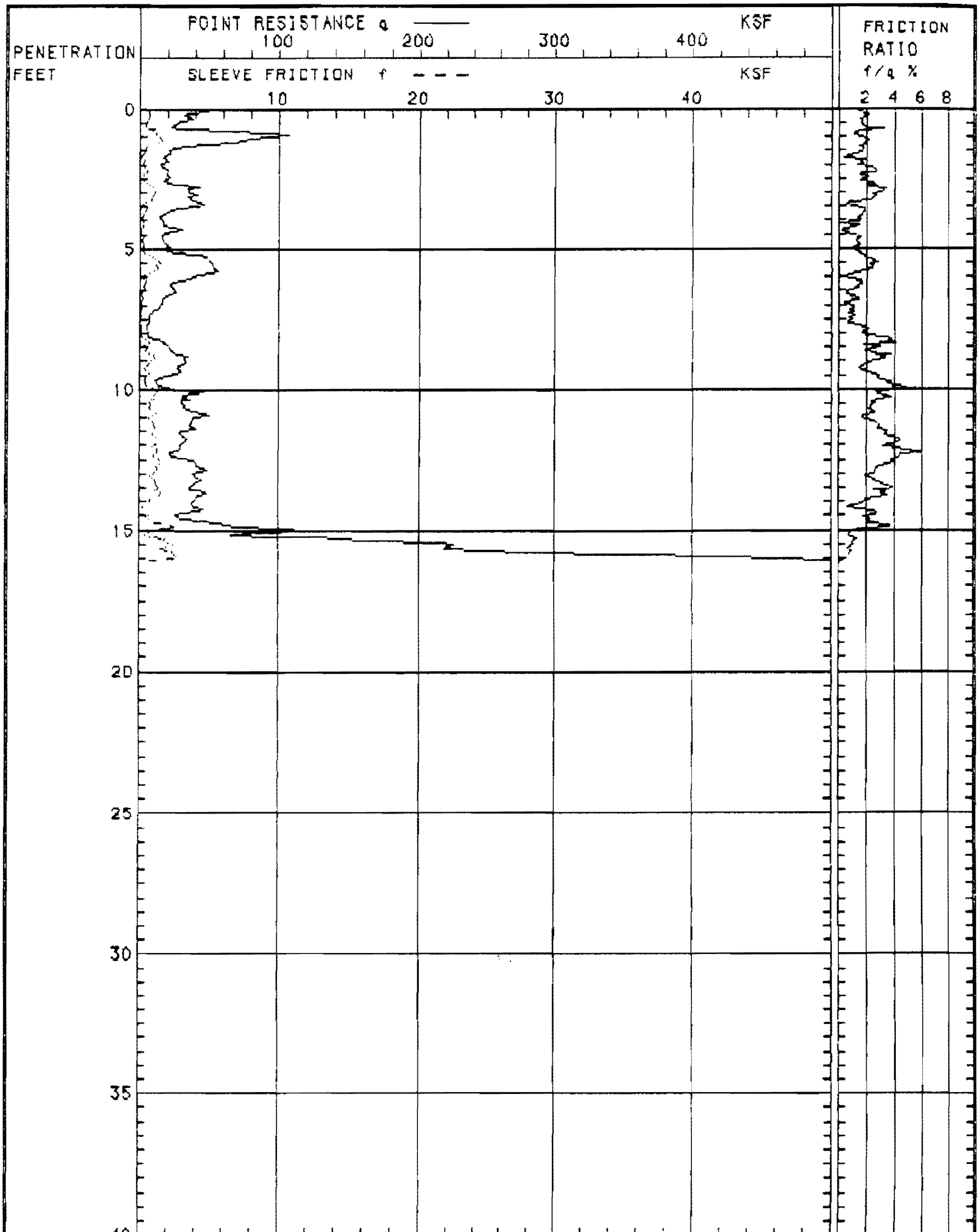
JOB NUMBER:	0601-0312	GROUND SURFACE ELEV.:	144.6 FEET
CONE NUMBER:	10	DEPTH TO GROUNDWATER:	0.0 FEET
LOCATION:	CYPRESS BYPASS WALL 8 STA. 1956+70: 18'S	DATE:	03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 9

JOB NUMBER:	0601-0312	GROUND SURFACE ELEV.:	143.7 FEET
CONE NUMBER:	10	DEPTH TO GROUNDWATER:	0.0 FEET
LOCATION:	CYPRESS BYPASS WALL 8 STA. 1963+00: 21'S	DATE:	03-08-1990



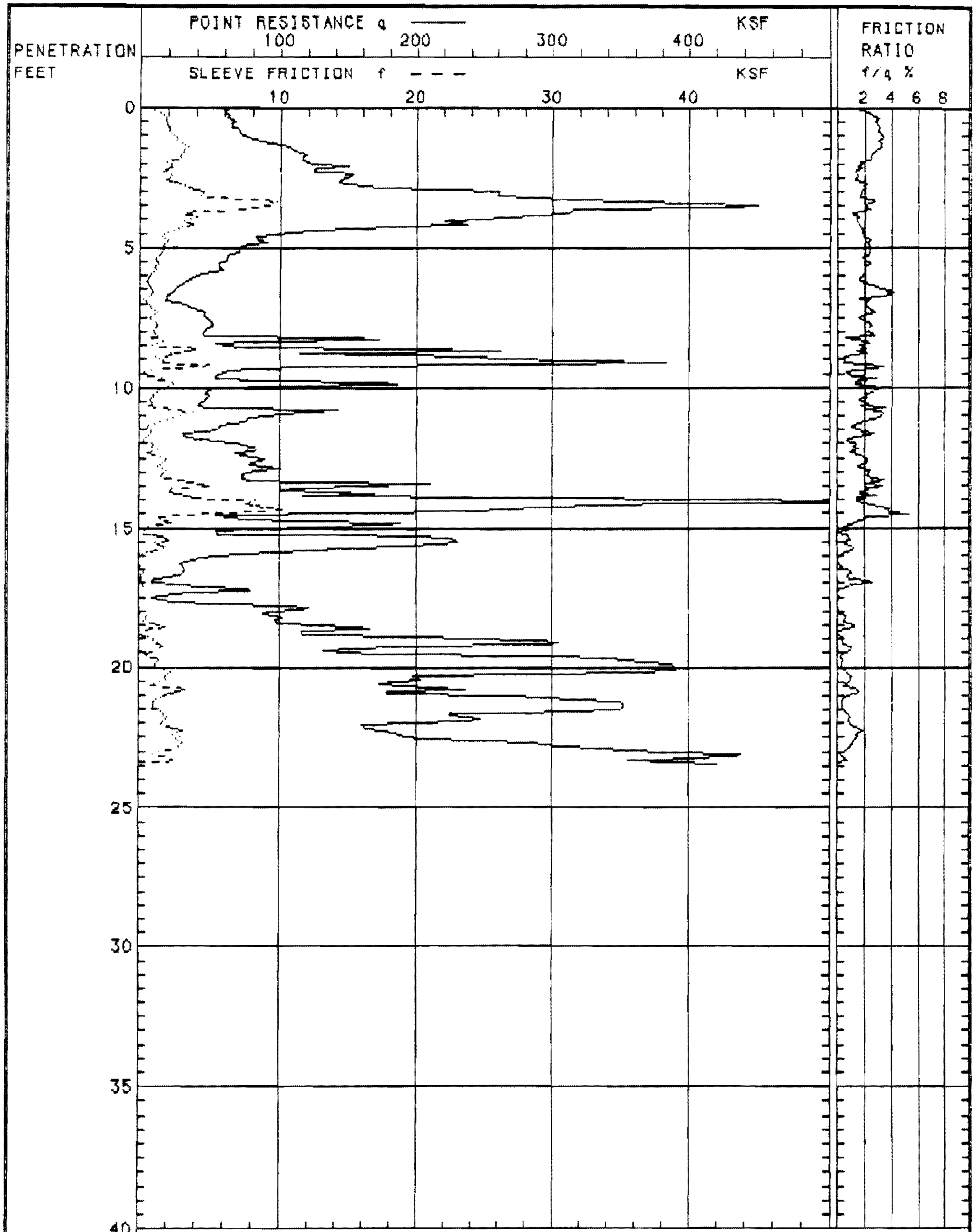
RESULT OF CONE PENETRATION TEST

CPT 10

JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 141.8 FEET

CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET

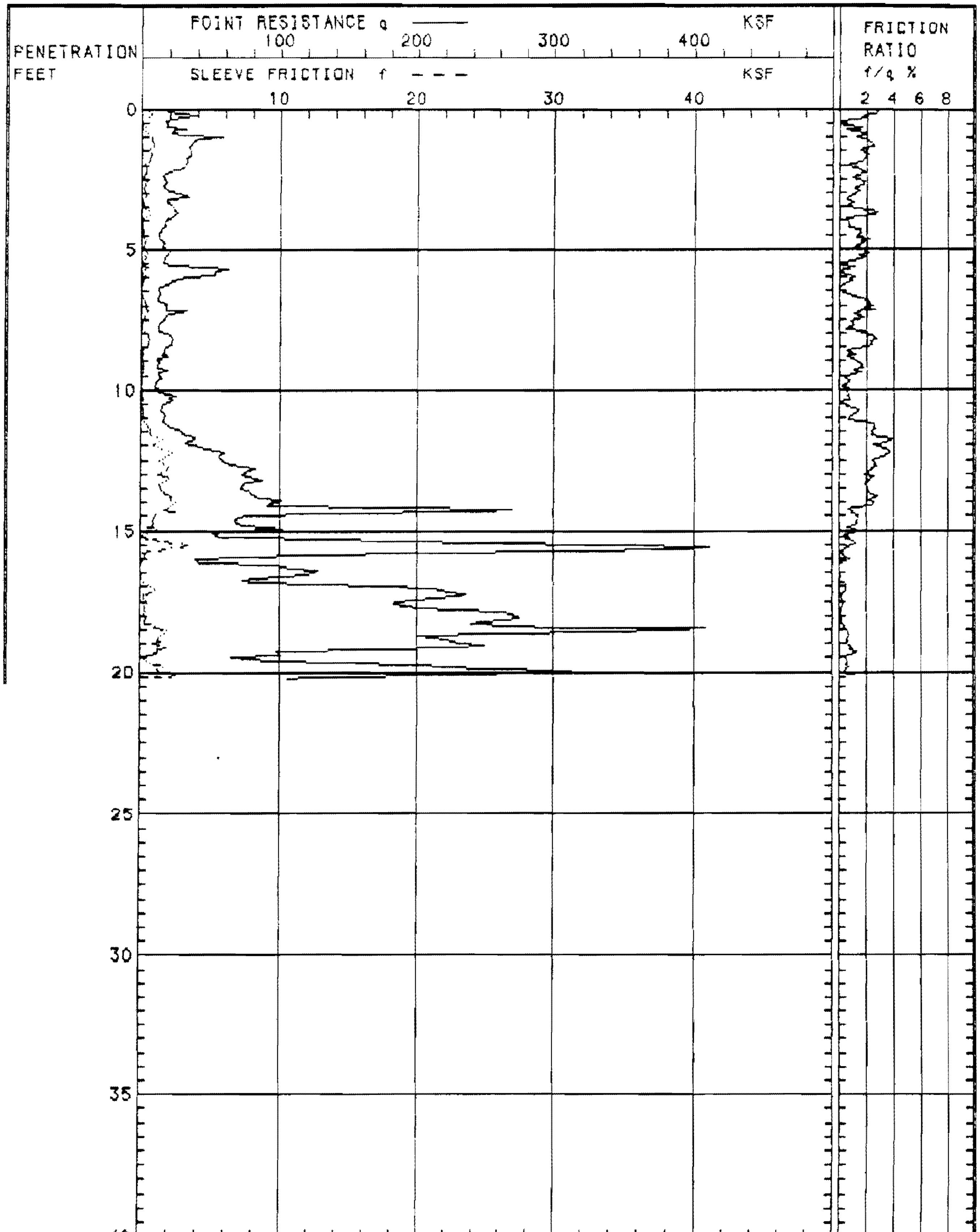
LOCATION: CYPRESS BYPASS WALL 9 STA. 1970+90 DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 11

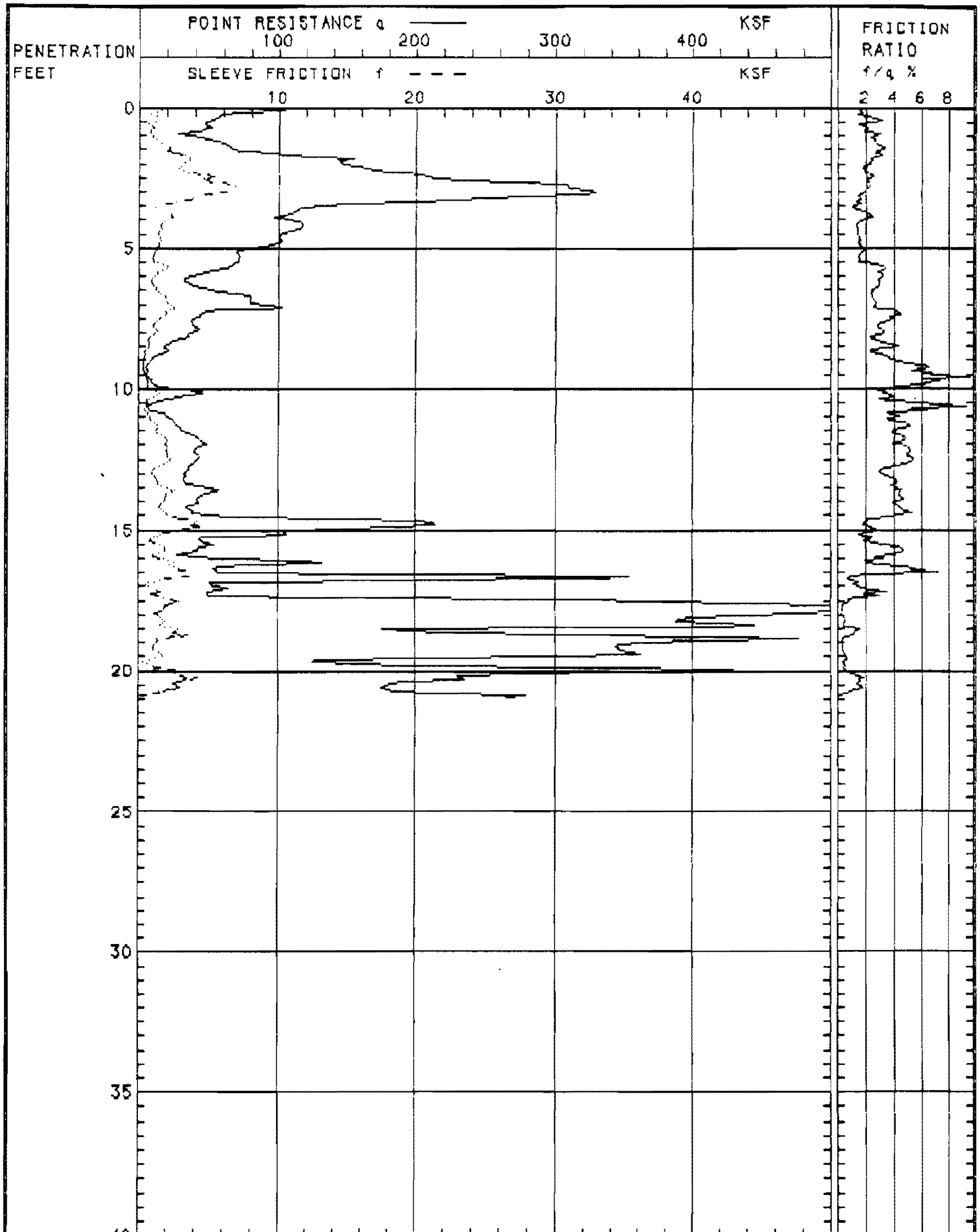
JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 141.8 FEET
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 9 STA. 1972+25 DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST

CPT 12

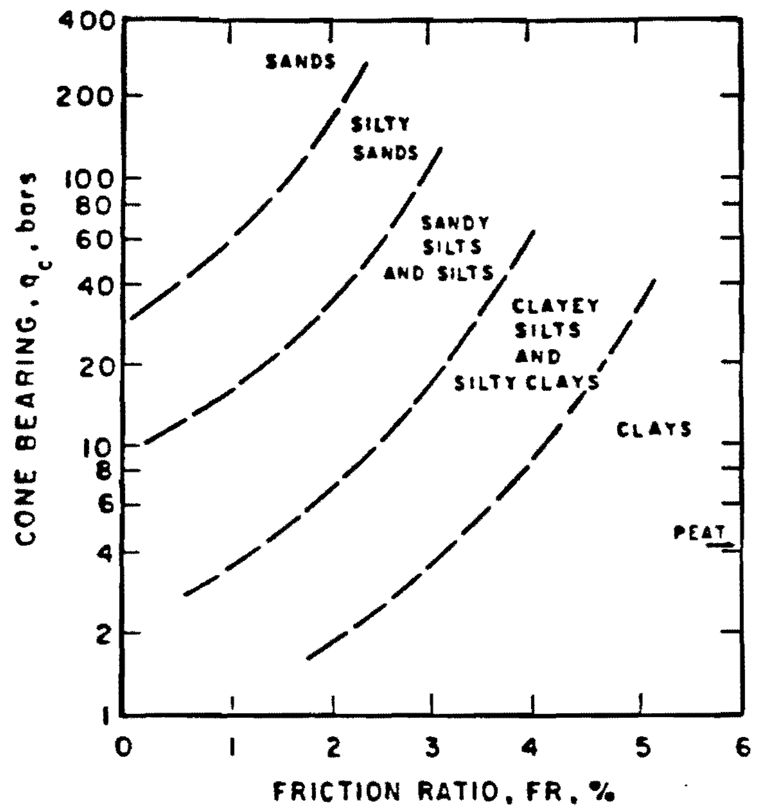
JOB NUMBER: 0601-0312
GROUND SURFACE ELEV.: 141.7 FEET
CONE NUMBER: 10
DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL 9 STA. 1974+25
DATE: 03-08-1990



RESULT OF CONE PENETRATION TEST
CPT 13

JOB NUMBER: 0601-0312 GROUND SURFACE ELEV.: 142.9 FEET
CONE NUMBER: 10 DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION: CYPRESS BYPASS WALL @ STA. 1970+00 DATE: 03-08-1990

INTERPRETATION OF CONE PENETROMETER DATA



Notes: 1) Cone penetrometer testing is a proven and widely accepted site investigation method that requires skill and judgement in the interpretation of the test results to provide a geotechnical characterization of the investigated soils and a development of their engineering properties.

2) Soil descriptions are based on correlations of tip resistance, q_c; and the friction ratio, FR, which is the ratio of the unit sleeve friction, f_s to q_c.

3) Correlations used for interpretation include: 1) Robertson, P.K., and Campanella, R.G., and 2) Schmertmann, J.H.

4) 1 bar is about 1 tsf.

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 1
CONE NO. : 10
DATE : 03/07/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	23.96	.00	.00	SANDY SILT TO CLAYEY SILT	ML-MH	.01	4	1.228	19.5	67	37	.50+
.40	16.09	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.01	3	0.825	19.5			
.60	6.85	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.02	1	0.350	19.5			
.80	6.16	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.02	1	0.315	19.5			
1.00	5.82	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.03	1	0.297	19.5			
1.20	4.79	.07	1.36	SENSITIVE FINE GRAINED	OL-CH	.04	1	0.244	19.5			
1.40	13.35	.14	1.03	SENSITIVE FINE GRAINED	OL-CH	.04	3	0.683	19.5			
1.60	8.56	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.05	2	0.436	19.5			
1.80	65.70	.99	1.50	SANDY SILT TO CLAYEY SILT	ML-MH	.09	12	3.365	19.5	91	41	.50+
2.00	55.80	1.26	2.25	SANDY SILT TO CLAYEY SILT	ML-MH	.10	10	2.856	19.5	82	40	.50+
2.20	77.51	1.83	2.36	SANDY SILT TO CLAYEY SILT	ML-MH	.11	14	3.969	19.5	97	42	.50+
2.40	99.62	2.11	2.12	SANDY SILT TO CLAYEY SILT	ML-MH	.12	19	5.103	19.5	100+	42	.50+
2.60	100.99	1.65	1.64	SILTY SAND TO SANDY SILT	SM-ML	.14	16			100+	42	.50+
2.80	77.74	1.31	1.68	SANDY SILT TO CLAYEY SILT	ML-MH	.14	14	3.979	19.5	93	41	.50+
3.00	79.42	1.17	1.47	SILTY SAND TO SANDY SILT	SM-ML	.17	12			91	41	.50+
3.20	97.27	1.71	1.76	SILTY SAND TO SANDY SILT	SM-ML	.18	15			100+	42	.50+
3.40	85.58	1.63	1.91	SANDY SILT TO CLAYEY SILT	ML-MH	.17	16	4.380	19.5	94	41	.50+
3.60	91.08	1.10	1.20	SILTY SAND TO SANDY SILT	SM-ML	.20	14			94	41	.50+
3.80	37.75	1.39	3.68	SILTY CLAY TO CLAY	CL-CH	.15	12	1.928	19.5			
4.00	44.50	.79	1.76	SANDY SILT TO CLAYEY SILT	ML-MH	.20	8	2.272	19.5	69	38	.50+
4.20	51.00	.88	1.72	SANDY SILT TO CLAYEY SILT	ML-MH	.21	9	2.605	19.5	72	38	.50+
4.40	52.04	.97	1.86	SANDY SILT TO CLAYEY SILT	ML-MH	.22	9	2.657	19.5	72	38	.50+
4.60	49.65	1.05	2.11	SANDY SILT TO CLAYEY SILT	ML-MH	.23	9	2.534	19.5	70	38	.50+
4.80	46.90	1.27	2.70	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	11	2.394	19.5			
5.00	43.81	1.64	3.75	SILTY CLAY TO CLAY	CL-CH	.20	13	2.236	19.5			
5.20	93.31	3.22	3.45	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	22	4.773	19.5			
5.40	38.69	1.30	3.37	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	9	1.972	19.5			
5.60	32.18	.97	3.01	CLAYEY SILT TO SILTY CLAY	MH-CL	.25	7	1.637	19.5			
5.80	32.53	.84	2.59	CLAYEY SILT TO SILTY CLAY	MH-CL	.26	7	1.655	19.5			
6.00	28.76	.77	2.67	CLAYEY SILT TO SILTY CLAY	MH-CL	.27	6	1.461	19.5			
6.20	23.28	.63	2.70	SILTY CLAY TO CLAY	CL-CH	.25	7	1.181	19.5			
6.40	18.15	.26	1.45	CLAYEY SILT TO SILTY CLAY	MH-CL	.29	4	0.916	19.5			
6.60	22.23	.50	2.26	CLAYEY SILT TO SILTY CLAY	MH-CL	.30	5	1.125	19.5			
6.80	23.97	.71	2.96	SILTY CLAY TO CLAY	CL-CH	.27	7	1.215	19.5			
7.00	34.89	1.23	3.52	SILTY CLAY TO CLAY	CL-CH	.28	11	1.775	19.5			
7.20	50.62	1.03	2.04	SANDY SILT TO CLAYEY SILT	ML-MH	.36	9	2.577	19.5	64	37	.32
7.40	42.82	1.01	2.37	CLAYEY SILT TO SILTY CLAY	MH-CL	.33	10	2.179	19.5			
7.60	44.16	1.47	3.33	CLAYEY SILT TO SILTY CLAY	MH-CL	.34	10	2.247	19.5			

7.80	47.97	.88	1.83	SANDY SILT TO CLAYEY SILT	ML-MH	.39	9	2.440	19.5	62	37	.25
8.00	43.31	.91	2.11	SANDY SILT TO CLAYEY SILT	ML-MH	.40	8	2.201	19.5	60	36	.21
8.20	45.53	1.01	2.22	SANDY SILT TO CLAYEY SILT	ML-MH	.41	8	2.314	19.5	60	37	.22
8.40	30.13	.73	2.43	CLAYEY SILT TO SILTY CLAY	MH-CL	.38	7	1.526	19.5			
8.60	28.08	.35	1.25	SANDY SILT TO CLAYEY SILT	ML-MH	.43	5	1.418	19.5	45	34	.15
8.80	23.64	.27	1.16	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	5	1.192	19.5			
9.00	11.65	.21	1.78	SILTY CLAY TO CLAY	CL-CH	.36	3	0.579	19.5			
9.20	7.70	.44	5.74	CLAY	CH	.37	3	0.376	19.5			
9.40	28.67	1.15	4.00	CLAY	CH	.38	13	1.451	19.5			
9.60	34.26	1.04	3.05	CLAYEY SILT TO SILTY CLAY	MH-CL	.43	8	1.735	19.5			
9.80	50.60	1.15	2.28	SANDY SILT TO CLAYEY SILT	ML-MH	.49	9	2.570	19.5	60	36	.22
10.00	47.08	1.89	4.01	SILTY CLAY TO CLAY	CL-CH	.40	15	2.394	19.5			
10.20	209.74	5.43	2.59	SILTY SAND TO SANDY SILT	SM-ML	.56	33			100+	42	.50+
10.40	115.11	2.86	2.48	SANDY SILT TO CLAYEY SILT	ML-MH	.52	22	5.876	19.5	83	40	.50+
10.60	112.20	2.51	2.24	SANDY SILT TO CLAYEY SILT	ML-MH	.53	21	5.727	19.5	82	40	.50+
10.80	226.82	3.25	1.43	SAND TO SILTY SAND	SM-SP	.65	27			100+	42	.50+
11.00	263.56	4.37	1.66	SAND TO SILTY SAND	SM-SP	.66	31			100+	42	.50+
11.20	83.62	2.00	2.39	SANDY SILT TO CLAYEY SILT	ML-MH	.56	16	4.259	19.5	71	38	.50+
11.40	64.32	8.18	12.71	CLAY	CH	.46	30	3.275	19.5			
11.60	609.02	8.93	1.47	SAND TO SILTY SAND	SM-SP	.70	72			100+	42	.50+
11.80	223.47	4.43	1.98	SILTY SAND TO SANDY SILT	SM-ML	.65	35			100+	42	.50+
12.00	44.61	1.88	4.21	SILTY CLAY TO CLAY	CL-CH	.48	14	2.263	19.5			
12.20	43.16	1.46	3.39	CLAYEY SILT TO SILTY CLAY	MH-CL	.55	10	2.185	19.5			
12.40	32.51	1.20	3.70	SILTY CLAY TO CLAY	CL-CH	.50	10	1.642	19.5			
12.60	50.32	2.00	3.98	SILTY CLAY TO CLAY	CL-CH	.50	16	2.555	19.5			
12.80	57.84	1.89	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.58	13	2.936	19.5			
13.00	80.51	2.15	2.67	SANDY SILT TO CLAYEY SILT	ML-MH	.65	15	4.095	19.5	67	37	.36
13.20	65.75	1.94	2.96	CLAYEY SILT TO SILTY CLAY	MH-CL	.59	15	3.342	19.5			
13.40	57.91	1.89	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.60	13	2.939	19.5			
13.60	201.02	2.60	1.29	SAND TO SILTY SAND	SM-SP	.82	24			92	42	.50+
13.80	166.93	4.07	2.44	SANDY SILT TO CLAYEY SILT	ML-MH	.69	31	8.525	19.5	90	41	.50+
14.00	79.45	2.39	3.01	SANDY SILT TO CLAYEY SILT	ML-MH	.70	15	4.038	19.5	65	37	.31
14.20	79.75	2.60	3.26	CLAYEY SILT TO SILTY CLAY	MH-CL	.64	19	4.057	19.5			
14.40	75.01	2.61	3.47	CLAYEY SILT TO SILTY CLAY	MH-CL	.65	17	3.813	19.5			
14.60	71.21	2.48	3.49	CLAYEY SILT TO SILTY CLAY	MH-CL	.66	17	3.618	19.5			
14.80	78.01	2.74	3.51	CLAYEY SILT TO SILTY CLAY	MH-CL	.67	18	3.966	19.5			
15.00	88.38	2.48	2.80	SANDY SILT TO CLAYEY SILT	ML-MH	.75	16	4.494	19.5	66	37	.36
15.20	78.01	2.55	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.68	18	3.966	19.5			
15.40	84.22	3.06	3.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.69	20	4.284	19.5			
15.60	138.61	3.18	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.78	26	7.068	19.5	81	39	.50+
15.80	267.26	2.91	1.09	SAND TO SILTY SAND	SM-SP	.95	32			100+	42	.50+
16.00	66.88	2.11	3.15	CLAYEY SILT TO SILTY CLAY	MH-CL	.72	16	3.393	19.5			

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 2
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	17.12	.09	.52	SENSITIVE FINE GRAINED	OL-CH	.01	4	0.877	19.5			
.40	20.54	.05	.23	SANDY SILT TO CLAYEY SILT	ML-MH	.02	3	1.052	19.5	64	37	.50+
.60	13.35	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.02	3	0.684	19.5			
.80	17.12	.19	1.14	CLAYEY SILT TO SILTY CLAY	MH-CL	.04	4	0.876	19.5			
1.00	18.49	.09	.51	SENSITIVE FINE GRAINED	OL-CH	.03	4	0.947	19.5			
1.20	12.32	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.04	2	0.630	19.5			
1.40	9.93	.03	.33	SENSITIVE FINE GRAINED	OL-CH	.04	2	0.507	19.5			
1.60	7.36	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.05	1	0.375	19.5			
1.80	7.19	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.05	1	0.366	19.5			
2.00	11.98	.08	.64	SENSITIVE FINE GRAINED	OL-CH	.06	2	0.611	19.5			
2.20	16.78	.25	1.50	CLAYEY SILT TO SILTY CLAY	MH-CL	.10	4	0.855	19.5			
2.40	16.77	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.07	4	0.856	19.5			
2.60	53.36	.66	1.24	SANDY SILT TO CLAYEY SILT	ML-MH	.13	10	2.730	19.5	78	39	.50+
2.80	12.84	.09	.68	SENSITIVE FINE GRAINED	OL-CH	.08	3	0.654	19.5			
3.00	9.41	.04	.43	SENSITIVE FINE GRAINED	OL-CH	.09	2	0.478	19.5			
3.20	10.96	.01	.07	SENSITIVE FINE GRAINED	OL-CH	.10	2	0.557	19.5			
3.40	10.27	.03	.31	SENSITIVE FINE GRAINED	OL-CH	.10	2	0.522	19.5			
3.60	13.35	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.11	3	0.679	19.5			
3.80	12.32	.05	.40	SENSITIVE FINE GRAINED	OL-CH	.11	2	0.626	19.5			
4.00	10.96	.08	.74	SENSITIVE FINE GRAINED	OL-CH	.12	2	0.556	19.5			
4.20	10.96	.06	.51	SENSITIVE FINE GRAINED	OL-CH	.13	2	0.555	19.5			
4.40	13.00	.10	.78	SENSITIVE FINE GRAINED	OL-CH	.13	3	0.660	19.5			
4.60	13.78	.18	1.34	SENSITIVE FINE GRAINED	OL-CH	.14	3	0.699	19.5			
4.80	10.96	.17	1.53	SENSITIVE FINE GRAINED	OL-CH	.14	2	0.555	19.5			
5.00	11.30	.22	1.97	SILTY CLAY TO CLAY	CL-CH	.20	3	0.569	19.5			
5.20	10.61	.22	2.03	SILTY CLAY TO CLAY	CL-CH	.21	3	0.533	19.5			
5.40	10.62	.06	.56	SENSITIVE FINE GRAINED	OL-CH	.16	2	0.536	19.5			
5.60	11.64	.06	.49	SENSITIVE FINE GRAINED	OL-CH	.17	2	0.588	19.5			
5.80	13.69	.13	.92	SENSITIVE FINE GRAINED	OL-CH	.17	3	0.693	19.5			
6.00	16.43	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.18	3	0.833	19.5			
6.20	16.45	.11	.68	SENSITIVE FINE GRAINED	OL-CH	.19	3	0.834	19.5			
6.40	706.66	.00	.00	GRAVELLY SAND TO SAND	SW-GW	.42	56			100+	46	.50+

Notes:

. Default values of submerged unit weight and cone factor are used.

2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 3
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	24.65	.40	1.64	CLAYEY SILT TO SILTY CLAY	MH-CL	.01	5	1.264	19.5			
.40	27.39	.47	1.72	CLAYEY SILT TO SILTY CLAY	MH-CL	.02	6	1.404	19.5			
.60	44.84	.99	2.22	SANDY SILT TO CLAYEY SILT	ML-MH	.03	8	2.298	19.5	82	39	.50+
.80	42.79	1.05	2.45	CLAYEY SILT TO SILTY CLAY	MH-CL	.04	10	2.192	19.5			
1.00	43.82	.95	2.18	SANDY SILT TO CLAYEY SILT	ML-MH	.05	8	2.245	19.5	78	39	.50+
1.20	48.96	1.02	2.09	SANDY SILT TO CLAYEY SILT	ML-MH	.06	9	2.508	19.5	81	39	.50+
1.40	44.85	.92	2.04	SANDY SILT TO CLAYEY SILT	ML-MH	.07	8	2.296	19.5	77	39	.50+
1.60	54.09	1.12	2.06	SANDY SILT TO CLAYEY SILT	ML-MH	.08	10	2.770	19.5	83	40	.50+
1.80	57.52	1.08	1.89	SANDY SILT TO CLAYEY SILT	ML-MH	.09	11	2.945	19.5	85	40	.50+
2.00	61.62	.47	.77	SILTY SAND TO SANDY SILT	SM-ML	.11	9			86	40	.50+
2.20	150.38	2.92	1.94	SILTY SAND TO SANDY SILT	SM-ML	.12	24			100+	42	.50+
2.40	83.27	2.61	3.14	CLAYEY SILT TO SILTY CLAY	MH-CL	.11	19	4.265	19.5			
2.60	74.98	1.21	1.61	SANDY SILT TO CLAYEY SILT	ML-MH	.13	14	3.838	19.5	92	41	.50+
2.80	74.97	1.39	1.85	SANDY SILT TO CLAYEY SILT	ML-MH	.14	14	3.837	19.5	91	41	.50+
3.00	78.39	1.18	1.51	SILTY SAND TO SANDY SILT	SM-ML	.17	12			90	41	.50+
3.20	85.58	1.04	1.22	SILTY SAND TO SANDY SILT	SM-ML	.18	13			93	41	.50+
3.40	95.13	1.28	1.35	SILTY SAND TO SANDY SILT	SM-ML	.19	15			97	42	.50+
3.60	123.92	2.17	1.76	SILTY SAND TO SANDY SILT	SM-ML	.20	19			100+	42	.50+
3.80	134.01	3.85	2.88	SANDY SILT TO CLAYEY SILT	ML-MH	.19	25	6.863	19.5	100+	42	.50+
4.00	142.08	3.85	2.71	SANDY SILT TO CLAYEY SILT	ML-MH	.20	27	7.276	19.5	100+	42	.50+
4.20	140.36	3.24	2.31	SANDY SILT TO CLAYEY SILT	ML-MH	.21	26	7.187	19.5	100+	42	.50+
4.40	141.91	2.88	2.03	SILTY SAND TO SANDY SILT	SM-ML	.24	22			100+	42	.50+
4.60	188.19	3.12	1.66	SILTY SAND TO SANDY SILT	SM-ML	.25	30			100+	42	.50+
4.80	183.97	4.74	2.58	SANDY SILT TO CLAYEY SILT	ML-MH	.24	35	9.422	19.5	100+	42	.50+
5.00	160.32	4.36	2.72	SANDY SILT TO CLAYEY SILT	ML-MH	.25	30	8.209	19.5	100+	42	.50+
5.20	169.09	4.07	2.41	SILTY SAND TO SANDY SILT	SM-ML	.29	26			100+	42	.50+
5.40	163.29	3.82	2.34	SILTY SAND TO SANDY SILT	SM-ML	.30	26			100+	42	.50+
5.60	155.27	2.86	1.84	SILTY SAND TO SANDY SILT	SM-ML	.31	24			100+	42	.50+
5.80	105.47	1.99	1.89	SILTY SAND TO SANDY SILT	SM-ML	.32	16			90	41	.50+
6.00	102.34	1.95	1.91	SILTY SAND TO SANDY SILT	SM-ML	.33	16			88	40	.50+
6.20	95.86	2.05	2.14	SANDY SILT TO CLAYEY SILT	ML-MH	.31	18	4.900	19.5	87	40	.50+
6.40	100.40	1.22	1.22	SILTY SAND TO SANDY SILT	SM-ML	.35	16			86	40	.50+
6.60	99.33	1.83	1.84	SILTY SAND TO SANDY SILT	SM-ML	.36	15			85	40	.50+
6.80	86.96	1.24	1.43	SILTY SAND TO SANDY SILT	SM-ML	.37	13			80	39	.50+
7.00	79.50	.69	.86	SILTY SAND TO SANDY SILT	SM-ML	.38	12			76	39	.50+
7.20	79.07	1.28	1.62	SILTY SAND TO SANDY SILT	SM-ML	.40	12			76	39	.50+
7.40	71.38	1.23	1.72	SANDY SILT TO CLAYEY SILT	ML-MH	.37	13	3.642	19.5	73	38	.50+
7.60	106.71	1.42	1.33	SILTY SAND TO SANDY SILT	SM-ML	.42	17			85	40	.50+

7.80	125.43	.89	.71	SAND TO SILTY SAND	SM-SP	.47	15			88	41	.50+
8.00	69.69	.00	.00	SAND TO SILTY SAND	SM-SP	.48	8			68	39	.45
8.20	64.06	.54	.84	SILTY SAND TO SANDY SILT	SM-ML	.45	10			67	37	.39
8.40	60.94	.95	1.56	SANDY SILT TO CLAYEY SILT	ML-MH	.42	11	3.104	19.5	67	37	.39
8.60	64.68	1.11	1.72	SANDY SILT TO CLAYEY SILT	ML-MH	.43	12	3.295	19.5	68	38	.45
8.80	52.06	.86	1.65	SANDY SILT TO CLAYEY SILT	ML-MH	.44	9	2.647	19.5	62	37	.25
9.00	49.31	.95	1.92	SANDY SILT TO CLAYEY SILT	ML-MH	.45	9	2.506	19.5	61	37	.23
9.20	56.68	.79	1.40	SANDY SILT TO CLAYEY SILT	ML-MH	.46	10	2.883	19.5	63	37	.28
9.40	94.32	.73	.77	SILTY SAND TO SANDY SILT	SM-ML	.52	15			76	39	.50+
9.60	60.43	.14	.22	SILTY SAND TO SANDY SILT	SM-ML	.53	9			63	37	.26
9.80	54.77	.42	.77	SILTY SAND TO SANDY SILT	SM-ML	.54	8			60	37	.22
10.00	57.17	1.08	1.90	SANDY SILT TO CLAYEY SILT	ML-MH	.50	10	2.906	19.5	62	37	.25
10.20	64.43	.70	1.09	SILTY SAND TO SANDY SILT	SM-ML	.56	10			63	37	.27
10.40	134.86	1.53	1.14	SILTY SAND TO SANDY SILT	SM-ML	.57	21			86	40	.50+
10.60	175.78	4.76	2.71	SANDY SILT TO CLAYEY SILT	ML-MH	.53	33	8.987	19.5	99	42	.50+
10.80	119.24	3.82	3.20	SANDY SILT TO CLAYEY SILT	ML-MH	.54	22	6.087	19.5	83	40	.50+
11.00	98.21	1.77	1.81	SILTY SAND TO SANDY SILT	SM-ML	.60	15			74	38	.50+
11.20	73.69	1.63	2.21	SANDY SILT TO CLAYEY SILT	ML-MH	.56	14	3.750	19.5	67	37	.38
11.40	54.79	.66	1.20	SANDY SILT TO CLAYEY SILT	ML-MH	.57	10	2.781	19.5	59	36	.21
11.60	32.59	.65	2.01	CLAYEY SILT TO SILTY CLAY	MH-CL	.52	7	1.645	19.5			
11.80	27.19	.14	.53	SANDY SILT TO CLAYEY SILT	ML-MH	.59	5	1.364	19.5	36	33	.13
12.00	67.16	1.04	1.55	SANDY SILT TO CLAYEY SILT	ML-MH	.60	12	3.413	19.5	63	37	.27
12.20	109.30	3.22	2.95	SANDY SILT TO CLAYEY SILT	ML-MH	.61	20	5.574	19.5	78	39	.50+
12.40	126.05	2.68	2.13	SANDY SILT TO CLAYEY SILT	ML-MH	.62	24	6.432	19.5	83	40	.50+
12.60	86.87	.46	.53	SAND TO SILTY SAND	SM-SP	.76	10			66	39	.34
12.80	114.89	.56	.49	SAND TO SILTY SAND	SM-SP	.77	13			75	40	.50+
13.00	144.31	.87	.60	SAND TO SILTY SAND	SM-SP	.78	17			82	40	.50+
13.20	131.45	.91	.69	SAND TO SILTY SAND	SM-SP	.79	15			79	40	.50+
13.40	113.78	.63	.56	SAND TO SILTY SAND	SM-SP	.80	13			73	39	.50+
13.60	77.80	.01	.01	SAND TO SILTY SAND	SM-SP	.82	9			61	38	.24
13.80	66.18	.41	.62	SILTY SAND TO SANDY SILT	SM-ML	.76	10			58	36	.21
14.00	39.54	.06	.16	SILTY SAND TO SANDY SILT	SM-ML	.77	6			41	34	.15
14.20	77.48	.52	.68	SILTY SAND TO SANDY SILT	SM-ML	.78	12			62	37	.25
14.40	48.48	.12	.25	SILTY SAND TO SANDY SILT	SM-ML	.79	7			46	34	.16
14.60	25.57	.11	.44	SANDY SILT TO CLAYEY SILT	ML-MH	.73	4	1.274	19.5	27	32	.12
14.80	14.07	.19	1.36	CLAYEY SILT TO SILTY CLAY	MH-CL	.67	3	0.687	19.5			
15.00	6.68	.01	.10	SENSITIVE FINE GRAINED	OL-CH	.45	1	0.319	19.5			
15.20	20.13	.00	.01	SENSITIVE FINE GRAINED	OL-CH	.46	4	1.009	19.5			
15.40	60.87	.36	.59	SILTY SAND TO SANDY SILT	SM-ML	.85	9			52	35	.18
15.60	63.96	.47	.73	SILTY SAND TO SANDY SILT	SM-ML	.86	10			53	36	.18
15.80	64.74	.72	1.11	SILTY SAND TO SANDY SILT	SM-ML	.87	10			53	36	.19
16.00	126.04	1.67	1.32	SILTY SAND TO SANDY SILT	SM-ML	.88	20			75	39	.50+
16.20	70.35	1.07	1.52	SANDY SILT TO CLAYEY SILT	ML-MH	.81	13	3.566	19.5	59	36	.21
16.40	56.83	.51	.90	SILTY SAND TO SANDY SILT	SM-ML	.90	9			48	35	.17
16.60	117.58	.88	.75	SAND TO SILTY SAND	SM-SP	1.00	14			69	39	.47
16.80	135.04	1.26	.93	SAND TO SILTY SAND	SM-SP	1.01	16			74	39	.50+
17.00	95.16	.71	.75	SILTY SAND TO SANDY SILT	SM-ML	.93	15			64	37	.29
17.20	47.89	.48	1.01	SANDY SILT TO CLAYEY SILT	ML-MH	.86	9	2.412	19.5	43	34	.15
17.40	88.75	1.40	1.57	SILTY SAND TO SANDY SILT	SM-ML	.96	14			62	37	.25
17.60	91.99	1.05	1.15	SILTY SAND TO SANDY SILT	SM-ML	.97	14			62	37	.26
17.80	110.14	2.04	1.85	SILTY SAND TO SANDY SILT	SM-ML	.98	17			67	38	.39
18.00	210.52	4.54	2.16	SILTY SAND TO SANDY SILT	SM-ML	.99	33			89	41	.50+

18.20	120.31	3.52	2.93	SANDY SILT TO CLAYEY SILT	ML-MH	.91	23	6.123	19.5	72	38	.50+
18.40	77.45	1.66	2.15	SANDY SILT TO CLAYEY SILT	ML-MH	.92	14	3.925	19.5	59	36	.21
18.60	96.65	.38	.39	SAND TO SILTY SAND	SM-SP	1.12	11			61	38	.24
18.80	92.70	.93	1.01	SILTY SAND TO SANDY SILT	SM-ML	1.03	14			61	37	.24
19.00	96.14	.90	.94	SILTY SAND TO SANDY SILT	SM-ML	1.04	15			62	37	.25
19.20	79.55	1.22	1.53	SILTY SAND TO SANDY SILT	SM-ML	1.06	12			55	36	.20
19.40	68.47	.93	1.37	SILTY SAND TO SANDY SILT	SM-ML	1.07	10			49	35	.17
19.60	59.93	.25	.41	SILTY SAND TO SANDY SILT	SM-ML	1.08	9			45	34	.16
19.80	48.69	.08	.16	SILTY SAND TO SANDY SILT	SM-ML	1.09	7			38	33	.14

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 4
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	41.94	.13	.30	SILTY SAND TO SANDY SILT	SM-ML	.01	6			81	39	.50+
.40	31.50	.00	.00	SANDY SILT TO CLAYEY SILT	ML-MH	.02	6	1.614	19.5	72	38	.50+
.60	260.48	.00	.00	SAND	SW-SP	.04	24			100+	44	.50+
.80	46.31	1.30	2.80	CLAYEY SILT TO SILTY CLAY	MH-CL	.04	11	2.373	19.5			
1.00	54.09	.69	1.27	SANDY SILT TO CLAYEY SILT	ML-MH	.05	10	2.771	19.5	88	40	.50+
1.20	43.48	.66	1.51	SANDY SILT TO CLAYEY SILT	ML-MH	.06	8	2.227	19.5	77	39	.50+
1.40	95.15	1.04	1.10	SILTY SAND TO SANDY SILT	SM-ML	.08	15			100+	42	.50+
1.60	116.72	1.16	1.00	SILTY SAND TO SANDY SILT	SM-ML	.09	18			100+	42	.50+
1.80	139.54	1.47	1.05	SAND TO SILTY SAND	SM-SP	.11	16			100+	42	.50+
2.00	125.95	2.66	2.11	SILTY SAND TO SANDY SILT	SM-ML	.11	20			100+	42	.50+
2.20	139.35	2.56	1.84	SILTY SAND TO SANDY SILT	SM-ML	.12	22			100+	42	.50+
2.40	99.63	1.36	1.37	SILTY SAND TO SANDY SILT	SM-ML	.13	15			100+	42	.50+
2.60	126.08	2.49	1.98	SILTY SAND TO SANDY SILT	SM-ML	.14	20			100+	42	.50+
2.80	81.57	1.67	2.05	SANDY SILT TO CLAYEY SILT	ML-MH	.14	15	4.176	19.5	95	41	.50+
3.00	44.17	.63	1.42	SANDY SILT TO CLAYEY SILT	ML-MH	.15	8	2.257	19.5	71	38	.50+
3.20	47.75	1.00	2.09	SANDY SILT TO CLAYEY SILT	ML-MH	.16	9	2.441	19.5	73	38	.50+
3.40	65.04	1.43	2.21	SANDY SILT TO CLAYEY SILT	ML-MH	.17	12	3.327	19.5	82	40	.50+
3.60	77.04	1.83	2.38	SANDY SILT TO CLAYEY SILT	ML-MH	.18	14	3.942	19.5	88	40	.50+
3.80	70.56	1.80	2.55	SANDY SILT TO CLAYEY SILT	ML-MH	.19	13	3.609	19.5	84	40	.50+
4.00	63.33	.83	1.32	SANDY SILT TO CLAYEY SILT	ML-MH	.20	12	3.237	19.5	79	39	.50+
4.20	50.64	1.13	2.23	SANDY SILT TO CLAYEY SILT	ML-MH	.21	9	2.586	19.5	71	38	.50+
4.40	69.80	1.43	2.06	SANDY SILT TO CLAYEY SILT	ML-MH	.22	13	3.568	19.5	81	39	.50+
4.60	64.70	1.46	2.26	SANDY SILT TO CLAYEY SILT	ML-MH	.23	12	3.306	19.5	78	39	.50+
4.80	68.12	1.48	2.17	SANDY SILT TO CLAYEY SILT	ML-MH	.24	13	3.481	19.5	79	39	.50+
5.00	57.86	1.25	2.16	SANDY SILT TO CLAYEY SILT	ML-MH	.25	11	2.954	19.5	73	38	.50+
5.20	34.60	.78	2.25	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	8	1.763	19.5			
5.40	23.44	.23	.97	SANDY SILT TO CLAYEY SILT	ML-MH	.27	4	1.188	19.5	50	35	.16
5.60	32.85	.53	1.62	SANDY SILT TO CLAYEY SILT	ML-MH	.28	6	1.670	19.5	58	36	.21
5.80	53.07	1.64	3.09	CLAYEY SILT TO SILTY CLAY	MH-CL	.26	12	2.708	19.5			
6.00	54.77	1.39	2.53	SANDY SILT TO CLAYEY SILT	ML-MH	.30	10	2.793	19.5	69	38	.50+
6.20	83.79	.71	.85	SILTY SAND TO SANDY SILT	SM-ML	.34	13			80	39	.50+
6.40	129.14	1.31	1.01	SAND TO SILTY SAND	SM-SP	.38	15			94	42	.50+
6.60	53.38	.91	1.70	SANDY SILT TO CLAYEY SILT	ML-MH	.33	10	2.721	19.5	67	37	.44
6.80	119.51	1.73	1.45	SILTY SAND TO SANDY SILT	SM-ML	.37	19			92	41	.50+
7.00	39.53	1.29	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	9	2.011	19.5			
7.20	18.51	.46	2.49	SILTY CLAY TO CLAY	CL-CH	.29	5	0.934	19.5			
7.40	22.23	.69	3.10	SILTY CLAY TO CLAY	CL-CH	.30	7	1.125	19.5			
7.60	102.41	2.03	1.98	SANDY SILT TO CLAYEY SILT	ML-MH	.38	19	5.232	19.5	85	40	.50+

7.80	143.65	2.44	1.70	SILTY SAND TO SANDY SILT	SM-ML	.43	22			96	42	.50+
8.00	49.31	1.02	2.07	SANDY SILT TO CLAYEY SILT	ML-MH	.40	9	2.508	19.5	62	37	.26
8.20	52.37	1.00	1.91	SANDY SILT TO CLAYEY SILT	ML-MH	.41	10	2.665	19.5	63	37	.28
8.40	158.69	2.89	1.82	SILTY SAND TO SANDY SILT	SM-ML	.46	25			99	42	.50+
8.60	107.11	2.13	1.99	SANDY SILT TO CLAYEY SILT	ML-MH	.43	20	5.471	19.5	84	40	.50+
8.80	139.74	3.70	2.65	SANDY SILT TO CLAYEY SILT	ML-MH	.44	26	7.144	19.5	94	41	.50+
9.00	73.61	.01	.01	SAND TO SILTY SAND	SM-SP	.54	8			68	39	.41
9.20	216.88	3.73	1.72	SILTY SAND TO SANDY SILT	SM-ML	.51	34			100+	42	.50+
9.40	104.61	1.25	1.19	SILTY SAND TO SANDY SILT	SM-ML	.52	16			80	39	.50+
9.60	124.45	1.35	1.08	SILTY SAND TO SANDY SILT	SM-ML	.53	19			85	40	.50+
9.80	150.04	1.56	1.04	SAND TO SILTY SAND	SM-SP	.59	17			90	41	.50+
10.00	205.91	3.25	1.58	SILTY SAND TO SANDY SILT	SM-ML	.55	32			100+	42	.50+
10.20	66.53	2.78	4.17	SILTY CLAY TO CLAY	CL-CH	.41	21	3.391	19.5			
10.40	117.21	5.76	4.92	SILTY CLAY TO CLAY	CL-CH	.42	37	5.989	19.5			
10.60	473.65	8.11	1.71	SAND TO SILTY SAND	SM-SP	.64	56			100+	42	.50+
10.80	155.18	2.94	1.89	SILTY SAND TO SANDY SILT	SM-ML	.59	24			91	41	.50+
11.00	69.47	1.26	1.81	SANDY SILT TO CLAYEY SILT	ML-MH	.55	13	3.534	19.5	66	37	.33
11.20	66.76	1.20	1.79	SANDY SILT TO CLAYEY SILT	ML-MH	.56	12	3.395	19.5	64	37	.29
11.40	114.87	2.96	2.58	SANDY SILT TO CLAYEY SILT	ML-MH	.57	22	5.862	19.5	81	39	.50+
11.60	131.07	3.99	3.05	SANDY SILT TO CLAYEY SILT	ML-MH	.58	25	6.692	19.5	85	40	.50+
11.80	139.57	2.81	2.01	SILTY SAND TO SANDY SILT	SM-ML	.65	22			85	40	.50+
12.00	243.03	5.64	2.32	SILTY SAND TO SANDY SILT	SM-ML	.66	38			100+	42	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 5
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	306.46	1.03	.34	SAND	SW-SP	.01	29			100+	44	.50+
.40	221.55	1.28	.58	SAND	SW-SP	.03	21			100+	44	.50+
.60	132.60	1.77	1.34	SILTY SAND TO SANDY SILT	SM-ML	.03	21			100+	42	.50+
.80	80.90	1.46	1.80	SANDY SILT TO CLAYEY SILT	ML-MH	.04	15	4.147	19.5	100+	42	.50+
1.00	58.75	1.17	2.00	SANDY SILT TO CLAYEY SILT	ML-MH	.05	11	3.010	19.5	92	41	.50+
1.20	49.34	1.06	2.16	SANDY SILT TO CLAYEY SILT	ML-MH	.06	9	2.527	19.5	82	40	.50+
1.40	45.65	1.04	2.27	SANDY SILT TO CLAYEY SILT	ML-MH	.07	8	2.337	19.5	78	39	.50+
1.60	86.60	.75	.87	SILTY SAND TO SANDY SILT	SM-ML	.09	13			100+	42	.50+
1.80	92.63	2.29	2.48	SANDY SILT TO CLAYEY SILT	ML-MH	.09	17	4.746	19.5	100+	42	.50+
2.00	88.03	2.19	2.49	SANDY SILT TO CLAYEY SILT	ML-MH	.10	16	4.509	19.5	100+	42	.50+
2.20	91.63	2.02	2.21	SANDY SILT TO CLAYEY SILT	ML-MH	.11	17	4.693	19.5	100+	42	.50+
2.40	204.60	3.78	1.85	SILTY SAND TO SANDY SILT	SM-ML	.13	32			100+	42	.50+
2.60	158.64	.84	.53	SAND TO SILTY SAND	SM-SP	.16	18			100+	42	.50+
2.80	170.20	3.84	2.26	SILTY SAND TO SANDY SILT	SM-ML	.15	27			100+	42	.50+
3.00	118.67	2.43	2.05	SILTY SAND TO SANDY SILT	SM-ML	.17	18			100+	42	.50+
3.20	116.13	1.86	1.60	SILTY SAND TO SANDY SILT	SM-ML	.18	18			100+	42	.50+
3.40	109.46	1.09	1.00	SILTY SAND TO SANDY SILT	SM-ML	.19	17			100+	42	.50+
3.60	116.49	1.89	1.62	SILTY SAND TO SANDY SILT	SM-ML	.20	18			100+	42	.50+
3.80	90.71	1.59	1.75	SILTY SAND TO SANDY SILT	SM-ML	.21	14			93	41	.50+
4.00	37.28	.86	2.30	CLAYEY SILT TO SILTY CLAY	MH-CL	.18	8	1.903	19.5			
4.20	37.26	.84	2.26	CLAYEY SILT TO SILTY CLAY	MH-CL	.19	8	1.901	19.5			
4.40	37.26	.90	2.41	CLAYEY SILT TO SILTY CLAY	MH-CL	.20	8	1.901	19.5			
4.60	40.61	1.02	2.52	CLAYEY SILT TO SILTY CLAY	MH-CL	.21	9	2.072	19.5			
4.80	37.93	1.37	3.62	SILTY CLAY TO CLAY	CL-CH	.19	12	1.935	19.5			
5.00	41.61	1.47	3.52	SILTY CLAY TO CLAY	CL-CH	.20	13	2.124	19.5			
5.20	52.33	1.75	3.34	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	12	2.672	19.5			
5.40	46.68	1.70	3.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	11	2.382	19.5			
5.60	48.99	1.62	3.31	CLAYEY SILT TO SILTY CLAY	MH-CL	.25	11	2.499	19.5			
5.80	38.60	1.24	3.21	CLAYEY SILT TO SILTY CLAY	MH-CL	.26	9	1.966	19.5			
6.00	42.96	1.32	3.06	CLAYEY SILT TO SILTY CLAY	MH-CL	.27	10	2.189	19.5			
6.20	43.97	1.27	2.88	CLAYEY SILT TO SILTY CLAY	MH-CL	.28	10	2.241	19.5			
6.40	47.99	2.22	4.64	CLAY	CH	.26	22	2.448	19.5			
6.60	58.06	1.91	3.29	CLAYEY SILT TO SILTY CLAY	MH-CL	.30	13	2.962	19.5			
6.80	65.49	1.92	2.94	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	15	3.343	19.5			
7.00	43.00	1.34	3.11	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	10	2.189	19.5			
7.20	46.70	1.17	2.50	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	11	2.378	19.5			
7.40	43.30	1.57	3.63	SILTY CLAY TO CLAY	CL-CH	.30	13	2.205	19.5			
7.60	38.95	1.32	3.39	CLAYEY SILT TO SILTY CLAY	MH-CL	.34	9	1.980	19.5			

7.80	39.58	1.20	3.04	CLAYEY SILT TO SILTY CLAY	MH-CL	.35	9	2.012	19.5			
8.00	54.33	1.64	3.02	CLAYEY SILT TO SILTY CLAY	MH-CL	.36	13	2.768	19.5			
8.20	42.96	1.00	2.34	SANDY SILT TO CLAYEY SILT	ML-MH	.41	8	2.182	19.5	59	36	.21
8.40	27.62	1.04	3.75	SILTY CLAY TO CLAY	CL-CH	.34	8	1.399	19.5			
8.60	28.54	1.05	3.67	SILTY CLAY TO CLAY	CL-CH	.34	9	1.446	19.5			
8.80	23.51	.95	4.04	CLAY	CH	.35	11	1.188	19.5			
9.00	25.86	.85	3.30	SILTY CLAY TO CLAY	CL-CH	.36	8	1.308	19.5			
9.20	32.90	.72	2.19	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	7	1.666	19.5			
9.40	78.68	1.64	2.08	SANDY SILT TO CLAYEY SILT	ML-MH	.47	15	4.011	19.5	72	38	.50+
9.60	51.00	1.30	2.54	SANDY SILT TO CLAYEY SILT	ML-MH	.48	9	2.591	19.5	60	37	.22
9.80	52.10	1.30	2.49	SANDY SILT TO CLAYEY SILT	ML-MH	.49	9	2.647	19.5	61	37	.22
10.00	70.27	1.02	1.45	SANDY SILT TO CLAYEY SILT	ML-MH	.50	13	3.578	19.5	68	38	.42
10.20	206.34	1.74	.84	SAND TO SILTY SAND	SM-SP	.61	24			100+	42	.50+
10.40	45.91	.99	2.15	SANDY SILT TO CLAYEY SILT	ML-MH	.52	8	2.328	19.5	55	36	.19
10.60	21.86	.29	1.31	CLAYEY SILT TO SILTY CLAY	MH-CL	.48	5	1.096	19.5			
10.80	9.44	.50	5.32	CLAY	CH	.43	4	0.462	19.5			
11.00	51.01	.92	1.80	SANDY SILT TO CLAYEY SILT	ML-MH	.55	9	2.588	19.5	58	36	.20
11.20	123.61	2.87	2.32	SANDY SILT TO CLAYEY SILT	ML-MH	.56	23	6.310	19.5	84	40	.50+
11.40	274.03	5.54	2.02	SILTY SAND TO SANDY SILT	SM-ML	.63	43			100+	42	.50+
11.60	225.08	4.77	2.12	SILTY SAND TO SANDY SILT	SM-ML	.64	35			100+	42	.50+
11.80	220.61	5.91	2.68	SANDY SILT TO CLAYEY SILT	ML-MH	.59	42	11.283	19.5	100+	42	.50+
12.00	219.06	6.06	2.77	SANDY SILT TO CLAYEY SILT	ML-MH	.60	41	11.203	19.5	100+	42	.50+
12.20	62.13	2.22	3.57	CLAYEY SILT TO SILTY CLAY	MH-CL	.55	14	3.158	19.5			
12.40	85.71	1.59	1.86	SANDY SILT TO CLAYEY SILT	ML-MH	.62	16	4.364	19.5	69	38	.49
12.60	97.14	2.13	2.19	SANDY SILT TO CLAYEY SILT	ML-MH	.63	18	4.949	19.5	73	38	.50+
12.80	222.23	3.23	1.45	SAND TO SILTY SAND	SM-SP	.77	26			99	42	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 6
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
1.20	59.57	.82	1.38	SANDY SILT TO CLAYEY SILT	ML-MH	.06	11	3.052	19.5	91	41	.50+
1.40	91.75	1.64	1.79	SILTY SAND TO SANDY SILT	SM-ML	.08	14			100+	42	.50+
1.60	112.98	2.37	2.10	SANDY SILT TO CLAYEY SILT	ML-MH	.08	21	5.790	19.5	100+	42	.50+
1.80	94.15	2.27	2.41	SANDY SILT TO CLAYEY SILT	ML-MH	.09	18	4.824	19.5	100+	42	.50+
2.00	89.01	1.98	2.23	SANDY SILT TO CLAYEY SILT	ML-MH	.10	17	4.559	19.5	100+	42	.50+
2.20	84.22	2.26	2.69	SANDY SILT TO CLAYEY SILT	ML-MH	.11	16	4.313	19.5	100+	42	.50+
2.40	75.32	1.86	2.47	SANDY SILT TO CLAYEY SILT	ML-MH	.12	14	3.856	19.5	94	41	.50+
2.60	64.37	1.84	2.87	SANDY SILT TO CLAYEY SILT	ML-MH	.13	12	3.294	19.5	85	40	.50+
2.80	67.78	2.75	4.06	SILTY CLAY TO CLAY	CL-CH	.11	21	3.470	19.5			
3.00	137.29	2.62	1.91	SILTY SAND TO SANDY SILT	SM-ML	.17	21			100+	42	.50+
3.20	82.58	3.22	3.90	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	19	4.228	19.5			
3.40	69.85	2.38	3.40	CLAYEY SILT TO SILTY CLAY	MH-CL	.15	16	3.574	19.5			
3.60	51.35	2.26	4.39	SILTY CLAY TO CLAY	CL-CH	.14	16	2.626	19.5			
3.80	56.83	2.18	3.83	CLAYEY SILT TO SILTY CLAY	MH-CL	.17	13	2.906	19.5			
4.00	43.15	2.17	5.03	CLAY	CH	.16	20	2.205	19.5			
4.20	43.47	1.81	4.17	SILTY CLAY TO CLAY	CL-CH	.17	13	2.221	19.5			
4.40	50.32	1.88	3.73	CLAYEY SILT TO SILTY CLAY	MH-CL	.20	12	2.570	19.5			
4.60	46.22	1.89	4.09	SILTY CLAY TO CLAY	CL-CH	.18	14	2.361	19.5			
4.80	43.31	1.83	4.23	SILTY CLAY TO CLAY	CL-CH	.19	13	2.211	19.5			
5.00	46.56	2.15	4.61	CLAY	CH	.20	22	2.377	19.5			
5.20	61.62	2.24	3.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	14	3.148	19.5			
5.40	69.13	2.66	3.85	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	16	3.533	19.5			
5.60	58.56	2.31	3.95	SILTY CLAY TO CLAY	CL-CH	.22	18	2.992	19.5			
5.80	53.06	1.86	3.51	CLAYEY SILT TO SILTY CLAY	MH-CL	.26	12	2.708	19.5			
6.00	54.07	2.26	4.17	SILTY CLAY TO CLAY	CL-CH	.24	17	2.761	19.5			
6.20	47.24	2.31	4.88	CLAY	CH	.25	22	2.410	19.5			
6.40	41.80	2.06	4.93	CLAY	CH	.26	20	2.130	19.5			
6.60	35.61	1.65	4.64	CLAY	CH	.26	17	1.813	19.5			
6.80	30.13	1.42	4.73	CLAY	CH	.27	14	1.531	19.5			
7.00	30.80	1.42	4.63	CLAY	CH	.28	14	1.565	19.5			
7.20	29.11	1.28	4.41	CLAY	CH	.29	13	1.478	19.5			
7.40	27.05	1.72	6.36	CLAY	CH	.30	12	1.372	19.5			
7.60	29.44	1.91	6.47	CLAY	CH	.30	14	1.494	19.5			
7.80	38.67	2.93	7.59	CLAY	CH	.31	18	1.967	19.5			
8.00	106.47	3.75	3.52	CLAYEY SILT TO SILTY CLAY	MH-CL	.36	25	5.442	19.5			
8.20	73.45	3.76	5.11	CLAY	CH	.33	35	3.750	19.5			
8.40	65.80	3.53	5.37	CLAY	CH	.34	31	3.357	19.5			
8.60	65.05	4.55	7.00	CLAY	CH	.34	31	3.318	19.5			

8.80	129.15	6.42	4.97	VERY STIFF FINE GRAINED	CH-CL	.48	61	6.598	19.5	89	28-	.50+
9.00	63.11	4.23	6.71	CLAY	CH	.36	30	3.218	19.5			
9.20	99.47	3.90	3.92	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	23	5.080	19.5			
9.40	82.89	3.66	4.41	SILTY CLAY TO CLAY	CL-CH	.38	26	4.231	19.5			
9.60	79.77	3.57	4.48	SILTY CLAY TO CLAY	CL-CH	.38	25	4.071	19.5			
9.80	69.17	2.51	3.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.44	16	3.525	19.5			
10.00	62.32	3.14	5.04	CLAY	CH	.40	29	3.175	19.5			
10.20	61.64	2.79	4.53	SILTY CLAY TO CLAY	CL-CH	.41	19	3.140	19.5			
10.40	60.28	2.71	4.49	SILTY CLAY TO CLAY	CL-CH	.42	19	3.070	19.5			
10.60	74.94	3.27	4.37	SILTY CLAY TO CLAY	CL-CH	.42	23	3.822	19.5			
10.80	185.82	10.68	5.75	VERY STIFF FINE GRAINED	CH-CL	.59	89	9.499	19.5	98	28-	.50+
11.00	100.44	5.05	5.03	SILTY CLAY TO CLAY	CL-CH	.44	32	5.128	19.5			
11.20	130.83	6.66	5.09	VERY STIFF FINE GRAINED	CH-CL	.62	62	6.677	19.5	84	28-	.50+
11.40	75.30	3.14	4.17	SILTY CLAY TO CLAY	CL-CH	.46	24	3.838	19.5			
11.60	91.59	3.93	4.29	SILTY CLAY TO CLAY	CL-CH	.46	29	4.673	19.5			
11.80	96.85	4.35	4.49	SILTY CLAY TO CLAY	CL-CH	.47	30	4.943	19.5			
12.00	78.97	4.17	5.28	CLAY	CH	.48	37	4.025	19.5			
12.20	79.77	4.09	5.13	CLAY	CH	.49	38	4.066	19.5			
12.40	81.81	4.44	5.43	CLAY	CH	.50	39	4.170	19.5			
12.60	78.73	6.40	8.13	CLAY	CH	.50	37	4.012	19.5			
12.80	259.51	8.81	3.40	SANDY SILT TO CLAYEY SILT	ML-MH	.64	49	13.275	19.5	100+	42	.50+
13.00	124.49	6.96	5.59	VERY STIFF FINE GRAINED	CH-CL	.71	59	6.348	19.5	80	28-	.47
13.20	110.28	5.26	4.77	SILTY CLAY TO CLAY	CL-CH	.53	35	5.628	19.5			
13.40	101.38	4.78	4.72	SILTY CLAY TO CLAY	CL-CH	.54	32	5.171	19.5			

otes:

1. Default values of submerged unit weight and cone factor are used.
. Relative density is estimated based on Schmertmann (1978).
. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 7
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	71.89	1.48	2.06	SANDY SILT TO CLAYEY SILT	ML-MH	.01	13	3.686	19.5	100+	42	.50+
.40	100.31	3.40	3.39	CLAYEY SILT TO SILTY CLAY	MH-CL	.02	24	5.143	19.5			
.60	85.59	2.25	2.63	SANDY SILT TO CLAYEY SILT	ML-MH	.03	16	4.388	19.5	100+	42	.50+
.80	66.42	1.65	2.48	SANDY SILT TO CLAYEY SILT	ML-MH	.04	12	3.404	19.5	100+	42	.50+
1.00	73.25	2.36	3.22	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	17	3.754	19.5			
1.20	53.24	1.30	2.44	SANDY SILT TO CLAYEY SILT	ML-MH	.06	10	2.727	19.5	85	40	.50+
1.40	70.52	1.58	2.24	SANDY SILT TO CLAYEY SILT	ML-MH	.07	13	3.613	19.5	99	42	.50+
1.60	59.23	1.39	2.35	SANDY SILT TO CLAYEY SILT	ML-MH	.08	11	3.033	19.5	88	40	.50+
1.80	60.60	1.39	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.09	11	3.103	19.5	87	40	.50+
2.00	67.27	1.30	1.93	SANDY SILT TO CLAYEY SILT	ML-MH	.10	12	3.445	19.5	91	41	.50+
2.20	48.97	.40	.82	SILTY SAND TO SANDY SILT	SM-ML	.12	7			76	39	.50+
2.40	43.48	.56	1.29	SANDY SILT TO CLAYEY SILT	ML-MH	.12	8	2.224	19.5	73	38	.50+
2.60	31.84	.51	1.61	SANDY SILT TO CLAYEY SILT	ML-MH	.13	6	1.626	19.5	65	37	.50+
2.80	27.90	.52	1.88	CLAYEY SILT TO SILTY CLAY	MH-CL	.13	6	1.424	19.5			
3.00	40.72	.93	2.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	9	2.081	19.5			
3.20	53.06	.96	1.81	SANDY SILT TO CLAYEY SILT	ML-MH	.16	10	2.713	19.5	76	39	.50+
3.40	33.64	.63	1.88	SANDY SILT TO CLAYEY SILT	ML-MH	.17	6	1.716	19.5	64	37	.45
3.60	42.94	.98	2.28	SANDY SILT TO CLAYEY SILT	ML-MH	.18	8	2.193	19.5	69	38	.50+
3.80	29.83	.30	.99	SANDY SILT TO CLAYEY SILT	ML-MH	.19	5	1.520	19.5	61	37	.27
4.00	30.82	.44	1.43	SANDY SILT TO CLAYEY SILT	ML-MH	.20	5	1.570	19.5	61	37	.26
4.20	29.46	1.06	3.60	SILTY CLAY TO CLAY	CL-CH	.17	9	1.502	19.5			
4.40	28.07	.54	1.93	CLAYEY SILT TO SILTY CLAY	MH-CL	.20	6	1.429	19.5			
4.60	32.52	.73	2.24	CLAYEY SILT TO SILTY CLAY	MH-CL	.21	7	1.657	19.5			
4.80	40.39	1.21	3.00	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	9	2.060	19.5			
5.00	34.05	.91	2.68	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	8	1.735	19.5			
5.20	35.43	.80	2.25	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	8	1.805	19.5			
5.40	31.84	.79	2.49	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	7	1.621	19.5			
5.60	27.90	.76	2.72	CLAYEY SILT TO SILTY CLAY	MH-CL	.25	6	1.418	19.5			
5.80	33.20	1.07	3.21	SILTY CLAY TO CLAY	CL-CH	.23	10	1.691	19.5			
6.00	32.87	1.06	3.24	SILTY CLAY TO CLAY	CL-CH	.24	10	1.673	19.5			
6.20	39.35	.50	1.27	SANDY SILT TO CLAYEY SILT	ML-MH	.31	7	2.002	19.5	61	37	.24
6.40	23.69	.72	3.04	SILTY CLAY TO CLAY	CL-CH	.26	7	1.202	19.5			
6.60	16.44	.35	2.13	SILTY CLAY TO CLAY	CL-CH	.26	5	0.830	19.5			
6.80	15.42	.37	2.40	SILTY CLAY TO CLAY	CL-CH	.27	4	0.777	19.5			
7.00	51.70	.61	1.18	SANDY SILT TO CLAYEY SILT	ML-MH	.35	9	2.633	19.5	65	37	.35
7.20	33.90	.86	2.55	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	8	1.722	19.5			
7.40	22.94	.41	1.79	CLAYEY SILT TO SILTY CLAY	MH-CL	.33	5	1.159	19.5			
7.60	30.13	.27	.91	SANDY SILT TO CLAYEY SILT	ML-MH	.38	5	1.526	19.5	50	35	.16

7.80	35.96	1.34	3.74	SILTY CLAY TO CLAY	CL-CH	.31	11	1.828	19.5			
8.00	24.31	.82	3.39	SILTY CLAY TO CLAY	CL-CH	.32	7	1.230	19.5			
8.20	29.08	.78	2.69	CLAYEY SILT TO SILTY CLAY	MH-CL	.37	6	1.472	19.5			
8.40	27.39	.96	3.50	SILTY CLAY TO CLAY	CL-CH	.34	8	1.387	19.5			
8.60	21.60	.81	3.76	CLAY	CH	.34	10	1.090	19.5			
8.80	17.46	.48	2.76	SILTY CLAY TO CLAY	CL-CH	.35	5	0.877	19.5			
9.00	23.01	.38	1.64	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	5	1.159	19.5			
9.20	23.96	.35	1.44	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	5	1.208	19.5			
9.40	43.34	.85	1.97	SANDY SILT TO CLAYEY SILT	ML-MH	.47	8	2.198	19.5	56	36	.19
9.60	49.71	1.18	2.37	SANDY SILT TO CLAYEY SILT	ML-MH	.48	9	2.525	19.5	60	36	.21
9.80	33.91	.94	2.76	CLAYEY SILT TO SILTY CLAY	MH-CL	.44	8	1.716	19.5			
10.00	39.01	.76	1.96	SANDY SILT TO CLAYEY SILT	ML-MH	.50	7	1.975	19.5	51	35	.17
10.20	39.69	.54	1.35	SANDY SILT TO CLAYEY SILT	ML-MH	.51	7	2.009	19.5	51	35	.17
10.40	36.46	.52	1.43	SANDY SILT TO CLAYEY SILT	ML-MH	.52	6	1.843	19.5	48	35	.16
10.60	34.91	.87	2.49	CLAYEY SILT TO SILTY CLAY	MH-CL	.48	8	1.766	19.5			
10.80	38.36	1.01	2.65	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	9	1.942	19.5			
11.00	62.53	.64	1.02	SILTY SAND TO SANDY SILT	SM-ML	.60	9			61	37	.24
11.20	62.30	.47	.75	SILTY SAND TO SANDY SILT	SM-ML	.62	9			61	37	.23
11.40	83.16	.35	.42	SAND TO SILTY SAND	SM-SP	.68	9			67	39	.36
11.60	69.16	.47	.68	SILTY SAND TO SANDY SILT	SM-ML	.64	11			63	37	.26
11.80	49.57	.34	.68	SILTY SAND TO SANDY SILT	SM-ML	.65	7			52	35	.18
12.00	14.93	.36	2.39	SILTY CLAY TO CLAY	CL-CH	.48	4	0.741	19.5			
12.20	12.67	.35	2.74	SILTY CLAY TO CLAY	CL-CH	.49	4	0.625	19.5			
12.40	11.30	.24	2.08	SILTY CLAY TO CLAY	CL-CH	.50	3	0.554	19.5			
12.60	13.37	.18	1.31	SENSITIVE FINE GRAINED	OL-CH	.38	3	0.666	19.5			
12.80	11.64	.10	.85	SENSITIVE FINE GRAINED	OL-CH	.38	2	0.577	19.5			
13.00	10.95	.15	1.33	SENSITIVE FINE GRAINED	OL-CH	.39	2	0.542	19.5			
13.20	12.14	.12	1.01	SENSITIVE FINE GRAINED	OL-CH	.40	2	0.602	19.5			
13.40	15.29	.34	2.19	SILTY CLAY TO CLAY	CL-CH	.54	4	0.756	19.5			
13.60	11.66	.13	1.07	SENSITIVE FINE GRAINED	OL-CH	.41	2	0.577	19.5			
13.80	10.27	.28	2.75	CLAY	CH	.55	4	0.498	19.5			
14.00	17.10	.35	2.03	CLAYEY SILT TO SILTY CLAY	MH-CL	.63	4	0.845	19.5			
14.20	23.23	.01	.04	SANDY SILT TO CLAYEY SILT	ML-MH	.71	4	1.155	19.5	25	32	.12
14.40	16.15	.25	1.58	CLAYEY SILT TO SILTY CLAY	MH-CL	.65	3	0.795	19.5			
14.60	10.28	.15	1.50	SENSITIVE FINE GRAINED	OL-CH	.44	2	0.505	19.5			
14.80	7.88	.06	.81	SENSITIVE FINE GRAINED	OL-CH	.44	1	0.382	19.5			
15.00	12.27	.12	.99	SENSITIVE FINE GRAINED	OL-CH	.45	2	0.606	19.5			
15.20	83.44	1.92	2.31	SANDY SILT TO CLAYEY SILT	ML-MH	.76	15	4.240	19.5	65	37	.30
15.40	103.27	4.18	4.04	CLAYEY SILT TO SILTY CLAY	MH-CL	.69	24	5.261	19.5			
15.60	137.38	4.75	3.46	SANDY SILT TO CLAYEY SILT	ML-MH	.78	26	7.005	19.5	81	39	.50+
15.80	128.72	5.53	4.29	CLAYEY SILT TO SILTY CLAY	MH-CL	.71	30	6.565	19.5			
16.00	154.45	7.96	5.15	VERY STIFF FINE GRAINED	CH-CL	.88	73	7.875	19.5	82	28-	.50+
16.20	103.05	3.36	3.26	SANDY SILT TO CLAYEY SILT	ML-MH	.81	19	5.243	19.5	70	38	.49
16.40	93.01	2.11	2.27	SANDY SILT TO CLAYEY SILT	ML-MH	.82	17	4.728	19.5	66	37	.35
16.60	110.44	2.73	2.48	SANDY SILT TO CLAYEY SILT	ML-MH	.83	21	5.621	19.5	71	38	.50+
16.80	94.26	3.94	4.18	CLAYEY SILT TO SILTY CLAY	MH-CL	.76	22	4.795	19.5			
17.00	91.33	2.43	2.66	SANDY SILT TO CLAYEY SILT	ML-MH	.85	17	4.640	19.5	65	37	.31
17.20	72.60	1.66	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.86	13	3.679	19.5	58	36	.21
17.40	136.46	2.85	2.09	SILTY SAND TO SANDY SILT	SM-ML	.96	21			75	39	.50+
17.60	111.58	2.45	2.19	SANDY SILT TO CLAYEY SILT	ML-MH	.88	21	5.677	19.5	70	38	.50+
17.80	137.22	2.73	1.99	SILTY SAND TO SANDY SILT	SM-ML	.98	21			75	39	.50+
18.00	130.31	3.86	2.96	SANDY SILT TO CLAYEY SILT	ML-MH	.90	24	6.636	19.5	75	39	.50+

18.20	130.40	4.89	3.75	CLAYEY SILT TO SILTY CLAY	MH-CL	.82	31	6.645	19.5			
18.40	100.41	4.43	4.41	SILTY CLAY TO CLAY	CL-CH	.74	32	5.111	19.5			
18.60	103.84	2.74	2.64	SANDY SILT TO CLAYEY SILT	ML-MH	.93	19	5.277	19.5	67	37	.37
18.80	117.68	2.98	2.53	SANDY SILT TO CLAYEY SILT	ML-MH	.94	22	5.987	19.5	70	38	.50+
19.00	135.60	2.47	1.82	SILTY SAND TO SANDY SILT	SM-ML	1.04	21			73	38	.50+
19.20	168.32	2.64	1.57	SILTY SAND TO SANDY SILT	SM-ML	1.06	26			81	39	.50+
19.40	172.17	2.75	1.59	SILTY SAND TO SANDY SILT	SM-ML	1.07	27			81	39	.50+
19.60	147.63	3.75	2.54	SANDY SILT TO CLAYEY SILT	ML-MH	.98	28	7.521	19.5	78	39	.50+
19.80	144.42	4.73	3.27	SANDY SILT TO CLAYEY SILT	ML-MH	.99	27	7.355	19.5	77	39	.50+
20.00	183.09	5.09	2.78	SANDY SILT TO CLAYEY SILT	ML-MH	1.00	35	9.338	19.5	84	40	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 8
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	33.21	.47	1.42	SANDY SILT TO CLAYEY SILT	ML-MH	.01	6	1.703	19.5	74	38	.50+
.40	39.71	1.03	2.59	CLAYEY SILT TO SILTY CLAY	MH-CL	.02	9	2.035	19.5			
.60	31.15	.65	2.09	CLAYEY SILT TO SILTY CLAY	MH-CL	.03	7	1.596	19.5			
.80	30.81	.34	1.10	SANDY SILT TO CLAYEY SILT	ML-MH	.04	5	1.578	19.5	70	38	.50+
1.00	46.24	.76	1.65	SANDY SILT TO CLAYEY SILT	ML-MH	.05	8	2.369	19.5	80	39	.50+
1.20	42.45	.38	.90	SANDY SILT TO CLAYEY SILT	ML-MH	.06	8	2.174	19.5	77	39	.50+
1.40	44.18	.75	1.70	SANDY SILT TO CLAYEY SILT	ML-MH	.07	8	2.262	19.5	77	39	.50+
1.60	34.58	.46	1.33	SANDY SILT TO CLAYEY SILT	ML-MH	.08	6	1.769	19.5	70	38	.50+
1.80	35.26	.62	1.76	SANDY SILT TO CLAYEY SILT	ML-MH	.09	6	1.804	19.5	70	38	.50+
2.00	26.03	.40	1.55	CLAYEY SILT TO SILTY CLAY	MH-CL	.09	6	1.330	19.5			
2.20	29.10	.47	1.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.10	6	1.487	19.5			
2.40	38.68	.77	2.00	SANDY SILT TO CLAYEY SILT	ML-MH	.12	7	1.977	19.5	70	38	.50+
2.60	35.62	.58	1.62	SANDY SILT TO CLAYEY SILT	ML-MH	.13	6	1.820	19.5	67	37	.50+
2.80	40.41	.55	1.35	SANDY SILT TO CLAYEY SILT	ML-MH	.14	7	2.065	19.5	70	38	.50+
3.00	49.46	.61	1.23	SANDY SILT TO CLAYEY SILT	ML-MH	.15	9	2.529	19.5	74	38	.50+
3.20	29.78	.62	2.08	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	7	1.520	19.5			
3.40	29.78	.48	1.62	SANDY SILT TO CLAYEY SILT	ML-MH	.17	5	1.518	19.5	62	37	.31
3.60	31.33	.51	1.62	SANDY SILT TO CLAYEY SILT	ML-MH	.18	6	1.597	19.5	62	37	.32
3.80	33.21	.70	2.11	CLAYEY SILT TO SILTY CLAY	MH-CL	.17	7	1.694	19.5			
4.00	59.53	1.59	2.67	SANDY SILT TO CLAYEY SILT	ML-MH	.20	11	3.043	19.5	77	39	.50+
4.20	75.99	1.96	2.58	SANDY SILT TO CLAYEY SILT	ML-MH	.21	14	3.886	19.5	85	40	.50+
4.40	77.55	1.64	2.11	SANDY SILT TO CLAYEY SILT	ML-MH	.22	14	3.966	19.5	85	40	.50+
4.60	74.54	1.38	1.84	SANDY SILT TO CLAYEY SILT	ML-MH	.23	14	3.811	19.5	83	40	.50+
4.80	83.18	2.07	2.49	SANDY SILT TO CLAYEY SILT	ML-MH	.24	15	4.253	19.5	86	40	.50+
5.00	52.04	.97	1.86	SANDY SILT TO CLAYEY SILT	ML-MH	.25	9	2.656	19.5	70	38	.50+
5.20	61.12	.96	1.58	SANDY SILT TO CLAYEY SILT	ML-MH	.26	11	3.121	19.5	74	38	.50+
5.40	61.00	1.05	1.72	SANDY SILT TO CLAYEY SILT	ML-MH	.27	11	3.114	19.5	74	38	.50+
5.60	42.79	.68	1.59	SANDY SILT TO CLAYEY SILT	ML-MH	.28	8	2.180	19.5	64	37	.32
5.80	57.48	1.11	1.94	SANDY SILT TO CLAYEY SILT	ML-MH	.29	11	2.933	19.5	71	38	.50+
6.00	55.15	1.07	1.95	SANDY SILT TO CLAYEY SILT	ML-MH	.30	10	2.813	19.5	69	38	.50+
6.20	53.07	1.24	2.34	SANDY SILT TO CLAYEY SILT	ML-MH	.31	10	2.706	19.5	68	38	.50+
6.40	58.52	1.89	3.23	CLAYEY SILT TO SILTY CLAY	MH-CL	.29	14	2.986	19.5			
6.60	50.33	1.65	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.30	12	2.566	19.5			
6.80	46.58	1.51	3.23	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	11	2.373	19.5			
7.00	40.75	1.27	3.12	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	9	2.074	19.5			
7.20	46.54	1.55	3.32	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	11	2.370	19.5			
7.40	49.97	1.52	3.04	CLAYEY SILT TO SILTY CLAY	MH-CL	.33	11	2.546	19.5			
7.60	49.82	1.71	3.43	CLAYEY SILT TO SILTY CLAY	MH-CL	.34	11	2.537	19.5			

7.80	38.04	.83	2.19	CLAYEY SILT TO SILTY CLAY	MH-CL	.35	9	1.933	19.5			
8.00	43.80	1.42	3.24	CLAYEY SILT TO SILTY CLAY	MH-CL	.36	10	2.228	19.5			
8.20	51.26	1.37	2.68	CLAYEY SILT TO SILTY CLAY	MH-CL	.37	12	2.610	19.5			
8.40	43.54	1.07	2.45	CLAYEY SILT TO SILTY CLAY	MH-CL	.38	10	2.213	19.5			
8.60	56.09	1.58	2.82	CLAYEY SILT TO SILTY CLAY	MH-CL	.39	13	2.856	19.5			
8.80	29.85	1.11	3.72	SILTY CLAY TO CLAY	CL-CH	.35	9	1.513	19.5			
9.00	30.80	.58	1.89	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	7	1.559	19.5			
9.20	38.66	.94	2.44	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	9	1.962	19.5			
9.40	42.48	1.45	3.40	CLAYEY SILT TO SILTY CLAY	MH-CL	.42	10	2.157	19.5			
9.60	44.52	1.81	4.07	SILTY CLAY TO CLAY	CL-CH	.38	14	2.264	19.5			
9.80	40.88	1.18	2.89	CLAYEY SILT TO SILTY CLAY	MH-CL	.44	9	2.074	19.5			
10.00	54.90	1.03	1.88	SANDY SILT TO CLAYEY SILT	ML-MH	.50	10	2.790	19.5	61	37	.24
10.20	35.95	.79	2.18	CLAYEY SILT TO SILTY CLAY	MH-CL	.46	8	1.820	19.5			
10.40	30.30	.49	1.61	SANDY SILT TO CLAYEY SILT	ML-MH	.52	5	1.527	19.5	43	34	.15
10.60	29.10	.52	1.80	CLAYEY SILT TO SILTY CLAY	MH-CL	.48	6	1.468	19.5			
10.80	28.41	.46	1.64	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	6	1.432	19.5			
11.00	40.74	1.28	3.13	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	9	2.064	19.5			
11.20	33.92	.98	2.88	CLAYEY SILT TO SILTY CLAY	MH-CL	.50	8	1.714	19.5			
11.40	32.88	.72	2.20	CLAYEY SILT TO SILTY CLAY	MH-CL	.51	7	1.660	19.5			
11.60	28.44	.56	1.98	CLAYEY SILT TO SILTY CLAY	MH-CL	.52	6	1.432	19.5			
11.80	31.64	.94	2.96	CLAYEY SILT TO SILTY CLAY	MH-CL	.53	7	1.595	19.5			
12.00	50.02	1.56	3.13	CLAYEY SILT TO SILTY CLAY	MH-CL	.54	11	2.537	19.5			
12.20	50.66	1.31	2.58	CLAYEY SILT TO SILTY CLAY	MH-CL	.55	12	2.570	19.5			
12.40	56.33	1.71	3.03	CLAYEY SILT TO SILTY CLAY	MH-CL	.56	13	2.860	19.5			
12.60	47.28	1.36	2.87	CLAYEY SILT TO SILTY CLAY	MH-CL	.57	11	2.395	19.5			
12.80	47.63	.98	2.05	SANDY SILT TO CLAYEY SILT	ML-MH	.64	9	2.410	19.5	51	35	.17
13.00	49.65	1.02	2.04	SANDY SILT TO CLAYEY SILT	ML-MH	.65	9	2.513	19.5	52	35	.18
13.20	60.93	1.99	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.59	14	3.094	19.5			
13.40	70.52	1.34	1.90	SANDY SILT TO CLAYEY SILT	ML-MH	.67	13	3.582	19.5	63	37	.26
13.60	110.86	1.88	1.70	SILTY SAND TO SANDY SILT	SM-ML	.75	17			74	38	.50+
13.80	146.60	1.39	.95	SAND TO SILTY SAND	SM-SP	.83	17			81	40	.50+
14.00	188.23	1.33	.70	SAND TO SILTY SAND	SM-SP	.84	22			89	41	.50+
14.20	165.92	1.76	1.06	SAND TO SILTY SAND	SM-SP	.85	19			85	41	.50+
14.40	64.41	1.32	2.05	SANDY SILT TO CLAYEY SILT	ML-MH	.72	12	3.266	19.5	59	36	.21
14.60	55.85	1.07	1.92	SANDY SILT TO CLAYEY SILT	ML-MH	.73	10	2.827	19.5	53	35	.18
14.80	59.53	2.00	3.36	CLAYEY SILT TO SILTY CLAY	MH-CL	.67	14	3.018	19.5			
15.00	161.69	5.43	3.36	SANDY SILT TO CLAYEY SILT	ML-MH	.75	30	8.253	19.5	87	40	.50+
15.20	172.80	6.36	3.68	CLAYEY SILT TO SILTY CLAY	MH-CL	.68	41	8.827	19.5			
15.40	153.33	4.72	3.08	SANDY SILT TO CLAYEY SILT	ML-MH	.77	29	7.824	19.5	84	40	.50+
15.60	218.38	6.08	2.78	SANDY SILT TO CLAYEY SILT	ML-MH	.78	41	11.159	19.5	97	42	.50+
15.80	162.02	6.06	3.74	CLAYEY SILT TO SILTY CLAY	MH-CL	.71	38	8.272	19.5			
16.00	79.97	1.67	2.08	SANDY SILT TO CLAYEY SILT	ML-MH	.80	15	4.060	19.5	62	37	.26
16.20	99.56	2.86	2.87	SANDY SILT TO CLAYEY SILT	ML-MH	.81	19	5.064	19.5	68	38	.43
16.40	115.07	3.86	3.35	SANDY SILT TO CLAYEY SILT	ML-MH	.82	22	5.859	19.5	73	38	.50+
16.60	114.35	3.47	3.03	SANDY SILT TO CLAYEY SILT	ML-MH	.83	21	5.822	19.5	72	38	.50+
16.80	171.79	3.39	1.97	SILTY SAND TO SANDY SILT	SM-ML	.92	27			84	40	.50+
17.00	319.17	3.74	1.17	SAND TO SILTY SAND	SM-SP	1.02	38			100+	42	.50+
17.20	344.76	3.68	1.07	SAND	SW-SP	1.12	33			100+	44	.50+
17.40	256.35	3.47	1.36	SAND TO SILTY SAND	SM-SP	1.04	30			95	42	.50+
17.60	214.38	2.99	1.39	SAND TO SILTY SAND	SM-SP	1.06	25			88	41	.50+
17.80	125.53	2.26	1.80	SILTY SAND TO SANDY SILT	SM-ML	.98	20			72	38	.50+
18.00	121.27	2.91	2.40	SANDY SILT TO CLAYEY SILT	ML-MH	.90	23	6.173	19.5	73	38	.50+

18.20	108.59	3.80	3.50	CLAYEY SILT TO SILTY CLAY	MH-CL	.82	26	5.527	19.5			
18.40	118.42	3.91	3.30	SANDY SILT TO CLAYEY SILT	ML-MH	.92	22	6.026	19.5	71	38	.50+
18.60	140.49	2.47	1.76	SILTY SAND TO SANDY SILT	SM-ML	1.02	22			75	39	.50+
18.80	258.18	3.90	1.51	SAND TO SILTY SAND	SM-SP	1.13	30			93	42	.50+
19.00	182.24	1.66	.91	SAND TO SILTY SAND	SM-SP	1.14	21			81	40	.50+
19.20	225.64	2.77	1.23	SAND TO SILTY SAND	SM-SP	1.15	27			88	41	.50+
19.40	268.07	3.04	1.13	SAND TO SILTY SAND	SM-SP	1.16	32			94	42	.50+
19.60	276.86	3.85	1.39	SAND TO SILTY SAND	SM-SP	1.18	33			95	42	.50+
19.80	175.06	4.69	2.68	SANDY SILT TO CLAYEY SILT	ML-MH	.99	33	8.927	19.5	83	40	.50+
20.00	174.45	5.18	2.97	SANDY SILT TO CLAYEY SILT	ML-MH	1.00	33	8.895	19.5	83	40	.50+
20.20	149.05	5.17	3.47	SANDY SILT TO CLAYEY SILT	ML-MH	1.01	28	7.592	19.5	78	39	.50+
20.40	159.77	5.53	3.46	SANDY SILT TO CLAYEY SILT	ML-MH	1.02	30	8.141	19.5	80	39	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 9
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	37.66	.43	1.14	SANDY SILT TO CLAYEY SILT	ML-MH	.01	7	1.931	19.5	77	39	.50+
.40	53.41	1.04	1.94	SANDY SILT TO CLAYEY SILT	ML-MH	.02	10	2.738	19.5	93	41	.50+
.60	42.80	1.12	2.62	CLAYEY SILT TO SILTY CLAY	MH-CL	.03	10	2.193	19.5			
.80	32.18	.65	2.02	CLAYEY SILT TO SILTY CLAY	MH-CL	.04	7	1.648	19.5			
1.00	27.56	.51	1.84	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	6	1.411	19.5			
1.20	31.15	.67	2.16	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	7	1.595	19.5			
1.40	28.07	.53	1.89	CLAYEY SILT TO SILTY CLAY	MH-CL	.06	6	1.436	19.5			
1.60	27.56	.72	2.61	CLAYEY SILT TO SILTY CLAY	MH-CL	.07	6	1.410	19.5			
1.80	26.53	.55	2.08	CLAYEY SILT TO SILTY CLAY	MH-CL	.08	6	1.356	19.5			
2.00	32.17	.55	1.72	SANDY SILT TO CLAYEY SILT	ML-MH	.10	6	1.645	19.5	67	37	.50+
2.20	24.99	.87	3.46	SILTY CLAY TO CLAY	CL-CH	.09	7	1.277	19.5			
2.40	27.76	.50	1.80	CLAYEY SILT TO SILTY CLAY	MH-CL	.11	6	1.418	19.5			
2.60	27.05	.60	2.23	CLAYEY SILT TO SILTY CLAY	MH-CL	.12	6	1.381	19.5			
2.80	27.05	.56	2.05	CLAYEY SILT TO SILTY CLAY	MH-CL	.13	6	1.381	19.5			
3.00	24.31	.47	1.94	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	5	1.239	19.5			
3.20	19.51	.39	2.01	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	4	0.993	19.5			
3.40	22.93	.54	2.35	CLAYEY SILT TO SILTY CLAY	MH-CL	.15	5	1.168	19.5			
3.60	44.85	.13	.30	SILTY SAND TO SANDY SILT	SM-ML	.20	7			69	38	.50+
3.80	17.13	.05	.29	SENSITIVE FINE GRAINED	OL-CH	.11	4	0.873	19.5			
4.00	14.37	.07	.46	SENSITIVE FINE GRAINED	OL-CH	.12	3	0.731	19.5			
4.20	41.07	1.05	2.56	CLAYEY SILT TO SILTY CLAY	MH-CL	.19	9	2.096	19.5			
4.40	40.25	.91	2.26	CLAYEY SILT TO SILTY CLAY	MH-CL	.20	9	2.054	19.5			
4.60	45.18	.44	.98	SANDY SILT TO CLAYEY SILT	ML-MH	.23	8	2.305	19.5	67	38	.50+
4.80	33.90	.15	.46	SANDY SILT TO CLAYEY SILT	ML-MH	.24	6	1.726	19.5	61	37	.25
5.00	32.69	.38	1.15	SANDY SILT TO CLAYEY SILT	ML-MH	.25	6	1.664	19.5	60	36	.23
5.20	25.69	.57	2.22	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	6	1.306	19.5			
5.40	26.02	.53	2.05	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	6	1.322	19.5			
5.60	15.93	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.17	3	0.808	19.5			
5.80	16.08	.16	.97	SENSITIVE FINE GRAINED	OL-CH	.17	3	0.816	19.5			
6.00	18.48	.14	.77	CLAYEY SILT TO SILTY CLAY	MH-CL	.27	4	0.934	19.5			
6.20	14.72	.08	.56	SENSITIVE FINE GRAINED	OL-CH	.19	3	0.745	19.5			
6.40	11.64	.27	2.32	SILTY CLAY TO CLAY	CL-CH	.26	3	0.584	19.5			
6.60	12.68	.31	2.47	SILTY CLAY TO CLAY	CL-CH	.26	4	0.637	19.5			
6.80	14.03	.20	1.40	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	3	0.704	19.5			
7.00	26.36	.85	3.22	SILTY CLAY TO CLAY	CL-CH	.28	8	1.337	19.5			
7.20	23.53	.41	1.75	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	5	1.190	19.5			
7.40	26.70	.68	2.56	CLAYEY SILT TO SILTY CLAY	MH-CL	.33	6	1.352	19.5			
7.60	17.14	.23	1.32	CLAYEY SILT TO SILTY CLAY	MH-CL	.34	4	0.862	19.5			

7.80	13.72	.14	.98	SENSITIVE FINE GRAINED	OL-CH	.23	3	0.692	19.5			
8.00	12.67	.17	1.31	SENSITIVE FINE GRAINED	OL-CH	.24	3	0.637	19.5			
8.20	12.67	.25	1.95	SILTY CLAY TO CLAY	CL-CH	.33	4	0.633	19.5			
8.40	20.15	.62	3.10	SILTY CLAY TO CLAY	CL-CH	.34	6	1.016	19.5			
8.60	21.58	.64	2.97	SILTY CLAY TO CLAY	CL-CH	.34	6	1.089	19.5			
8.80	21.59	.37	1.73	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	5	1.087	19.5			
9.00	22.97	.72	3.13	SILTY CLAY TO CLAY	CL-CH	.36	7	1.159	19.5			
9.20	23.00	.44	1.92	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	5	1.158	19.5			
9.40	16.26	.23	1.42	CLAYEY SILT TO SILTY CLAY	MH-CL	.42	3	0.812	19.5			
9.60	23.98	.29	1.22	CLAYEY SILT TO SILTY CLAY	MH-CL	.43	5	1.208	19.5			
9.80	17.46	.17	.95	CLAYEY SILT TO SILTY CLAY	MH-CL	.44	4	0.873	19.5			
10.00	17.29	.23	1.35	CLAYEY SILT TO SILTY CLAY	MH-CL	.45	4	0.864	19.5			
10.20	15.10	.37	2.47	SILTY CLAY TO CLAY	CL-CH	.41	4	0.753	19.5			
10.40	13.01	.16	1.27	SENSITIVE FINE GRAINED	OL-CH	.31	3	0.651	19.5			
10.60	13.69	.23	1.70	SILTY CLAY TO CLAY	CL-CH	.42	4	0.681	19.5			
10.80	10.96	.20	1.86	SILTY CLAY TO CLAY	CL-CH	.43	3	0.540	19.5			
11.00	13.71	.00	.01	SENSITIVE FINE GRAINED	OL-CH	.33	3	0.686	19.5			
11.20	14.72	.29	1.99	SILTY CLAY TO CLAY	CL-CH	.45	4	0.732	19.5			
11.40	15.75	.23	1.48	CLAYEY SILT TO SILTY CLAY	MH-CL	.51	3	0.782	19.5			
11.60	13.35	.16	1.19	SENSITIVE FINE GRAINED	OL-CH	.35	3	0.667	19.5			
11.80	20.83	.37	1.77	CLAYEY SILT TO SILTY CLAY	MH-CL	.53	4	1.041	19.5			
12.00	18.47	.25	1.36	CLAYEY SILT TO SILTY CLAY	MH-CL	.54	4	0.919	19.5			
12.20	29.05	.80	2.75	CLAYEY SILT TO SILTY CLAY	MH-CL	.55	6	1.462	19.5			
12.40	18.87	.25	1.30	CLAYEY SILT TO SILTY CLAY	MH-CL	.56	4	0.939	19.5			
12.60	16.78	.35	2.10	CLAYEY SILT TO SILTY CLAY	MH-CL	.57	4	0.831	19.5			
12.80	13.86	.25	1.83	SILTY CLAY TO CLAY	CL-CH	.51	4	0.685	19.5			
13.00	11.31	.15	1.30	SENSITIVE FINE GRAINED	OL-CH	.39	2	0.560	19.5			
13.20	10.61	.15	1.37	SENSITIVE FINE GRAINED	OL-CH	.40	2	0.524	19.5			
13.40	10.61	.15	1.41	SENSITIVE FINE GRAINED	OL-CH	.40	2	0.524	19.5			
13.60	16.44	.16	.99	CLAYEY SILT TO SILTY CLAY	MH-CL	.61	3	0.812	19.5			
13.80	13.39	.12	.90	SENSITIVE FINE GRAINED	OL-CH	.41	3	0.666	19.5			
14.00	9.58	.11	1.18	SENSITIVE FINE GRAINED	OL-CH	.42	2	0.470	19.5			
14.20	15.35	.22	1.44	CLAYEY SILT TO SILTY CLAY	MH-CL	.64	3	0.754	19.5			
14.40	9.60	.09	.97	SENSITIVE FINE GRAINED	OL-CH	.43	2	0.470	19.5			
14.60	11.64	.00	.02	SENSITIVE FINE GRAINED	OL-CH	.44	2	0.574	19.5			
14.80	58.09	.00	.00	SILTY SAND TO SANDY SILT	SM-ML	.81	9			51	35	.18
15.00	61.35	.00	.00	SILTY SAND TO SANDY SILT	SM-ML	.82	9			53	35	.18
15.20	49.04	.01	.03	SILTY SAND TO SANDY SILT	SM-ML	.84	7			45	34	.16
15.40	20.58	.09	.43	SANDY SILT TO CLAYEY SILT	ML-MH	.77	3	1.016	19.5	20	31	.11
15.60	9.96	.26	2.59	CLAY	CH	.62	4	0.479	19.5			
15.80	10.60	.42	3.93	CLAY	CH	.63	5	0.511	19.5			
16.00	61.11	1.17	1.92	SANDY SILT TO CLAYEY SILT	ML-MH	.80	11	3.093	19.5	54	36	.19
16.20	48.49	.00	.01	SILTY SAND TO SANDY SILT	SM-ML	.89	7			43	34	.15
16.40	196.11	1.37	.70	SAND TO SILTY SAND	SM-SP	.98	23			87	41	.50+
16.60	523.18	1.34	.26	GRAVELLY SAND TO SAND	SW-GW	1.08	41			100+	46	.50+
16.80	447.06	3.12	.70	SAND	SW-SP	1.09	42			100+	44	.50+
17.00	153.61	.94	.61	SAND TO SILTY SAND	SM-SP	1.02	18			79	40	.50+
17.20	199.38	1.34	.67	SAND TO SILTY SAND	SM-SP	1.03	23			86	41	.50+
17.40	122.24	.87	.71	SAND TO SILTY SAND	SM-SP	1.04	14			69	39	.48
17.60	65.03	.84	1.29	SILTY SAND TO SANDY SILT	SM-ML	.97	10			50	35	.18
17.80	65.04	1.26	1.94	SANDY SILT TO CLAYEY SILT	ML-MH	.89	12	3.290	19.5	53	35	.18
18.00	69.45	1.42	2.05	SANDY SILT TO CLAYEY SILT	ML-MH	.90	13	3.515	19.5	55	36	.19

18.20	79.25	1.73	2.19	SANDY SILT TO CLAYEY SILT	ML-MH	.91	15	4.017	19.5	60	36	.22
18.40	86.91	1.52	1.75	SANDY SILT TO CLAYEY SILT	ML-MH	.92	16	4.410	19.5	62	37	.25
18.60	95.16	2.07	2.17	SANDY SILT TO CLAYEY SILT	ML-MH	.93	18	4.832	19.5	64	37	.29
18.80	92.44	2.18	2.36	SANDY SILT TO CLAYEY SILT	ML-MH	.94	17	4.692	19.5	63	37	.27
19.00	84.61	1.66	1.96	SANDY SILT TO CLAYEY SILT	ML-MH	.95	16	4.290	19.5	61	37	.23
19.20	77.02	1.27	1.65	SANDY SILT TO CLAYEY SILT	ML-MH	.96	14	3.901	19.5	57	36	.20
19.40	79.07	1.44	1.82	SANDY SILT TO CLAYEY SILT	ML-MH	.97	15	4.005	19.5	58	36	.21
19.60	86.55	1.88	2.18	SANDY SILT TO CLAYEY SILT	ML-MH	.98	16	4.388	19.5	61	37	.23
19.80	91.76	2.00	2.18	SANDY SILT TO CLAYEY SILT	ML-MH	.99	17	4.655	19.5	62	37	.25
20.00	68.87	.70	1.02	SILTY SAND TO SANDY SILT	SM-ML	1.10	10			49	35	.17
20.20	65.35	1.11	1.70	SANDY SILT TO CLAYEY SILT	ML-MH	1.01	12	3.299	19.5	49	35	.17

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 10
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	32.18	.69	2.15	CLAYEY SILT TO SILTY CLAY	MH-CL	.01	7	1.650	19.5			
.40	29.45	.47	1.61	SANDY SILT TO CLAYEY SILT	ML-MH	.02	5	1.509	19.5	71	38	.50+
.60	22.60	.51	2.24	CLAYEY SILT TO SILTY CLAY	MH-CL	.03	5	1.157	19.5			
.80	65.03	.97	1.49	SANDY SILT TO CLAYEY SILT	ML-MH	.04	12	3.333	19.5	100+	42	.50+
1.00	92.10	1.46	1.59	SILTY SAND TO SANDY SILT	SM-ML	.05	14			100+	42	.50+
1.20	70.19	1.36	1.94	SANDY SILT TO CLAYEY SILT	ML-MH	.06	13	3.596	19.5	100+	42	.50+
1.40	25.00	.38	1.53	CLAYEY SILT TO SILTY CLAY	MH-CL	.06	5	1.279	19.5			
1.60	22.25	.20	.88	SANDY SILT TO CLAYEY SILT	ML-MH	.08	4	1.137	19.5	62	37	.50+
1.80	18.91	.36	1.89	CLAYEY SILT TO SILTY CLAY	MH-CL	.08	4	0.966	19.5			
2.00	15.58	.32	2.03	SILTY CLAY TO CLAY	CL-CH	.08	4	0.795	19.5			
2.20	20.03	.53	2.63	SILTY CLAY TO CLAY	CL-CH	.09	6	1.023	19.5			
2.40	19.51	.30	1.52	CLAYEY SILT TO SILTY CLAY	MH-CL	.11	4	0.995	19.5			
2.60	17.12	.26	1.52	CLAYEY SILT TO SILTY CLAY	MH-CL	.12	4	0.872	19.5			
2.80	43.10	.94	2.18	SANDY SILT TO CLAYEY SILT	ML-MH	.14	8	2.203	19.5	71	38	.50+
3.00	35.60	1.09	3.05	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	8	1.818	19.5			
3.20	35.77	.88	2.46	CLAYEY SILT TO SILTY CLAY	MH-CL	.14	8	1.827	19.5			
3.40	47.23	.27	.57	SILTY SAND TO SANDY SILT	SM-ML	.19	7			71	38	.50+
3.60	23.63	.43	1.83	CLAYEY SILT TO SILTY CLAY	MH-CL	.16	5	1.204	19.5			
3.80	14.39	.24	1.67	CLAYEY SILT TO SILTY CLAY	MH-CL	.17	3	0.729	19.5			
4.00	15.74	.25	1.56	CLAYEY SILT TO SILTY CLAY	MH-CL	.18	3	0.798	19.5			
4.20	16.78	.25	1.50	CLAYEY SILT TO SILTY CLAY	MH-CL	.19	4	0.851	19.5			
4.40	24.33	.06	.26	SANDY SILT TO CLAYEY SILT	ML-MH	.22	4	1.236	19.5	54	36	.18
4.60	15.92	.18	1.10	CLAYEY SILT TO SILTY CLAY	MH-CL	.21	3	0.806	19.5			
4.80	17.46	.22	1.29	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	4	0.884	19.5			
5.00	17.14	.20	1.17	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	4	0.868	19.5			
5.20	41.70	.73	1.76	SANDY SILT TO CLAYEY SILT	ML-MH	.26	7	2.125	19.5	64	37	.35
5.40	50.32	1.31	2.61	CLAYEY SILT TO SILTY CLAY	MH-CL	.24	12	2.568	19.5			
5.60	52.89	1.38	2.61	SANDY SILT TO CLAYEY SILT	ML-MH	.28	10	2.698	19.5	69	38	.50+
5.80	56.47	1.03	1.83	SANDY SILT TO CLAYEY SILT	ML-MH	.29	10	2.881	19.5	70	38	.50+
6.00	37.35	.15	.40	SANDY SILT TO CLAYEY SILT	ML-MH	.30	7	1.900	19.5	60	37	.23
6.20	25.35	.40	1.59	CLAYEY SILT TO SILTY CLAY	MH-CL	.28	6	1.286	19.5			
6.40	23.44	.25	1.06	SANDY SILT TO CLAYEY SILT	ML-MH	.32	4	1.186	19.5	47	35	.15
6.60	22.95	.12	.53	SANDY SILT TO CLAYEY SILT	ML-MH	.33	4	1.160	19.5	46	34	.15
6.80	15.75	.22	1.38	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	3	0.792	19.5			
7.00	14.73	.16	1.06	SENSITIVE FINE GRAINED	OL-CH	.21	3	0.745	19.5			
7.20	9.94	.08	.78	SENSITIVE FINE GRAINED	OL-CH	.22	2	0.498	19.5			
7.40	7.20	.08	1.16	SENSITIVE FINE GRAINED	OL-CH	.22	1	0.358	19.5			
7.60	6.50	.04	.63	SENSITIVE FINE GRAINED	OL-CH	.23	1	0.322	19.5			

7.80	5.14	.09	1.66	SENSITIVE FINE GRAINED	OL-CH	.23	1	0.252	19.5			
8.00	6.16	.10	1.63	SENSITIVE FINE GRAINED	OL-CH	.24	1	0.304	19.5			
8.20	11.95	.45	3.75	CLAY	CH	.33	5	0.596	19.5			
8.40	19.84	.35	1.78	CLAYEY SILT TO SILTY CLAY	MH-CL	.38	4	0.998	19.5			
8.60	22.93	.45	1.94	CLAYEY SILT TO SILTY CLAY	MH-CL	.39	5	1.156	19.5			
8.80	27.38	1.03	3.76	SILTY CLAY TO CLAY	CL-CH	.35	8	1.386	19.5			
9.00	32.69	.88	2.70	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	7	1.656	19.5			
9.20	28.43	.46	1.63	CLAYEY SILT TO SILTY CLAY	MH-CL	.41	6	1.437	19.5			
9.40	30.46	.64	2.11	CLAYEY SILT TO SILTY CLAY	MH-CL	.42	7	1.541	19.5			
9.60	15.77	.34	2.14	SILTY CLAY TO CLAY	CL-CH	.38	5	0.789	19.5			
9.80	12.32	.43	3.47	CLAY	CH	.39	5	0.612	19.5			
10.00	25.62	1.20	4.70	CLAY	CH	.40	12	1.293	19.5			
10.20	31.22	1.00	3.20	SILTY CLAY TO CLAY	CL-CH	.41	9	1.580	19.5			
10.40	29.84	.87	2.91	CLAYEY SILT TO SILTY CLAY	MH-CL	.47	7	1.506	19.5			
10.60	32.50	.84	2.58	CLAYEY SILT TO SILTY CLAY	MH-CL	.48	7	1.642	19.5			
10.80	37.29	.97	2.61	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	8	1.887	19.5			
11.00	38.79	.65	1.68	SANDY SILT TO CLAYEY SILT	ML-MH	.55	7	1.961	19.5	48	35	.16
11.20	37.66	1.19	3.17	CLAYEY SILT TO SILTY CLAY	MH-CL	.50	9	1.906	19.5			
11.40	40.72	1.14	2.79	CLAYEY SILT TO SILTY CLAY	MH-CL	.51	9	2.062	19.5			
11.60	29.77	1.11	3.71	SILTY CLAY TO CLAY	CL-CH	.46	9	1.503	19.5			
11.80	33.90	1.48	4.37	CLAY	CH	.47	16	1.714	19.5			
12.00	29.62	1.11	3.74	SILTY CLAY TO CLAY	CL-CH	.48	9	1.494	19.5			
12.20	28.76	1.37	4.77	CLAY	CH	.49	13	1.450	19.5			
12.40	22.59	.98	4.32	CLAY	CH	.50	10	1.133	19.5			
12.60	40.38	1.58	3.90	SILTY CLAY TO CLAY	CL-CH	.50	12	2.045	19.5			
12.80	45.49	1.24	2.72	CLAYEY SILT TO SILTY CLAY	MH-CL	.58	10	2.303	19.5			
13.00	38.39	1.00	2.60	CLAYEY SILT TO SILTY CLAY	MH-CL	.58	9	1.939	19.5			
13.20	45.49	1.09	2.40	SANDY SILT TO CLAYEY SILT	ML-MH	.66	8	2.299	19.5	48	35	.17
13.40	38.19	1.30	3.40	SILTY CLAY TO CLAY	CL-CH	.54	12	1.931	19.5			
13.60	43.06	1.09	2.53	CLAYEY SILT TO SILTY CLAY	MH-CL	.61	10	2.177	19.5			
13.80	41.85	1.47	3.52	SILTY CLAY TO CLAY	CL-CH	.55	13	2.118	19.5			
14.00	39.38	.64	1.62	SANDY SILT TO CLAYEY SILT	ML-MH	.70	7	1.984	19.5	43	34	.15
14.20	39.02	.27	.69	SANDY SILT TO CLAYEY SILT	ML-MH	.71	7	1.965	19.5	42	34	.15
14.40	32.30	1.14	3.54	SILTY CLAY TO CLAY	CL-CH	.58	10	1.627	19.5			
14.60	29.76	.67	2.25	CLAYEY SILT TO SILTY CLAY	MH-CL	.66	7	1.492	19.5			
14.80	62.24	2.24	3.61	CLAYEY SILT TO SILTY CLAY	MH-CL	.67	14	3.157	19.5			
15.00	121.65	.92	.76	SAND TO SILTY SAND	SM-SP	.90	14			73	39	.50+
15.20	96.20	.77	.80	SILTY SAND TO SANDY SILT	SM-ML	.84	15			67	37	.37
15.40	161.39	1.74	1.08	SAND TO SILTY SAND	SM-SP	.92	19			82	40	.50+
15.60	222.40	1.96	.88	SAND TO SILTY SAND	SM-SP	.94	26			93	42	.50+
15.80	277.63	2.34	.84	SAND	SW-SP	1.03	26			99	44	.50+
16.00	460.37	2.55	.55	SAND	SW-SP	1.04	44			100+	44	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 11
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	62.14	1.47	2.37	SANDY SILT TO CLAYEY SILT	ML-MH	.01	11	3.186	19.5	100+	42	.50+
.40	65.05	1.93	2.97	CLAYEY SILT TO SILTY CLAY	MH-CL	.02	15	3.335	19.5			
.60	65.39	1.96	3.00	CLAYEY SILT TO SILTY CLAY	MH-CL	.03	15	3.352	19.5			
.80	70.52	2.13	3.02	CLAYEY SILT TO SILTY CLAY	MH-CL	.04	16	3.614	19.5			
1.00	73.60	2.41	3.27	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	17	3.772	19.5			
1.20	85.93	2.59	3.02	SANDY SILT TO CLAYEY SILT	ML-MH	.06	16	4.404	19.5	100+	42	.50+
1.40	105.78	3.43	3.24	SANDY SILT TO CLAYEY SILT	ML-MH	.07	20	5.421	19.5	100+	42	.50+
1.60	114.00	3.06	2.69	SANDY SILT TO CLAYEY SILT	ML-MH	.08	21	5.842	19.5	100+	42	.50+
1.80	116.40	2.81	2.42	SANDY SILT TO CLAYEY SILT	ML-MH	.09	22	5.965	19.5	100+	42	.50+
2.00	123.24	2.25	1.82	SILTY SAND TO SANDY SILT	SM-ML	.11	19			100+	42	.50+
2.20	124.28	1.83	1.47	SILTY SAND TO SANDY SILT	SM-ML	.12	19			100+	42	.50+
2.40	154.03	2.21	1.44	SILTY SAND TO SANDY SILT	SM-ML	.13	24			100+	42	.50+
2.60	143.46	1.81	1.26	SILTY SAND TO SANDY SILT	SM-ML	.14	22			100+	42	.50+
2.80	165.32	3.14	1.90	SILTY SAND TO SANDY SILT	SM-ML	.15	26			100+	42	.50+
3.00	260.78	4.59	1.76	SILTY SAND TO SANDY SILT	SM-ML	.17	41			100+	42	.50+
3.20	285.29	6.15	2.15	SILTY SAND TO SANDY SILT	SM-ML	.18	45			100+	42	.50+
3.40	401.48	9.54	2.38	SILTY SAND TO SANDY SILT	SM-ML	.19	64			100+	42	.50+
3.60	315.27	7.49	2.38	SILTY SAND TO SANDY SILT	SM-ML	.20	50			100+	42	.50+
3.80	285.94	3.29	1.15	SAND TO SILTY SAND	SM-SP	.23	34			100+	42	.50+
4.00	254.38	3.68	1.45	SAND TO SILTY SAND	SM-SP	.24	30			100+	42	.50+
4.20	237.91	3.83	1.61	SAND TO SILTY SAND	SM-SP	.25	28			100+	42	.50+
4.40	124.72	2.45	1.97	SILTY SAND TO SANDY SILT	SM-ML	.24	19			100+	42	.50+
4.60	82.56	1.59	1.93	SANDY SILT TO CLAYEY SILT	ML-MH	.23	15	4.222	19.5	87	40	.50+
4.80	90.70	1.97	2.17	SANDY SILT TO CLAYEY SILT	ML-MH	.24	17	4.639	19.5	90	41	.50+
5.00	70.53	1.61	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.25	13	3.604	19.5	80	39	.50+
5.20	65.07	1.52	2.34	SANDY SILT TO CLAYEY SILT	ML-MH	.26	12	3.324	19.5	76	39	.50+
5.40	62.31	1.25	2.01	SANDY SILT TO CLAYEY SILT	ML-MH	.27	11	3.182	19.5	74	38	.50+
5.60	56.84	1.39	2.44	SANDY SILT TO CLAYEY SILT	ML-MH	.28	10	2.901	19.5	71	38	.50+
5.80	59.90	1.18	1.97	SANDY SILT TO CLAYEY SILT	ML-MH	.29	11	3.057	19.5	72	38	.50+
6.00	40.94	.79	1.92	SANDY SILT TO CLAYEY SILT	ML-MH	.30	7	2.084	19.5	62	37	.26
6.20	33.91	.56	1.64	SANDY SILT TO CLAYEY SILT	ML-MH	.31	6	1.723	19.5	57	36	.20
6.40	27.06	.79	2.93	CLAYEY SILT TO SILTY CLAY	MH-CL	.29	6	1.373	19.5			
6.60	23.63	.97	4.12	CLAY	CH	.26	11	1.198	19.5			
6.80	19.17	.54	2.82	SILTY CLAY TO CLAY	CL-CH	.27	6	0.969	19.5			
7.00	32.49	.71	2.19	CLAYEY SILT TO SILTY CLAY	MH-CL	.31	7	1.650	19.5			
7.20	37.99	.85	2.23	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	9	1.932	19.5			
7.40	46.39	1.06	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.37	8	2.360	19.5	62	37	.25
7.60	49.97	.89	1.77	SANDY SILT TO CLAYEY SILT	ML-MH	.38	9	2.543	19.5	63	37	.28

7.80	52.38	1.30	2.48	SANDY SILT TO CLAYEY SILT	ML-MH	.39	10	2.666	19.5	64	37	.30
8.00	46.22	1.01	2.18	SANDY SILT TO CLAYEY SILT	ML-MH	.40	8	2.350	19.5	61	37	.23
8.20	148.97	.84	.56	SAND TO SILTY SAND	SM-SP	.49	17			94	42	.50+
8.40	54.26	1.24	2.28	SANDY SILT TO CLAYEY SILT	ML-MH	.42	10	2.761	19.5	64	37	.29
8.60	189.24	3.99	2.11	SILTY SAND TO SANDY SILT	SM-ML	.47	30			100+	42	.50+
8.80	114.52	2.86	2.49	SANDY SILT TO CLAYEY SILT	ML-MH	.44	21	5.850	19.5	86	40	.50+
9.00	257.71	1.03	.40	SAND	SW-SP	.58	24			100+	44	.50+
9.20	281.44	5.03	1.79	SILTY SAND TO SANDY SILT	SM-ML	.51	44			100+	42	.50+
9.40	63.46	.86	1.36	SANDY SILT TO CLAYEY SILT	ML-MH	.47	12	3.230	19.5	66	37	.35
9.60	54.12	.59	1.09	SANDY SILT TO CLAYEY SILT	ML-MH	.48	10	2.751	19.5	62	37	.24
9.80	170.90	2.30	1.35	SILTY SAND TO SANDY SILT	SM-ML	.54	27			97	42	.50+
10.00	54.78	1.86	3.40	CLAYEY SILT TO SILTY CLAY	MH-CL	.45	13	2.786	19.5			
10.20	47.25	1.00	2.12	SANDY SILT TO CLAYEY SILT	ML-MH	.51	9	2.397	19.5	57	36	.19
10.40	47.76	.88	1.84	SANDY SILT TO CLAYEY SILT	ML-MH	.52	9	2.423	19.5	57	36	.19
10.60	42.47	.95	2.24	SANDY SILT TO CLAYEY SILT	ML-MH	.53	8	2.151	19.5	52	35	.18
10.80	141.99	2.78	1.96	SILTY SAND TO SANDY SILT	SM-ML	.59	22			87	40	.50+
11.00	92.84	2.98	3.21	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	22	4.736	19.5			
11.20	75.67	1.53	2.02	SANDY SILT TO CLAYEY SILT	ML-MH	.56	14	3.852	19.5	68	38	.41
11.40	57.85	.58	1.00	SILTY SAND TO SANDY SILT	SM-ML	.63	9			59	36	.21
11.60	30.96	.84	2.71	CLAYEY SILT TO SILTY CLAY	MH-CL	.52	7	1.561	19.5			
11.80	44.40	.35	.79	SANDY SILT TO CLAYEY SILT	ML-MH	.59	8	2.247	19.5	51	35	.17
12.00	72.18	.97	1.35	SILTY SAND TO SANDY SILT	SM-ML	.66	11			63	37	.27
12.20	83.86	1.31	1.56	SILTY SAND TO SANDY SILT	SM-ML	.67	13			67	37	.38
12.40	74.58	1.06	1.42	SILTY SAND TO SANDY SILT	SM-ML	.68	11			64	37	.28
12.60	86.64	1.89	2.19	SANDY SILT TO CLAYEY SILT	ML-MH	.63	16	4.411	19.5	69	38	.49
12.80	89.59	1.77	1.98	SANDY SILT TO CLAYEY SILT	ML-MH	.64	17	4.562	19.5	70	38	.50+
13.00	78.44	1.81	2.30	SANDY SILT TO CLAYEY SILT	ML-MH	.65	15	3.989	19.5	66	37	.34
13.20	73.61	1.86	2.53	SANDY SILT TO CLAYEY SILT	ML-MH	.66	14	3.741	19.5	64	37	.28
13.40	210.63	4.57	2.17	SILTY SAND TO SANDY SILT	SM-ML	.74	33			97	42	.50+
13.60	114.86	2.20	1.92	SILTY SAND TO SANDY SILT	SM-ML	.75	18			75	39	.50+
13.80	168.77	2.28	1.35	SILTY SAND TO SANDY SILT	SM-ML	.76	26			88	40	.50+
14.00	429.35	8.28	1.93	SAND TO SILTY SAND	SM-SP	.84	51			100+	42	.50+
14.20	341.85	9.06	2.65	SILTY SAND TO SANDY SILT	SM-ML	.78	54			100+	42	.50+
14.40	241.30	9.00	3.73	SANDY SILT TO CLAYEY SILT	ML-MH	.72	46	12.337	19.5	100+	42	.50+
14.60	65.45	2.26	3.46	CLAYEY SILT TO SILTY CLAY	MH-CL	.66	15	3.323	19.5			
14.80	188.12	2.38	1.26	SAND TO SILTY SAND	SM-SP	.89	22			88	41	.50+
15.00	82.70	.36	.44	SAND TO SILTY SAND	SM-SP	.90	9			61	38	.24
15.20	56.14	.34	.61	SILTY SAND TO SANDY SILT	SM-ML	.84	8			49	35	.17
15.40	225.62	2.00	.89	SAND TO SILTY SAND	SM-SP	.92	27			94	42	.50+
15.60	217.22	1.32	.61	SAND	SW-SP	1.01	20			90	43	.50+
15.80	122.20	1.35	1.11	SILTY SAND TO SANDY SILT	SM-ML	.87	19			74	38	.50+
16.00	56.37	.27	.47	SILTY SAND TO SANDY SILT	SM-ML	.88	8			48	35	.17
16.20	34.30	.00	.00	SILTY SAND TO SANDY SILT	SM-ML	.89	5			31	32	.13
16.40	32.16	.09	.27	SANDY SILT TO CLAYEY SILT	ML-MH	.82	6	1.607	19.5	31	32	.13
16.60	31.34	.28	.89	SANDY SILT TO CLAYEY SILT	ML-MH	.83	6	1.565	19.5	30	32	.13
16.80	14.80	.12	.80	SENSITIVE FINE GRAINED	OL-CH	.50	3	0.733	19.5			
17.00	9.25	.24	2.56	CLAY	CH	.68	4	0.439	19.5			
17.20	77.98	.21	.26	SAND TO SILTY SAND	SM-SP	1.03	9			55	37	.20
17.40	27.49	.00	.00	SANDY SILT TO CLAYEY SILT	ML-MH	.87	5	1.365	19.5	25	32	.12
17.60	16.51	.02	.09	SENSITIVE FINE GRAINED	OL-CH	.53	3	0.819	19.5			
17.80	103.79	.00	.00	SAND TO SILTY SAND	SM-SP	1.07	12			64	38	.29
18.00	93.42	.00	.00	SAND TO SILTY SAND	SM-SP	1.08	11			61	38	.23

18.20	102.61	.40	.39	SAND TO SILTY SAND	SM-SP	1.09	12	63	38	.27
18.40	98.59	.76	.77	SAND TO SILTY SAND	SM-SP	1.10	11	62	38	.25
18.60	156.33	1.90	1.21	SAND TO SILTY SAND	SM-SP	1.12	18	77	40	.50+
18.80	116.40	.07	.06	SAND TO SILTY SAND	SM-SP	1.13	13	66	39	.35
19.00	235.11	1.10	.47	SAND	SW-SP	1.23	22	88	43	.50+
19.20	297.60	1.28	.43	SAND	SW-SP	1.25	28	96	44	.50+
19.40	132.14	1.22	.92	SAND TO SILTY SAND	SM-SP	1.16	15	69	39	.48
19.60	295.95	.58	.19	SAND	SW-SP	1.27	28	95	44	.50+
19.80	367.14	.75	.20	SAND	SW-SP	1.29	35	100+	44	.50+
20.00	389.25	1.31	.34	SAND	SW-SP	1.30	37	100+	44	.50+
20.20	360.95	2.27	.63	SAND	SW-SP	1.31	34	100+	44	.50+
20.40	200.75	1.44	.72	SAND TO SILTY SAND	SM-SP	1.22	24	83	40	.50+
20.60	173.26	1.39	.80	SAND TO SILTY SAND	SM-SP	1.24	20	78	40	.50+
20.80	236.52	3.34	1.41	SAND TO SILTY SAND	SM-SP	1.25	28	87	41	.50+
21.00	267.68	1.74	.65	SAND	SW-SP	1.37	25	90	43	.50+
21.20	348.68	1.02	.29	SAND	SW-SP	1.38	33	100+	44	.50+
21.40	351.59	.01	.00	SAND	SW-SP	1.39	33	100+	44	.50+
21.60	276.86	1.70	.61	SAND	SW-SP	1.40	26	90	43	.50+
21.80	236.12	1.02	.43	SAND	SW-SP	1.42	22	85	43	.50+
22.00	198.67	1.71	.86	SAND TO SILTY SAND	SM-SP	1.32	23	81	40	.50+
22.20	162.94	2.74	1.68	SILTY SAND TO SANDY SILT	SM-ML	1.22	26	76	39	.50+
22.40	188.54	2.43	1.29	SAND TO SILTY SAND	SM-SP	1.34	22	79	40	.50+
22.60	245.47	2.85	1.16	SAND TO SILTY SAND	SM-SP	1.36	29	87	41	.50+
22.80	308.83	2.84	.92	SAND	SW-SP	1.48	29	93	44	.50+
23.00	381.97	1.41	.37	SAND	SW-SP	1.50	36	100+	44	.50+
23.20	398.76	2.38	.60	SAND	SW-SP	1.51	38	100+	44	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 12
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	40.05	.69	1.73	SANDY SILT TO CLAYEY SILT	ML-MH	.01	7	2.053	19.5	79	39	.50+
.40	16.78	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.01	4	0.860	19.5			
.60	17.80	.30	1.66	CLAYEY SILT TO SILTY CLAY	MH-CL	.03	4	0.911	19.5			
.80	22.60	.23	1.01	SANDY SILT TO CLAYEY SILT	ML-MH	.04	4	1.157	19.5	64	37	.50+
1.00	58.18	.88	1.51	SANDY SILT TO CLAYEY SILT	ML-MH	.05	11	2.981	19.5	92	41	.50+
1.20	37.15	.82	2.21	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	8	1.903	19.5			
1.40	34.75	.65	1.88	SANDY SILT TO CLAYEY SILT	ML-MH	.07	6	1.778	19.5	71	38	.50+
1.60	35.26	.75	2.13	CLAYEY SILT TO SILTY CLAY	MH-CL	.07	8	1.805	19.5			
1.80	32.35	.57	1.77	SANDY SILT TO CLAYEY SILT	ML-MH	.09	6	1.654	19.5	68	38	.50+
2.00	30.81	.15	.48	SANDY SILT TO CLAYEY SILT	ML-MH	.10	5	1.575	19.5	66	37	.50+
2.20	19.52	.34	1.73	CLAYEY SILT TO SILTY CLAY	MH-CL	.10	4	0.996	19.5			
2.40	15.41	.22	1.41	CLAYEY SILT TO SILTY CLAY	MH-CL	.11	3	0.785	19.5			
2.60	18.48	.33	1.79	CLAYEY SILT TO SILTY CLAY	MH-CL	.12	4	0.942	19.5			
2.80	15.75	.19	1.19	CLAYEY SILT TO SILTY CLAY	MH-CL	.13	3	0.801	19.5			
3.00	26.69	.30	1.13	SANDY SILT TO CLAYEY SILT	ML-MH	.15	5	1.361	19.5	61	37	.30
3.20	18.87	.11	.61	SANDY SILT TO CLAYEY SILT	ML-MH	.16	3	0.959	19.5	53	35	.18
3.40	21.56	.31	1.42	CLAYEY SILT TO SILTY CLAY	MH-CL	.15	5	1.098	19.5			
3.60	23.62	.11	.48	SANDY SILT TO CLAYEY SILT	ML-MH	.18	4	1.202	19.5	56	36	.20
3.80	22.60	.32	1.40	CLAYEY SILT TO SILTY CLAY	MH-CL	.17	5	1.150	19.5			
4.00	18.49	.09	.51	SENSITIVE FINE GRAINED	OL-CH	.12	4	0.942	19.5			
4.20	17.46	.13	.76	SENSITIVE FINE GRAINED	OL-CH	.13	4	0.889	19.5			
4.40	14.04	.20	1.44	CLAYEY SILT TO SILTY CLAY	MH-CL	.20	3	0.710	19.5			
4.60	13.01	.16	1.20	SENSITIVE FINE GRAINED	OL-CH	.14	3	0.660	19.5			
4.80	16.77	.28	1.70	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	4	0.849	19.5			
5.00	18.47	.31	1.70	CLAYEY SILT TO SILTY CLAY	MH-CL	.22	4	0.936	19.5			
5.20	18.32	.37	2.00	CLAYEY SILT TO SILTY CLAY	MH-CL	.23	4	0.928	19.5			
5.40	15.75	.14	.86	SENSITIVE FINE GRAINED	OL-CH	.16	3	0.799	19.5			
5.60	20.53	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.17	4	1.044	19.5			
5.80	53.11	.00	.00	SILTY SAND TO SANDY SILT	SM-ML	.32	8			67	37	.47
6.00	31.93	.37	1.15	SANDY SILT TO CLAYEY SILT	ML-MH	.30	6	1.622	19.5	56	36	.19
6.20	21.23	.11	.54	SANDY SILT TO CLAYEY SILT	ML-MH	.31	4	1.073	19.5	45	34	.15
6.40	13.02	.00	.00	SENSITIVE FINE GRAINED	OL-CH	.19	3	0.658	19.5			
6.60	13.69	.10	.70	SENSITIVE FINE GRAINED	OL-CH	.20	3	0.692	19.5			
6.80	11.98	.24	1.99	SILTY CLAY TO CLAY	CL-CH	.27	3	0.601	19.5			
7.00	17.47	.43	2.47	SILTY CLAY TO CLAY	CL-CH	.28	5	0.882	19.5			
7.20	32.44	.53	1.64	SANDY SILT TO CLAYEY SILT	ML-MH	.36	6	1.645	19.5	53	35	.18
7.40	18.49	.26	1.42	CLAYEY SILT TO SILTY CLAY	MH-CL	.33	4	0.931	19.5			
7.60	15.07	.12	.82	SENSITIVE FINE GRAINED	OL-CH	.23	3	0.761	19.5			

7.80	12.67	.10	.80	SENSITIVE FINE GRAINED	OL-CH	.23	3	0.638	19.5			
8.00	18.79	.29	1.53	CLAYEY SILT TO SILTY CLAY	MH-CL	.36	4	0.945	19.5			
8.20	22.59	.62	2.73	SILTY CLAY TO CLAY	CL-CH	.33	7	1.142	19.5			
8.40	21.91	.53	2.42	CLAYEY SILT TO SILTY CLAY	MH-CL	.38	5	1.104	19.5			
8.60	16.77	.18	1.04	CLAYEY SILT TO SILTY CLAY	MH-CL	.39	4	0.840	19.5			
8.80	17.28	.15	.87	CLAYEY SILT TO SILTY CLAY	MH-CL	.40	4	0.866	19.5			
9.00	11.64	.00	.01	SENSITIVE FINE GRAINED	OL-CH	.27	2	0.583	19.5			
9.20	11.90	.19	1.60	SILTY CLAY TO CLAY	CL-CH	.37	3	0.591	19.5			
9.40	13.06	.21	1.62	SILTY CLAY TO CLAY	CL-CH	.38	4	0.650	19.5			
9.60	15.05	.06	.43	SENSITIVE FINE GRAINED	OL-CH	.29	3	0.757	19.5			
9.80	11.30	.07	.60	SENSITIVE FINE GRAINED	OL-CH	.29	2	0.565	19.5			
10.00	14.01	.14	1.00	SENSITIVE FINE GRAINED	OL-CH	.30	3	0.703	19.5			
10.20	24.95	.27	1.10	SANDY SILT TO CLAYEY SILT	ML-MH	.51	4	1.253	19.5	38	33	.13
10.40	22.93	.11	.47	SANDY SILT TO CLAYEY SILT	ML-MH	.52	4	1.149	19.5	34	33	.13
10.60	15.07	.08	.55	SENSITIVE FINE GRAINED	OL-CH	.32	3	0.756	19.5			
10.80	15.40	.22	1.43	CLAYEY SILT TO SILTY CLAY	MH-CL	.49	3	0.765	19.5			
11.00	16.61	.13	.78	SENSITIVE FINE GRAINED	OL-CH	.33	3	0.835	19.5			
11.20	17.45	.45	2.56	SILTY CLAY TO CLAY	CL-CH	.45	5	0.872	19.5			
11.40	25.64	.68	2.66	CLAYEY SILT TO SILTY CLAY	MH-CL	.51	6	1.289	19.5			
11.60	33.85	.80	2.36	CLAYEY SILT TO SILTY CLAY	MH-CL	.52	8	1.709	19.5			
11.80	35.64	1.45	4.07	SILTY CLAY TO CLAY	CL-CH	.47	11	1.804	19.5			
12.00	38.66	.96	2.49	CLAYEY SILT TO SILTY CLAY	MH-CL	.54	9	1.955	19.5			
12.20	54.35	1.98	3.64	CLAYEY SILT TO SILTY CLAY	MH-CL	.55	13	2.759	19.5			
12.40	56.15	1.76	3.14	CLAYEY SILT TO SILTY CLAY	MH-CL	.56	13	2.851	19.5			
12.60	61.42	1.52	2.47	SANDY SILT TO CLAYEY SILT	ML-MH	.63	11	3.117	19.5	60	37	.22
12.80	82.75	2.01	2.43	SANDY SILT TO CLAYEY SILT	ML-MH	.64	15	4.211	19.5	68	38	.41
13.00	72.60	1.31	1.80	SANDY SILT TO CLAYEY SILT	ML-MH	.65	13	3.690	19.5	64	37	.28
13.20	87.58	2.00	2.29	SANDY SILT TO CLAYEY SILT	ML-MH	.66	16	4.457	19.5	69	38	.46
13.40	72.67	1.52	2.09	SANDY SILT TO CLAYEY SILT	ML-MH	.67	13	3.692	19.5	63	37	.27
13.60	78.02	1.55	1.98	SANDY SILT TO CLAYEY SILT	ML-MH	.68	14	3.966	19.5	65	37	.31
13.80	82.31	2.19	2.66	SANDY SILT TO CLAYEY SILT	ML-MH	.69	15	4.186	19.5	66	37	.34
14.00	97.61	2.55	2.61	SANDY SILT TO CLAYEY SILT	ML-MH	.70	18	4.970	19.5	71	38	.50+
14.20	179.29	1.93	1.07	SAND TO SILTY SAND	SM-SP	.85	21			87	41	.50+
14.40	134.43	1.19	.88	SAND TO SILTY SAND	SM-SP	.86	16			78	40	.50+
14.60	67.49	.90	1.33	SILTY SAND TO SANDY SILT	SM-ML	.80	10			57	36	.20
14.80	73.21	.83	1.13	SILTY SAND TO SANDY SILT	SM-ML	.81	11			60	36	.22
15.00	62.43	.91	1.46	SANDY SILT TO CLAYEY SILT	ML-MH	.75	11	3.163	19.5	56	36	.20
15.20	56.46	.44	.78	SILTY SAND TO SANDY SILT	SM-ML	.84	9			49	35	.17
15.40	194.62	1.74	.90	SAND TO SILTY SAND	SM-SP	.92	23			88	41	.50+
15.60	410.73	2.30	.56	SAND	SW-SP	1.01	39			100+	44	.50+
15.80	203.66	.21	.10	SAND	SW-SP	1.03	19			87	43	.50+
16.00	39.79	.32	.79	SANDY SILT TO CLAYEY SILT	ML-MH	.80	7	1.999	19.5	40	34	.14
16.20	97.56	.00	.00	SAND TO SILTY SAND	SM-SP	.97	11			64	38	.29
16.40	128.13	.00	.00	SAND TO SILTY SAND	SM-SP	.98	15			72	39	.50+
16.60	102.26	.05	.04	SAND TO SILTY SAND	SM-SP	1.00	12			65	39	.31
16.80	80.20	.14	.18	SAND TO SILTY SAND	SM-SP	1.01	9			57	38	.20
17.00	174.28	.77	.44	SAND TO SILTY SAND	SM-SP	1.02	20			82	40	.50+
17.20	221.14	.88	.40	SAND	SW-SP	1.12	21			88	43	.50+
17.40	214.19	.29	.14	SAND	SW-SP	1.13	20			86	43	.50+
17.60	183.18	.45	.25	SAND	SW-SP	1.14	17			81	43	.50+
17.80	228.33	.00	.00	SAND	SW-SP	1.16	21			88	43	.50+
18.00	272.79	.41	.15	SAND	SW-SP	1.17	26			94	44	.50+

18.20	247.93	.39	.16	SAND	SW-SP	1.18	23	90	43	.50+
18.40	300.85	.98	.33	SAND	SW-SP	1.20	28	98	44	.50+
18.60	334.17	2.08	.62	SAND	SW-SP	1.21	32	100+	44	.50+
18.80	210.75	1.39	.66	SAND TO SILTY SAND	SM-SP	1.13	25	86	41	.50+
19.00	231.02	.97	.42	SAND	SW-SP	1.23	22	87	43	.50+
19.20	175.37	1.05	.60	SAND TO SILTY SAND	SM-SP	1.15	21	80	40	.50+
19.40	100.97	.83	.82	SAND TO SILTY SAND	SM-SP	1.16	12	61	38	.24
19.60	93.59	.52	.56	SAND TO SILTY SAND	SM-SP	1.18	11	59	38	.22
19.80	223.35	1.15	.51	SAND	SW-SP	1.29	21	85	43	.50+
20.00	314.59	.91	.29	SAND	SW-SP	1.30	30	97	44	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

McCLELLAND CONSULTANTS (Southwest), INC.
LAND CONE TEST DATA INTERPRETATION PROGRAM

PROJECT NO. : 0601-0312
BORING NO. : 13
CONE NO. : 10
DATE : 03/08/90

DEPTH (FT)	POINT RESIST. (KSF)	SLEEVE FRICTION (KSF)	FRIC. RATIO (%)	SOIL BEHAVIOR TYPES		EFF. V. STRESS (KSF)	EQUIV. "N"	UND. SHEAR STRENGTH (KSF)	CONE FACTOR	RELATIVE DENSITY (%)	FRICTION ANGLE	STRESS RATIO
.20	59.57	.84	1.42	SANDY SILT TO CLAYEY SILT	ML-MH	.01	11	3.054	19.5	100+	42	.50+
.40	52.38	1.65	3.14	CLAYEY SILT TO SILTY CLAY	MH-CL	.02	12	2.685	19.5			
.60	49.64	.92	1.85	SANDY SILT TO CLAYEY SILT	ML-MH	.03	9	2.544	19.5	87	40	.50+
.80	45.19	.85	1.87	SANDY SILT TO CLAYEY SILT	ML-MH	.04	8	2.315	19.5	80	39	.50+
1.00	36.11	.94	2.61	CLAYEY SILT TO SILTY CLAY	MH-CL	.05	8	1.849	19.5			
1.20	55.80	1.44	2.58	SANDY SILT TO CLAYEY SILT	ML-MH	.06	10	2.858	19.5	88	40	.50+
1.40	64.70	1.99	3.07	CLAYEY SILT TO SILTY CLAY	MH-CL	.06	15	3.315	19.5			
1.60	99.59	2.83	2.84	SANDY SILT TO CLAYEY SILT	ML-MH	.08	19	5.103	19.5	100+	42	.50+
1.80	155.05	3.95	2.55	SANDY SILT TO CLAYEY SILT	ML-MH	.09	29	7.947	19.5	100+	42	.50+
2.00	146.87	3.37	2.30	SANDY SILT TO CLAYEY SILT	ML-MH	.10	28	7.527	19.5	100+	42	.50+
2.20	168.43	3.17	1.88	SILTY SAND TO SANDY SILT	SM-ML	.12	26			100+	42	.50+
2.40	203.34	4.83	2.37	SILTY SAND TO SANDY SILT	SM-ML	.13	32			100+	42	.50+
2.60	237.20	4.73	1.99	SILTY SAND TO SANDY SILT	SM-ML	.14	37			100+	42	.50+
2.80	308.29	6.90	2.24	SILTY SAND TO SANDY SILT	SM-ML	.15	49			100+	42	.50+
3.00	328.65	6.29	1.91	SILTY SAND TO SANDY SILT	SM-ML	.17	52			100+	42	.50+
3.20	252.03	2.84	1.13	SAND TO SILTY SAND	SM-SP	.19	30			100+	42	.50+
3.40	152.13	2.46	1.62	SILTY SAND TO SANDY SILT	SM-ML	.19	24			100+	42	.50+
3.60	113.67	1.17	1.03	SILTY SAND TO SANDY SILT	SM-ML	.20	18			100+	42	.50+
3.80	104.43	2.35	2.25	SANDY SILT TO CLAYEY SILT	ML-MH	.19	20	5.346	19.5	100+	42	.50+
4.00	106.46	2.10	1.97	SANDY SILT TO CLAYEY SILT	ML-MH	.20	20	5.449	19.5	100+	42	.50+
4.20	117.60	1.61	1.37	SILTY SAND TO SANDY SILT	SM-ML	.23	18			100+	42	.50+
4.40	110.59	1.54	1.39	SILTY SAND TO SANDY SILT	SM-ML	.24	17			99	42	.50+
4.60	100.32	1.48	1.48	SILTY SAND TO SANDY SILT	SM-ML	.25	16			93	41	.50+
4.80	100.32	1.44	1.44	SILTY SAND TO SANDY SILT	SM-ML	.26	16			92	41	.50+
5.00	79.81	1.12	1.41	SILTY SAND TO SANDY SILT	SM-ML	.27	12			82	40	.50+
5.20	70.78	1.13	1.60	SANDY SILT TO CLAYEY SILT	ML-MH	.26	13	3.616	19.5	79	39	.50+
5.40	70.87	1.06	1.50	SANDY SILT TO CLAYEY SILT	ML-MH	.27	13	3.621	19.5	79	39	.50+
5.60	67.45	1.76	2.61	SANDY SILT TO CLAYEY SILT	ML-MH	.28	12	3.445	19.5	76	39	.50+
5.80	53.44	1.72	3.21	CLAYEY SILT TO SILTY CLAY	MH-CL	.26	12	2.727	19.5			
6.00	38.37	1.24	3.23	CLAYEY SILT TO SILTY CLAY	MH-CL	.27	9	1.954	19.5			
6.20	33.21	.97	2.93	CLAYEY SILT TO SILTY CLAY	MH-CL	.28	7	1.689	19.5			
6.40	45.50	1.20	2.64	CLAYEY SILT TO SILTY CLAY	MH-CL	.29	10	2.318	19.5			
6.60	69.09	1.58	2.28	SANDY SILT TO CLAYEY SILT	ML-MH	.33	13	3.526	19.5	74	38	.50+
6.80	79.77	1.97	2.47	SANDY SILT TO CLAYEY SILT	ML-MH	.34	15	4.073	19.5	79	39	.50+
7.00	91.17	2.33	2.55	SANDY SILT TO CLAYEY SILT	ML-MH	.35	17	4.657	19.5	83	40	.50+
7.20	59.06	2.26	3.82	CLAYEY SILT TO SILTY CLAY	MH-CL	.32	14	3.012	19.5			
7.40	43.49	1.95	4.48	CLAY	CH	.30	20	2.215	19.5			
7.60	38.01	1.37	3.60	SILTY CLAY TO CLAY	CL-CH	.30	12	1.934	19.5			

7.80	40.90	1.17	2.87	CLAYEY SILT TO SILTY CLAY	MH-CL	.35	9	2.079	19.5			
8.00	37.00	1.19	3.22	CLAYEY SILT TO SILTY CLAY	MH-CL	.36	8	1.879	19.5			
8.20	32.20	.74	2.29	CLAYEY SILT TO SILTY CLAY	MH-CL	.37	7	1.632	19.5			
8.40	21.58	.80	3.72	CLAY	CH	.34	10	1.089	19.5			
8.60	21.89	.51	2.34	CLAYEY SILT TO SILTY CLAY	MH-CL	.39	5	1.103	19.5			
8.80	16.10	.49	3.05	SILTY CLAY TO CLAY	CL-CH	.35	5	0.808	19.5			
9.00	9.59	.36	3.78	CLAY	CH	.36	4	0.473	19.5			
9.20	5.83	.35	6.03	CLAY	CH	.37	2	0.280	19.5			
9.40	7.52	.42	5.58	CLAY	CH	.38	3	0.366	19.5			
9.60	5.82	.69	11.91	ORGANIC MATERIAL	OL-OH	.29	2	0.284	19.5			
9.80	10.43	.70	6.72	CLAY	CH	.39	4	0.515	19.5			
10.00	27.30	1.07	3.91	SILTY CLAY TO CLAY	CL-CH	.40	8	1.379	19.5			
10.20	36.36	1.34	3.67	SILTY CLAY TO CLAY	CL-CH	.41	11	1.844	19.5			
10.40	20.56	.60	2.91	SILTY CLAY TO CLAY	CL-CH	.42	6	1.033	19.5			
10.60	8.92	.77	8.61	ORGANIC MATERIAL	OL-OH	.32	4	0.441	19.5			
10.80	8.55	.55	6.44	CLAY	CH	.43	4	0.416	19.5			
11.00	19.85	.87	4.38	CLAY	CH	.44	9	0.995	19.5			
11.20	25.67	1.20	4.67	CLAY	CH	.45	12	1.293	19.5			
11.40	29.95	1.35	4.51	CLAY	CH	.46	14	1.512	19.5			
11.60	38.66	1.71	4.42	CLAY	CH	.46	18	1.959	19.5			
11.80	44.49	2.10	4.71	CLAY	CH	.47	21	2.257	19.5			
12.00	46.58	2.02	4.33	SILTY CLAY TO CLAY	CL-CH	.48	14	2.364	19.5			
12.20	40.07	2.06	5.14	CLAY	CH	.49	19	2.030	19.5			
12.40	43.47	2.12	4.88	CLAY	CH	.50	20	2.204	19.5			
12.60	40.41	2.12	5.24	CLAY	CH	.50	19	2.047	19.5			
12.80	36.12	1.30	3.60	SILTY CLAY TO CLAY	CL-CH	.51	11	1.826	19.5			
13.00	32.87	1.00	3.04	CLAYEY SILT TO SILTY CLAY	MH-CL	.58	7	1.656	19.5			
13.20	32.87	1.35	4.11	SILTY CLAY TO CLAY	CL-CH	.53	10	1.658	19.5			
13.40	45.42	1.70	3.74	SILTY CLAY TO CLAY	CL-CH	.54	14	2.302	19.5			
13.60	57.52	2.63	4.58	SILTY CLAY TO CLAY	CL-CH	.54	18	2.922	19.5			
13.80	45.88	1.64	3.58	CLAYEY SILT TO SILTY CLAY	MH-CL	.62	10	2.321	19.5			
14.00	42.44	1.80	4.24	SILTY CLAY TO CLAY	CL-CH	.56	13	2.148	19.5			
14.20	34.61	1.49	4.31	CLAY	CH	.57	16	1.746	19.5			
14.40	38.19	1.99	5.22	CLAY	CH	.58	18	1.929	19.5			
14.60	149.00	3.13	2.10	SILTY SAND TO SANDY SILT	SM-ML	.80	23			82	40	.50+
14.80	214.97	3.77	1.75	SILTY SAND TO SANDY SILT	SM-ML	.81	34			95	41	.50+
15.00	103.61	2.97	2.87	SANDY SILT TO CLAYEY SILT	ML-MH	.75	19	5.275	19.5	71	38	.50+
15.20	77.37	1.41	1.82	SANDY SILT TO CLAYEY SILT	ML-MH	.76	14	3.929	19.5	63	37	.26
15.40	45.42	1.15	2.52	CLAYEY SILT TO SILTY CLAY	MH-CL	.69	10	2.294	19.5			
15.60	43.61	1.90	4.35	SILTY CLAY TO CLAY	CL-CH	.62	13	2.205	19.5			
15.80	35.18	1.54	4.37	CLAY	CH	.63	16	1.772	19.5			
16.00	56.99	1.83	3.21	CLAYEY SILT TO SILTY CLAY	MH-CL	.72	13	2.886	19.5			
16.20	81.10	2.28	2.81	SANDY SILT TO CLAYEY SILT	ML-MH	.81	15	4.117	19.5	63	37	.26
16.40	55.44	3.55	6.41	CLAY	CH	.66	26	2.809	19.5			
16.60	351.73	3.67	1.04	SAND	SW-SP	1.08	33			100+	44	.50+
16.80	79.81	.96	1.20	SILTY SAND TO SANDY SILT	SM-ML	.92	12			60	36	.22
17.00	53.05	1.19	2.25	SANDY SILT TO CLAYEY SILT	ML-MH	.85	10	2.677	19.5	47	35	.16
17.20	49.05	1.71	3.48	CLAYEY SILT TO SILTY CLAY	MH-CL	.77	11	2.476	19.5			
17.40	135.46	1.85	1.36	SILTY SAND TO SANDY SILT	SM-ML	.96	21			75	39	.50+
17.60	439.70	2.51	.57	SAND	SW-SP	1.14	42			100+	44	.50+
17.80	533.15	1.84	.35	GRAVELLY SAND TO SAND	SW-GW	1.16	42			100+	46	.50+
18.00	435.07	1.38	.32	SAND	SW-SP	1.17	41			100+	44	.50+

18.20	387.41	1.78	.46	SAND	SW-SP	1.18	37	100+	44	.50+
18.40	415.68	1.93	.46	SAND	SW-SP	1.20	39	100+	44	.50+
18.60	196.90	2.67	1.36	SAND TO SILTY SAND	SM-SP	1.12	23	84	41	.50+
18.80	419.19	2.63	.63	SAND	SW-SP	1.22	40	100+	44	.50+
19.00	365.19	1.50	.41	SAND	SW-SP	1.23	34	100+	44	.50+
19.20	344.81	1.05	.31	SAND	SW-SP	1.25	33	100+	44	.50+
19.40	361.76	.01	.00	SAND	SW-SP	1.26	34	100+	44	.50+
19.60	128.94	.88	.68	SAND TO SILTY SAND	SM-SP	1.18	15	68	39	.43
19.80	192.24	1.12	.58	SAND TO SILTY SAND	SM-SP	1.19	23	82	40	.50+
20.00	423.58	3.04	.72	SAND	SW-SP	1.30	40	100+	44	.50+
20.20	231.43	3.01	1.30	SAND TO SILTY SAND	SM-SP	1.21	27	87	41	.50+
20.40	192.38	2.68	1.39	SAND TO SILTY SAND	SM-SP	1.22	23	82	40	.50+
20.60	175.68	3.02	1.72	SILTY SAND TO SANDY SILT	SM-ML	1.13	28	81	39	.50+
20.80	214.53	1.64	.76	SAND TO SILTY SAND	SM-SP	1.25	25	84	41	.50+

Notes:

1. Default values of submerged unit weight and cone factor are used.
2. Relative density is estimated based on Schmertmann (1978).
3. Friction angle is estimated based on Schmertmann (1978).

APPENDIX E

RESULTS OF DISTRICT 12 LABORATORY TESTING ON SUBSURFACE SOIL SAMPLES AT SITE

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS	STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HIGHWAY NO.- US-290	HOLE NO.- W71
CONTROL- 0050-06-033	STATION- 1967+25.00
PROJECT NO.-	OFFSET- 00.00'
DATE- 04/17/90	CODED BY- PAZ
GRD. ELEV.- 142.8	GRD. WATER ELEV.- 142.8
DISTRICT- 12	

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0										
1				1						CLAY,SANDY,GR-TAN,SOFT,MOIST,
2										CALCAREOUS,FER.
3				2						
4										
5				4						

DRILLER- MIKE BAHM LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.8 FEET

HOLE NUMBER W71

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.8	136.8	0.00	0.00	0.00	0.7	0.00	0.00	3.62	0.00	0.0

DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W71	DATE	04/17/90
CONTROL	0050-06-033	STATION	1967+25.00	GRD. ELEV.	142.6
IPE		LOCATION	00.00'	GRD.WATER ELEV.	142.6

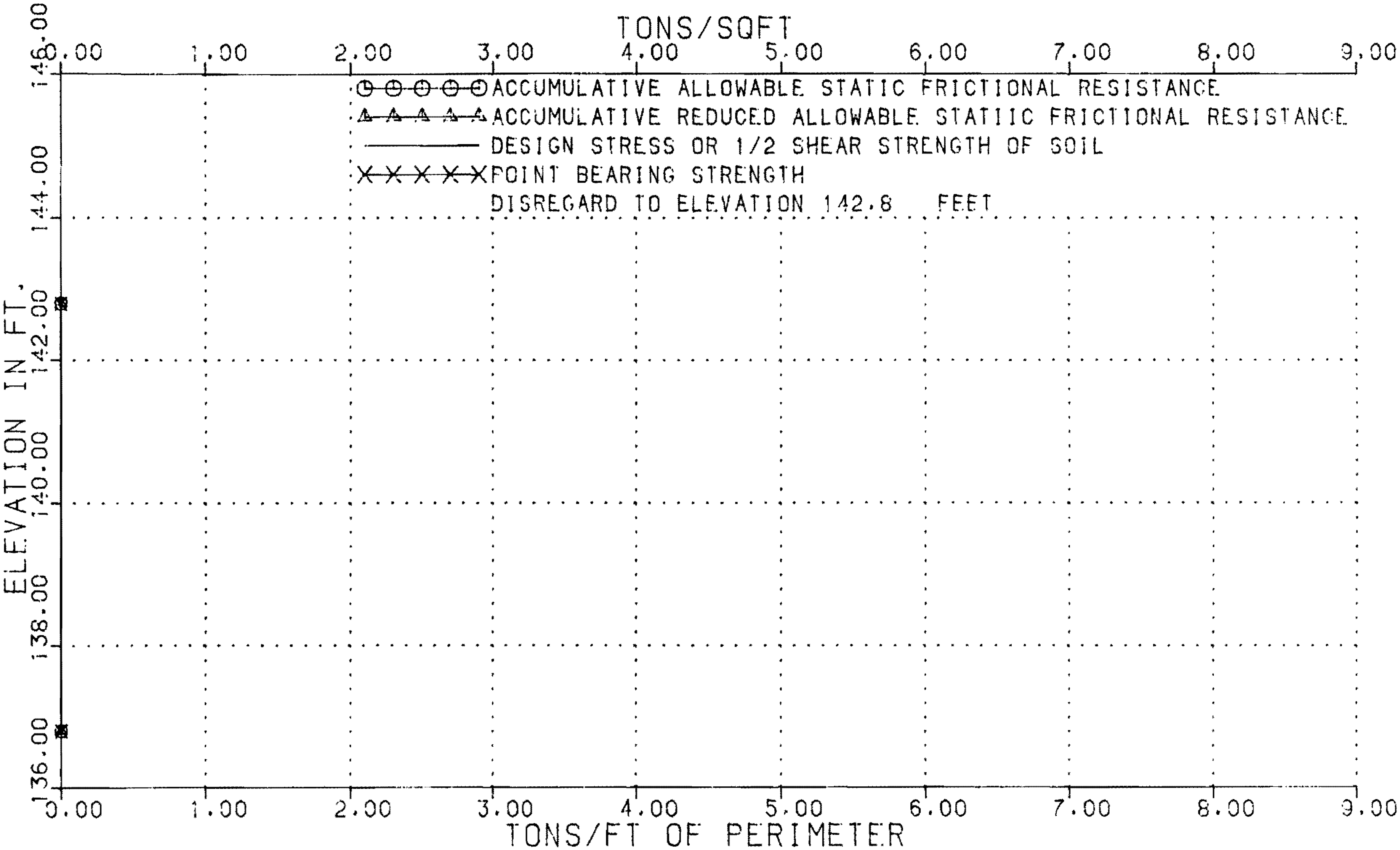
ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
				0	
142.6 0				CLAY,SANDY,GR-TAN,SOFT,MOI,CALCS.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

E5

FOUNDIN' MATERIAL PROP'RTIES



COUNTY	HARRIS	STRUCTURE US-290 CYPRESS BYPASS RET. WALL.		THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W71	DATE	04/17/90
CONTROL	0050-06-033	STATION	1967+25.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W71

GROUND ELEV. 142.8

CLAY,SANDY,GR-TAN,SOFT,MOI,CALCS.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.1
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W72
STATION- 1966+75.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.1

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1											SAND(CEMENT STABILIZED SAND TO 4').
2											
3											
4					1	0- 5	126	21			CLAY,SANDY,MULTICOLORED,SOFT,MOIST, CALCAREOUS.
5											
6											
7					2	0- 3	124	20	40		- V.SOFT.
8											
9					3	0- 2	116	26	45		
10											
11					5	0- 4	122	24	31		
12											
13					7	0- 2	122	24	31		
14					9	0- 14	132	18			- STIFF.
15											

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W72

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	142	138	4.0	2.0	OTHE								
2	138	127	11.0	9.5	CL			359	124	0.0	4.4	1.5	5.8

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.1 FEET

HOLE NUMBER W72

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.1	138.1	0.00	0.00	0.00	0.7	0.00	0.00	4.29	0.00	0.0
2	138.1	127.1	0.09	0.99	0.99	0.7	0.69	0.69	3.62	0.00	0.0

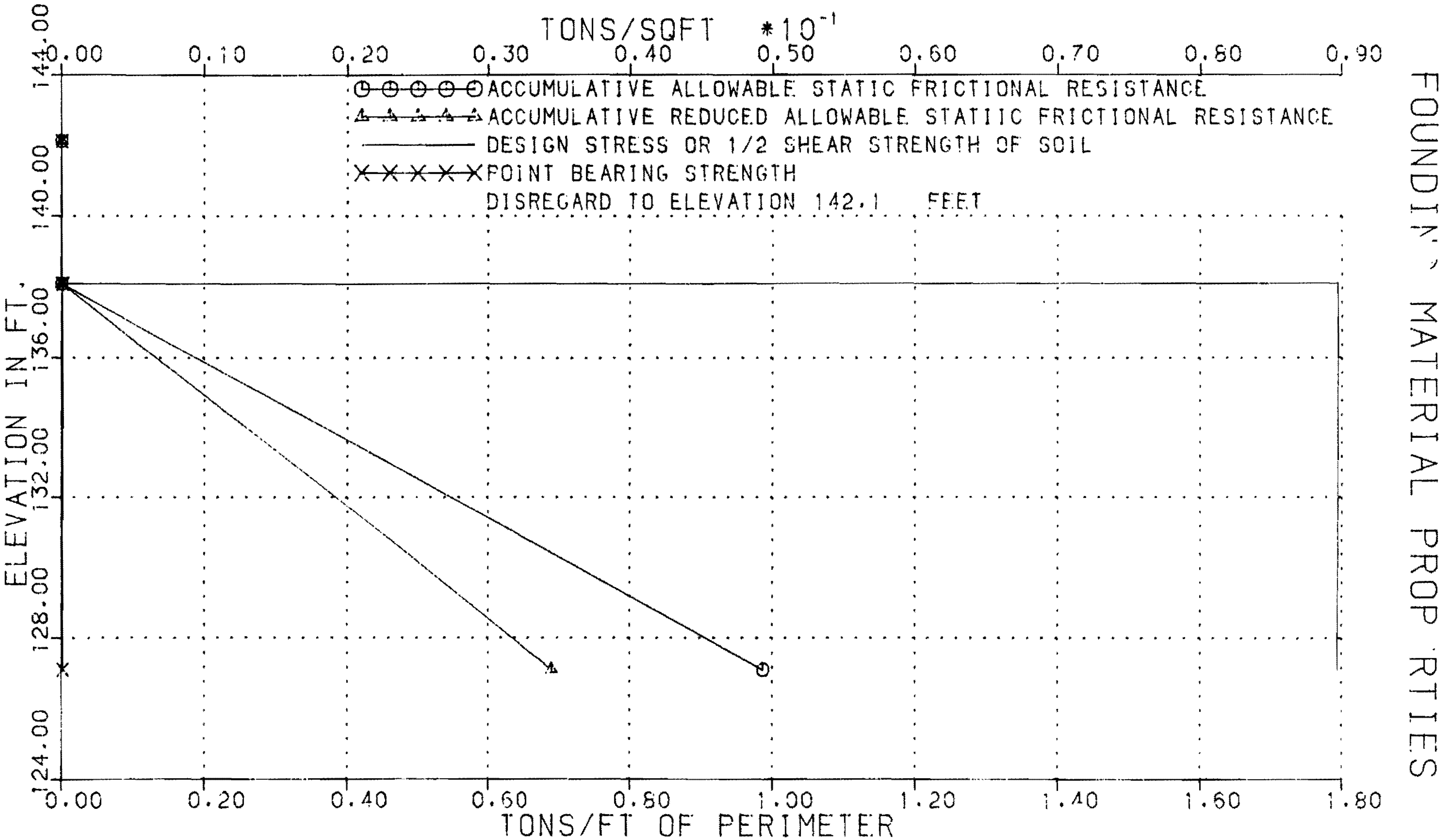
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W72	DATE	04/17/90
CONTROL	0050-06-033	STATION	1966+75.00	GRD. ELEV.	142.1
IPE		LOCATION	00.00'	GRD.WATER ELEV.	142.1

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
142.1	0				SAND. (CEMENT STABILIZED SAND).	0	
136.1					CLAY, SANDY, MULTICOLORED, SOFT, MOI.		
	10					10	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



E11

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W72	DATE	04/17/90
CONTROL	0050-06-033	STATION	1966+75.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W72

GROUND ELEV. 142.1

SAND. (CEMENT STABILIZED SAND).

CLAY,SANDY,MULTICOLORED,.SOFT,MOI.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 144.1
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W73
STATION- 1963+50.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 144.1

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1					2	0- 7	128	23	39		CLAY,SANDY,MULTICOLORED,SOFT,MOIST,
2					3			23			CALCAREOUS.
3					5	0- 4	126	21	39		
4					6			22			
5											
6					7	0- 4	127	21	33		
7											- ENCOUNTERED BOX CULVERT 6.5'

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W73

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	144	137	7.0	3.5	CL			358	127	0.0	1.0	0.6	3.4

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.1 FEET

HOLE NUMBER W73

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	144.1	137.1	0.09	0.63	0.63	0.7	0.44	0.44	3.62	0.00	0.0

DRILLING LOG

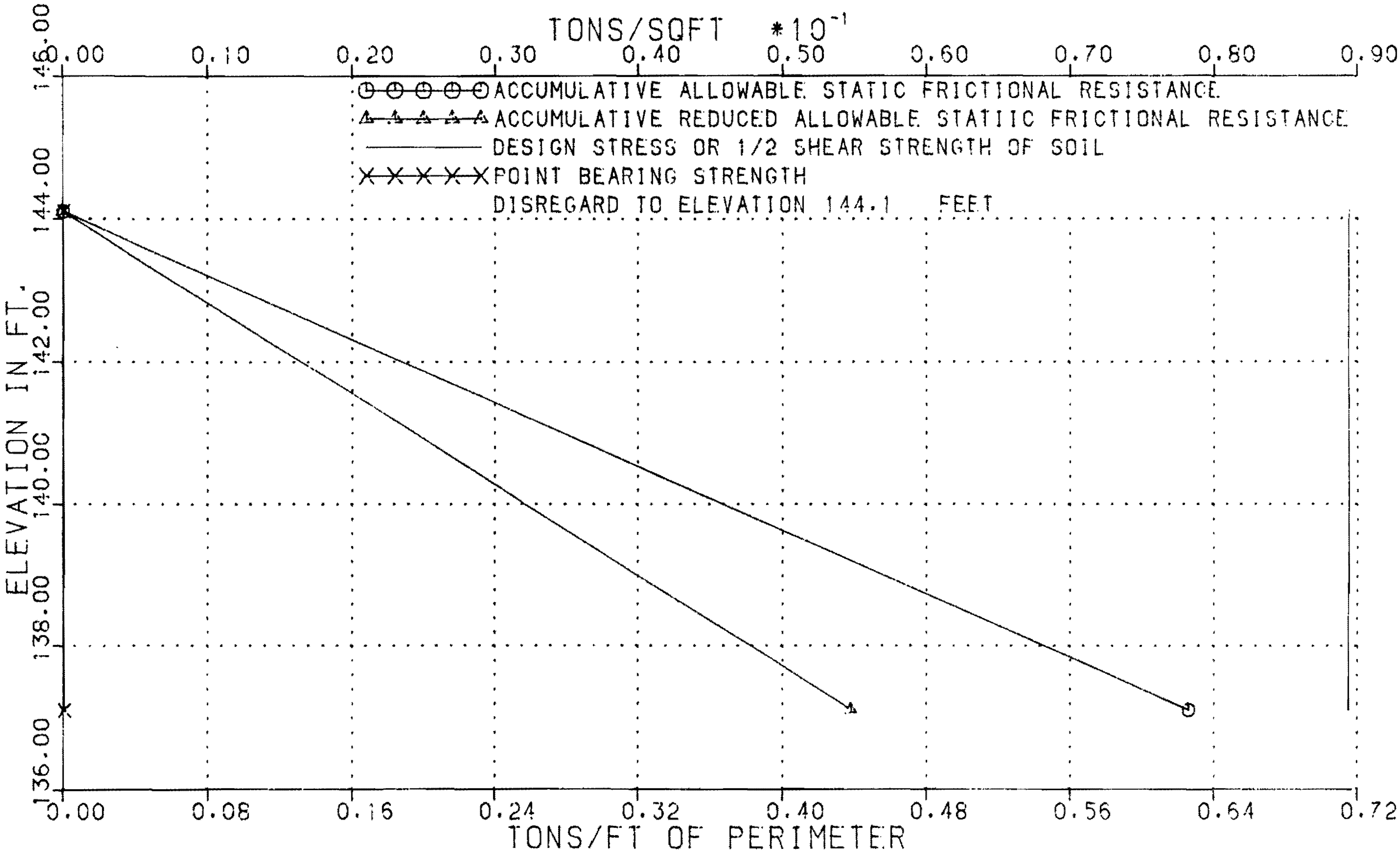
COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W73	DATE	04/17/90
CONTROL	0050-06-033	STATION	1963+50.00	GRD. ELEV.	144.1
IPE		LOCATION	00.00'	GRD.WATER ELEV.	141.1

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
141.1 0				CLAY, SANDY, MULTICOLORED, SOFT	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E. TITLE NGR. TECH III

FOUNDING MATERIAL PROPERTIES



COUNTY	HARRIS	STRUCTURE US-290 CYPRESS BYPASS RET. WALL.		THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W73	DATE	04/17/90
CONTROL	0050-06-033	STATION	1963+50.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W73

GROUND ELEV. 144.1

CLAY,SANDY,MULTICOLORED,SOFT

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 144.8
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W74
STATION- 1960+25.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 144.8

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT- ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0										
1				2	0- 13	130	33	32		CLAY,SANDY,DK.GR-TAN,STIFF,MOIST.
2										
3				4			17			- CALCAREOUS.
4										
5				5			19	40		- FIRM.
6				7	0- 17	134	17	39		- STIFF.
7										
8										
9				9	0- 22	125	22	61		CLAY,TAN-GR,STIFF,MOIST,CALCS.
10				11			17			- CALCAREOUS LAYERS.
11										
12				13	0- 25	133	16	51		
13				14	0- 31	127	21			- V.STIFF.
14										
15										

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- ENGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W74

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE PEAK	AVG	TAT RATIO
1	145	137	8.0	4.0	CL			1084	132	0.0			
2	137	130	7.0	11.5	CH			1884	128	0.0	2.6	1.7	3.7

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.6 FEET

HOLE NUMBER W74

STRATA NUMBER	ELEVATION -FEET- FROM	ELEVATION -FEET- TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	144.6	136.8	0.27	2.17	2.17	0.7	1.52	1.52	3.62	0.00	0.0
2	136.8	129.8	0.47	3.30	5.46	0.7	2.31	3.83	2.57	0.00	0.0

DRILLING LOG

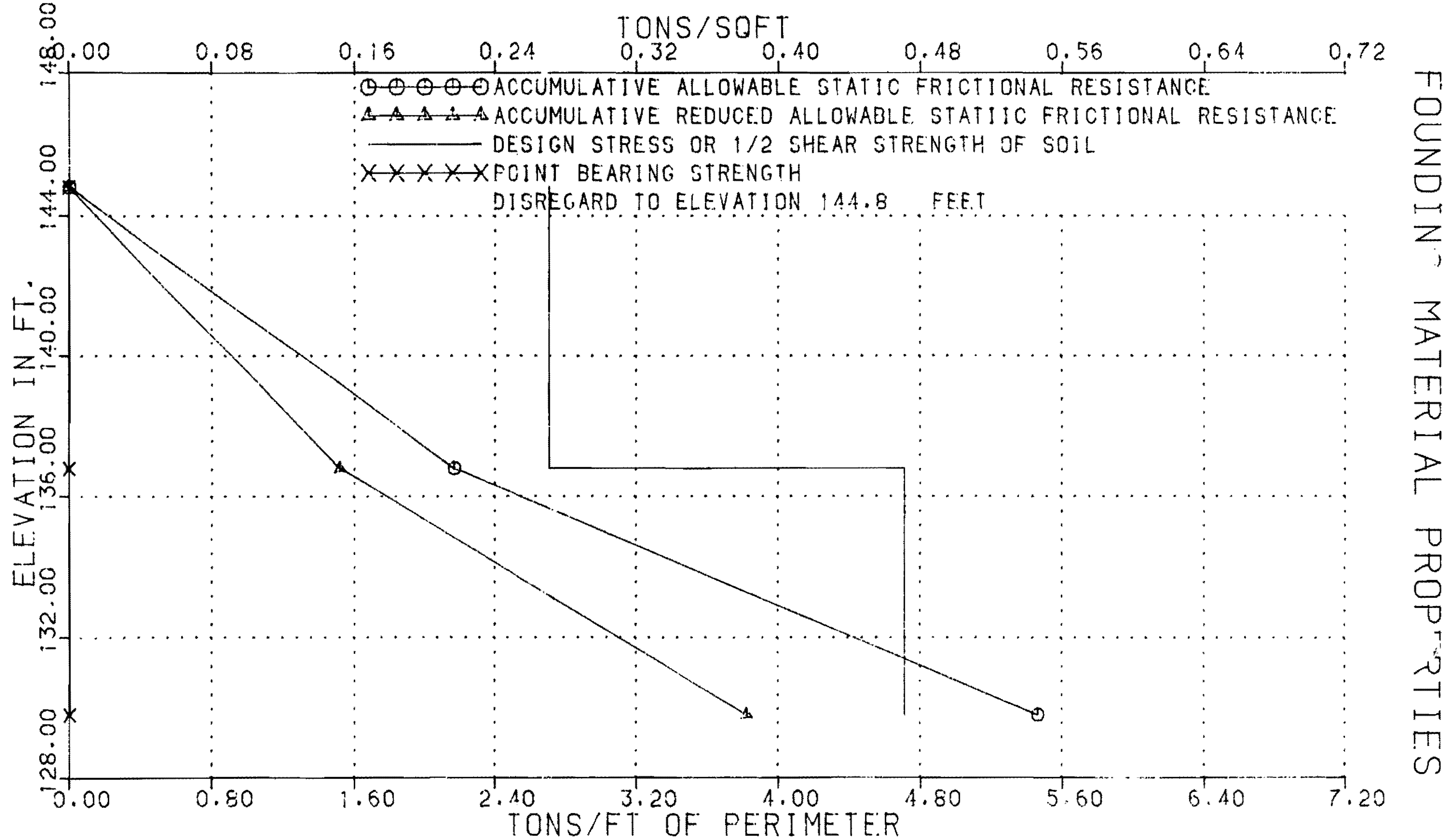
COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W74	DATE	04/17/90
CONTROL	0050-06-033	STATION	1950+25.00	GRD. ELEV.	141.8
IPE		LOCATION	00.00'	GRD. WATER ELEV.	141.8

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
141.8 0				CLAY, SANDY, DK. GR-TAN, STIFF, MOI.	
135.8				CLAY, TAN-GR, STIFF, MOIST, CALAS.	
10					

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

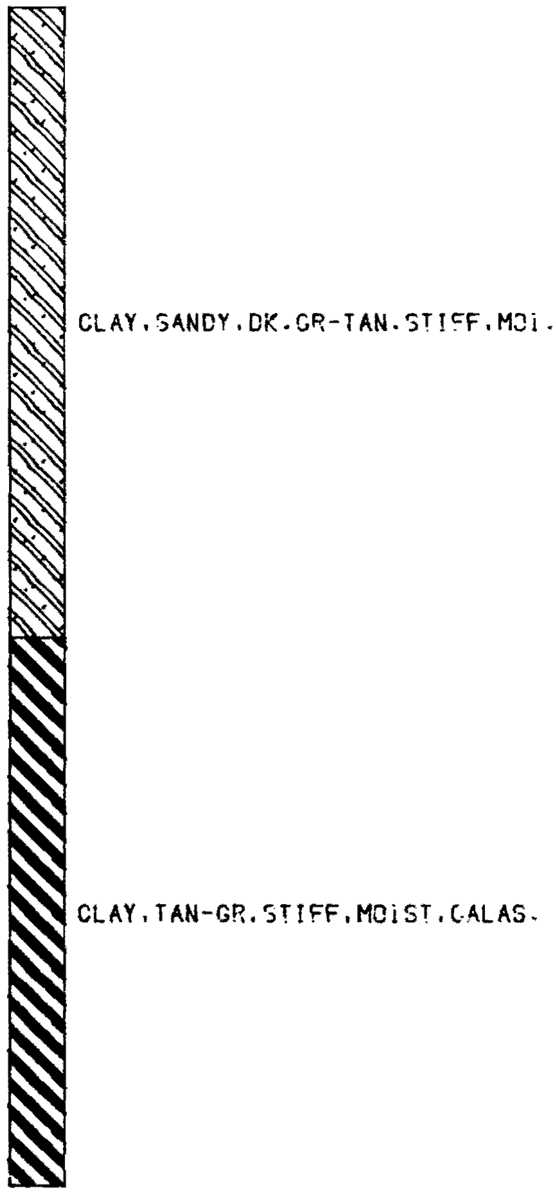
E23



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W74	DATE	04/17/90
CONTROL	0050-06-033	STATION	1960+25.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W74

GROUND ELEV. 144.8



PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 144.2
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W81
STATION- 1965+80.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 144.2

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
	0										
	1				1				38		CLAY,SANDY,TAN-GR,SOFT,MOIST,
	2										CALCAREOUS.
	3				3						
	4				4						
	5										- ENCOUNTERED BOX CULVERT 4.5'

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.2 FEET

HOLE NUMBER W81

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TA TO PE RA
1	144.2	139.2	0.00	0.00	0.00	0.7	0.00	0.00	3.62	0.00	0.[illegible]

DRILLING LOG

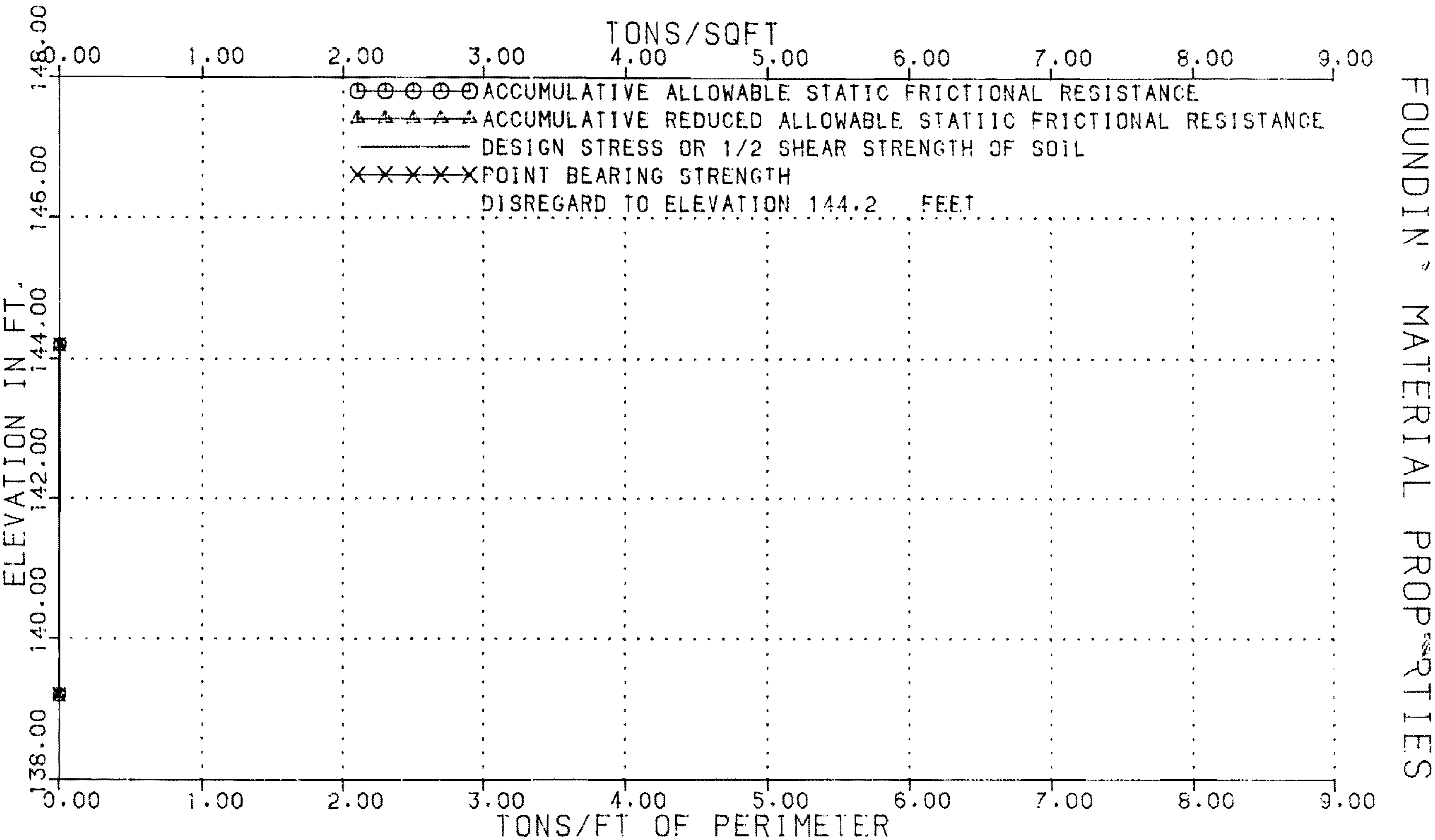
COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W81	DATE	04/17/90
CONTROL	0050-06-033	STATION	1965+60.00	GRD. ELEV.	144.2
IPE		LOCATION	00.00'	GRD. WATER ELEV.	144.2

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
144.2 0				CLAY, SANDY, TAN-GR, SOFT, MOI, CALCS. C	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

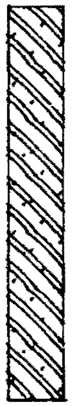
E28



COUNTY	HARRIS	STRUCTURE US-290 CYPRESS BYPASS RET. WALL.		THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W81	DATE	04/17/90
CONTROL	0050-06-033	STATION	1965+60.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W81

GROUND ELEV. 144.2



CLAY,SANDY,TAN-GR,SOFT,MOI,CALCS.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 147.2
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W82
STATION- 1962+65.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 147.2

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									
1			1	0- 7	131	21			CLAY,SILTY,TAN-GR,SOFT,(FILL-
2			2	0- 18	137	16	41		MATERIAL).- STIFF.
3			3	0- 38	139	18			- V.STIFF.
4			4	0- 17	137	19	45		- STIFF.
5			5	0- 42	136	15			- V.STIFF.
6			6	0- 47	138	14			- ENCOUNTERED STORM SEWER 5.5'

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W82

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	147	141	6.0	3.0	CL			1751	136	0.0	8.8	6.3	3.9

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 147.2 FEET

HOLE NUMBER W82

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	147.2	141.2	0.44	2.63	2.63	0.7	1.84	1.84	3.62	0.00	0.0

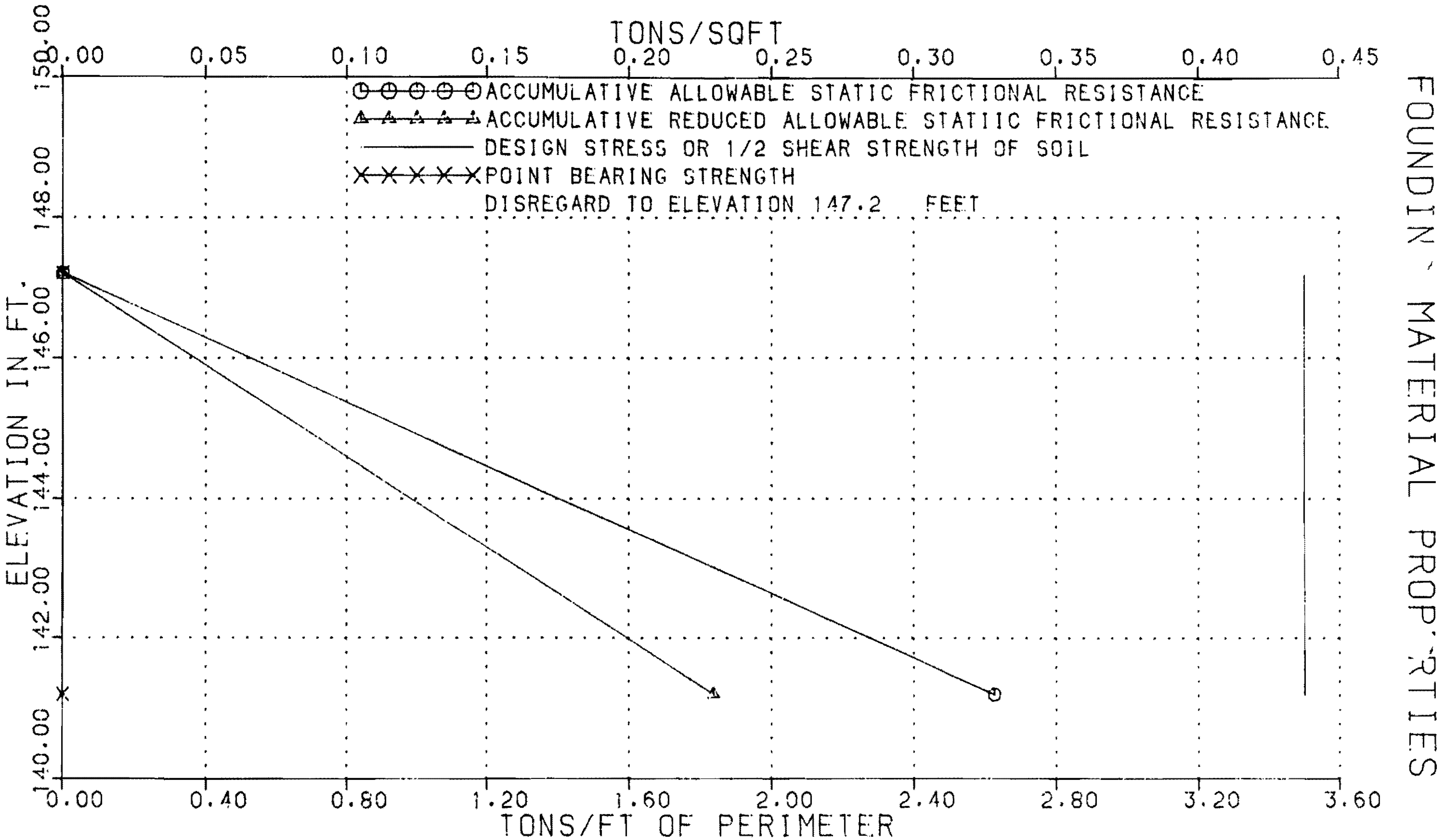
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W82	DATE	04/17/90
CONTROL	0050-06-033	STATION	1962+65.00	GRD. ELEV.	147.2
IPE		LOCATION	00.00'	GRD. WATER ELEV.	147.2

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
147.2 0				CLAY, SILTY, TAN-GR, SOFT, (FILL).	C

*REMARKS:

DRILLER MIKE BAHM   LOGGER LOGGER AL FERRALL E.   TITLE NGR. TECH III

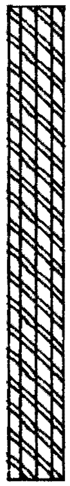


E34

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W82	DATE	04/17/90
CONTROL	0050-06-033	STATION	1962+65.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W82

GROUND ELEV. 147.2



CLAY,SILTY,TAN-GR,SOFT,(FILL).

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 146.3
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W83
STATION- 1959+50.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 146.3

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									
1			1			16			CLAY,SILTY,TAN-GR,STIFF,(FILL
2			2	0- 16	114	16	37		MATERIAL).
3			3						- GRAVEL.
4			4	0- 26	115	17	39		
5									- ENCOUNTERED STORM SEWER 4.2'

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W83

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	146	141	5.0	2.5	CL			1487	114	0.0			

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 146.3 FEET

HOLE NUMBER W83

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	146.3	141.3	0.37	1.86	1.86	0.7	1.30	1.30	3.62	0.00	0.0

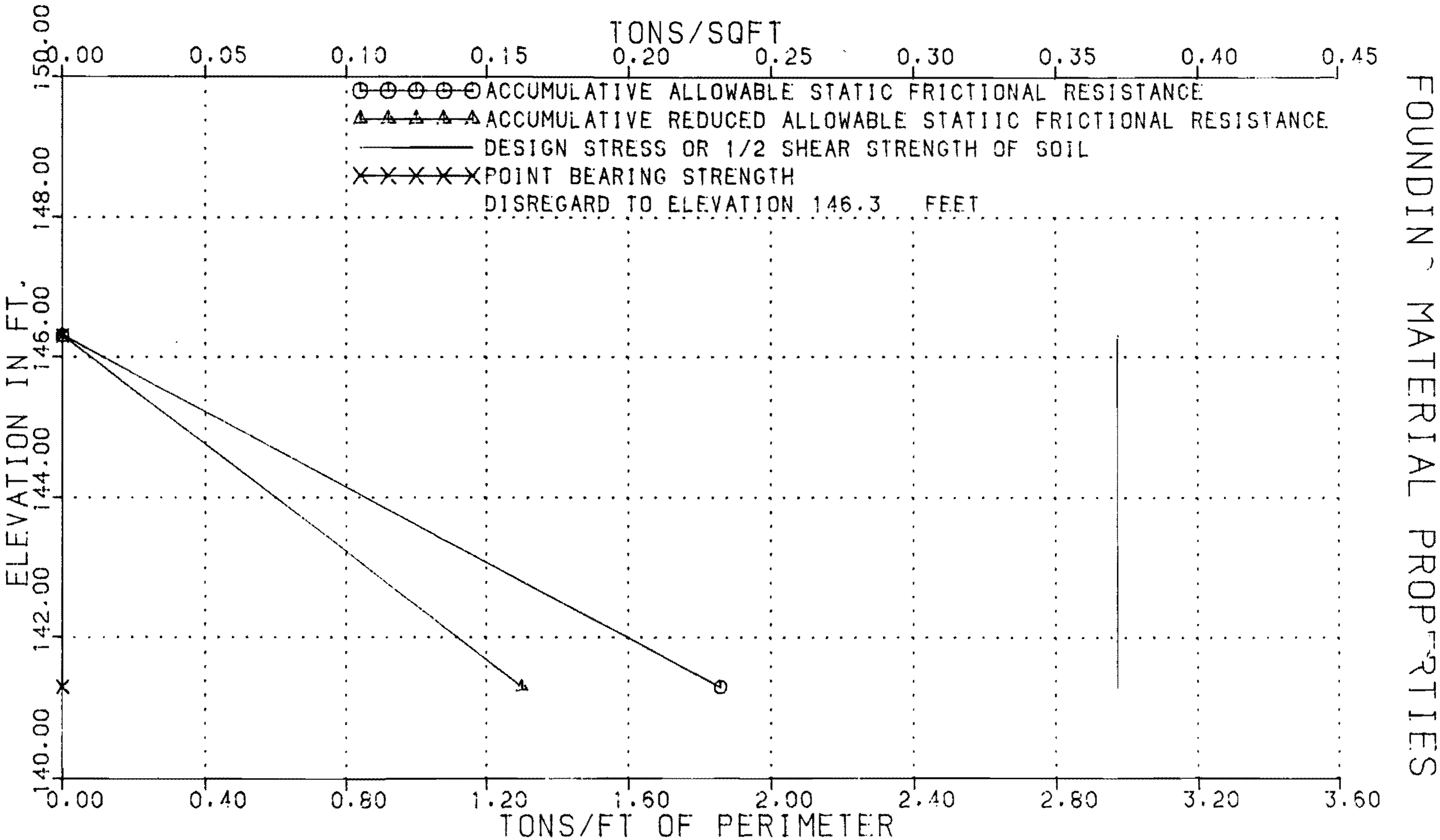
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W63	DATE	04/17/90
CONTROL	0050-06-033	STATION	1959+50.00	GRD. ELEV.	146.3
IPE		LOCATION	00.00'	GRD. WATER ELEV.	146.3

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
146.3 0				CLAY, SILTY, TAN-GR, STIFF, (FILL).	

*REMARKS:

DRILLER MIKE BAHM   LOGGER LOGGER AL FERRALL E   TITLE NGR. TECH III

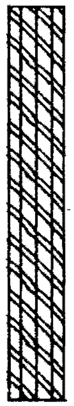


E40

COUNTY	HARRIS	STRUCTURE US-290 CYPRESS BYPASS RET. WALL.		THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W83	DATE	04/17/90
CONTROL	0050-06-033	STATION	1959+50.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W83

GROUND ELEV. 146.3



CLAY,SILTY,TAN-GR,STIFF,(FILL).

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 145.5
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W84
STATION- 1957+25.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 145.5

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0										
1				1			21			CLAY,SILTY,TAN-GR,SOFT,(FILL MATERIAL).
2				2	0- 6	112	16	38		
3				3	0- 4	106	23			
4				4				36		
5				5	0- 9	113	19	37		- FIRM.
6				6			16			- ENCOUNTERED STORM SEWER 6'.

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W84

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	145	139	6.0	3.0	CL			466	110	0.0	1.2	0.8	4.2

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 145.5 FEET

HOLE NUMBER W84

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM- ULATIVE	SK	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM- ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	145.5	139.5	0.12	0.70	0.70	0.7	0.49	0.49	3.62	0.00	0.0

DRILLING LOG

COUNTY HARRIS
HIGHWAY NO US-290
CONTROL 0050-06-033
IPE

STRUCTURE US-290 CYPRESS BYPASS RET. WALL.
HOLE NO W84
STATION 1957+25.00
LOCATION 00.00'

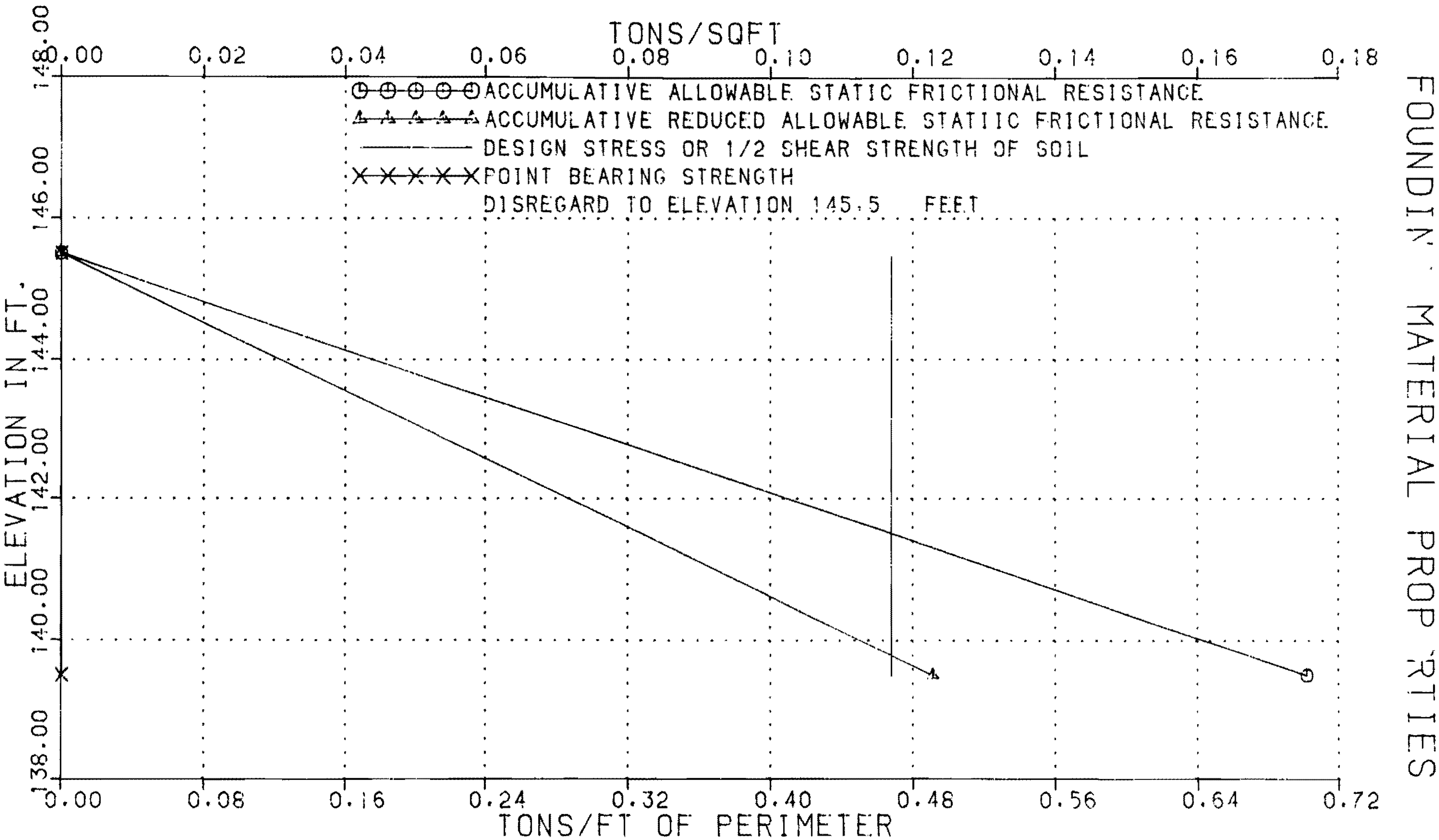
THD DIST 12
DATE 04/17/90
GRD. ELEV. 145.5
GRD.WATER ELEV. 145.5

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
145.5 0				C	
				CLAY,SILTY,TAN-GR,SOFT,(FILL).	

*REMARKS:

DRILLER MIKE BAHM
LOGGER LOGGER AL FERRALL E
TITLE NGR. TECH III

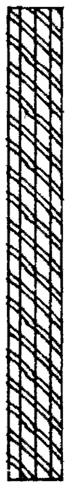
E46



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W84	DATE	04/17/90
CONTROL	0050-05-033	STATION	1957+25.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W84

GROUND ELEV. 145.5



CLAY, SILTY, TAN-GR, SOFT, (FILL).

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 145.7
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W85
STATION- 1954+25.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 145.7

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT- ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0										
1				1			17			CLAY,SILTY,TAN-GR,FIRM,GRVL,(FILL
2				2			18	40		MATERIAL).
3				3	0- 9	109	16			
4				4	0- 17	115	15			- STIFF.
5				5			19	34		
6										- ENCOUNTERED STORM SEWER 5.3'

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W85

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	146	140	6.0	3.0	CL			950	112	0.0			

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 145.7 FEET

HOLE NUMBER W85

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	145.7	139.7	0.24	1.43	1.43	0.7	1.00	1.00	3.62	0.00	0.0

DRILLING LOG

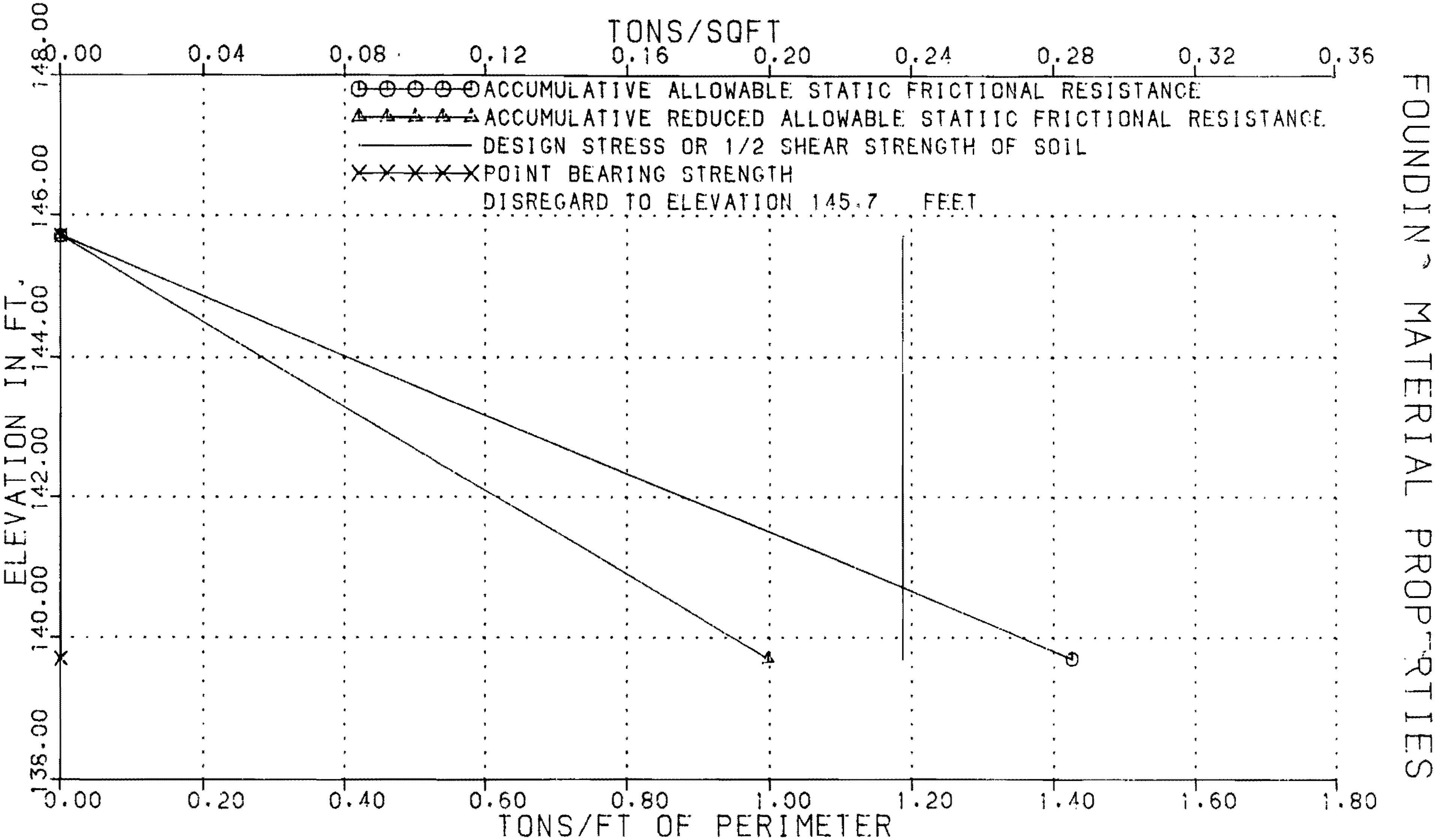
COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W85	DATE	04/17/90
CONTROL	0050-05-033	STATION	1954+25.00	GRD. ELEV.	145.7
IPE		LOCATION	00.00'	GRD. WATER ELEV.	145.7

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
145.7 0				CLAY, SILTY, TAN-GR, FIRM, GRVL. (FILL).	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

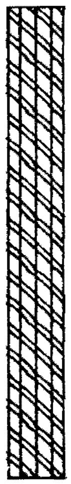
E52



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W85	DATE	04/17/90
CONTROL	0050-06-033	STATION	1954+25.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W85

GROUND ELEV. 145.7



CLAY,SILTY,TAN-GR,FIRM,GRVL,(FILL).

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.5
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W91
STATION- 1970+20.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.5

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
	0										
	1				1						CLAY,SANDY,MULTICOLORED,SOFT,MOIST.
	2										CALCAREOUS.
	3				2						
	4				4						
	5										SAND,F-MED,TAN,LOOSE,MOIST.

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

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 PROB (CONTD)
 2165        (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

                  SOIL STRENGTH ANALYSIS RESULTS

  HOLE NUMBER W91

                 STRAT  CEN-        PEN          SOIL WET
      ELEVATION  THICK  TROID UNIF. TEST         COHES DENS INT.   VARIANCE TAT
STRAT  -FEET-    NESS   DEPTH SOIL BLOWS/ TEST   LBS/ LBS/ FRIC  -------------
 NUM.  FROM  TO  FEET   FEET  CLASS FOOT  CONST  SQFT CUFT ANG.  PEAK AVG RATIO

   1   142  138   4.0    2.0  CL
   2   138  137   1.0    4.5  OTHE

FY209I VFNTH : PROGRAM INTERRUPT - FLOATING-POINT DIVIDE EXCEPTION
       VFNTH : PSW 078D000FA015-46C0, REGISTER CONTAINS 42410000000000000000000000
       VFNTH : LAST EXECUTED FORTRAN STATEMENT IN PROGRAM LINHO AT ISN 40 (OFFSET 0

RACEBACK OF CALLING ROUTINES; MODULE ENTRY ADDRESS = 125C70.
-------------------------------------------------------------------------------
 NHO (1503C0) CALLED BY BFAST (125C70) AT ISN 146 AT OFFSET 0015E6.
 RGUMENT LIST AT 1261AC.
  ARG. NO.   ADDRESS          INTEGER        REAL         CHAR   HEXADECIMAL
        1   0012601C :             19   0.000010E-78   '....'   00000013
        2   801260C0 :     1116635136   0.142500E+03   '....'   428E8000
-------------------------------------------------------------------------------
BFAST (125C70) CALLED BY OPERATING SYSTEM.
-------------------------------------------------------------------------------

TANDARD CORRECTIVE ACTION TAKEN. EXECUTION CONTINUING.
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PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.5 FEET

HOLE NUMBER W91

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.5	138.5	0.00	0.00	0.00	0.7	0.00	0.00	3.62	0.00	0.0
2	138.5	137.5	0.00	0.00	0.00	0.7	0.00	0.00	4.29	0.00	0.0

DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W91	DATE	04/17/90
CONTROL	0050-06-033	STATION	1970+20.00	GRD. ELEV.	142.5
TYPE		LOCATION	00.00'	GRD. WATER ELEV.	142.5

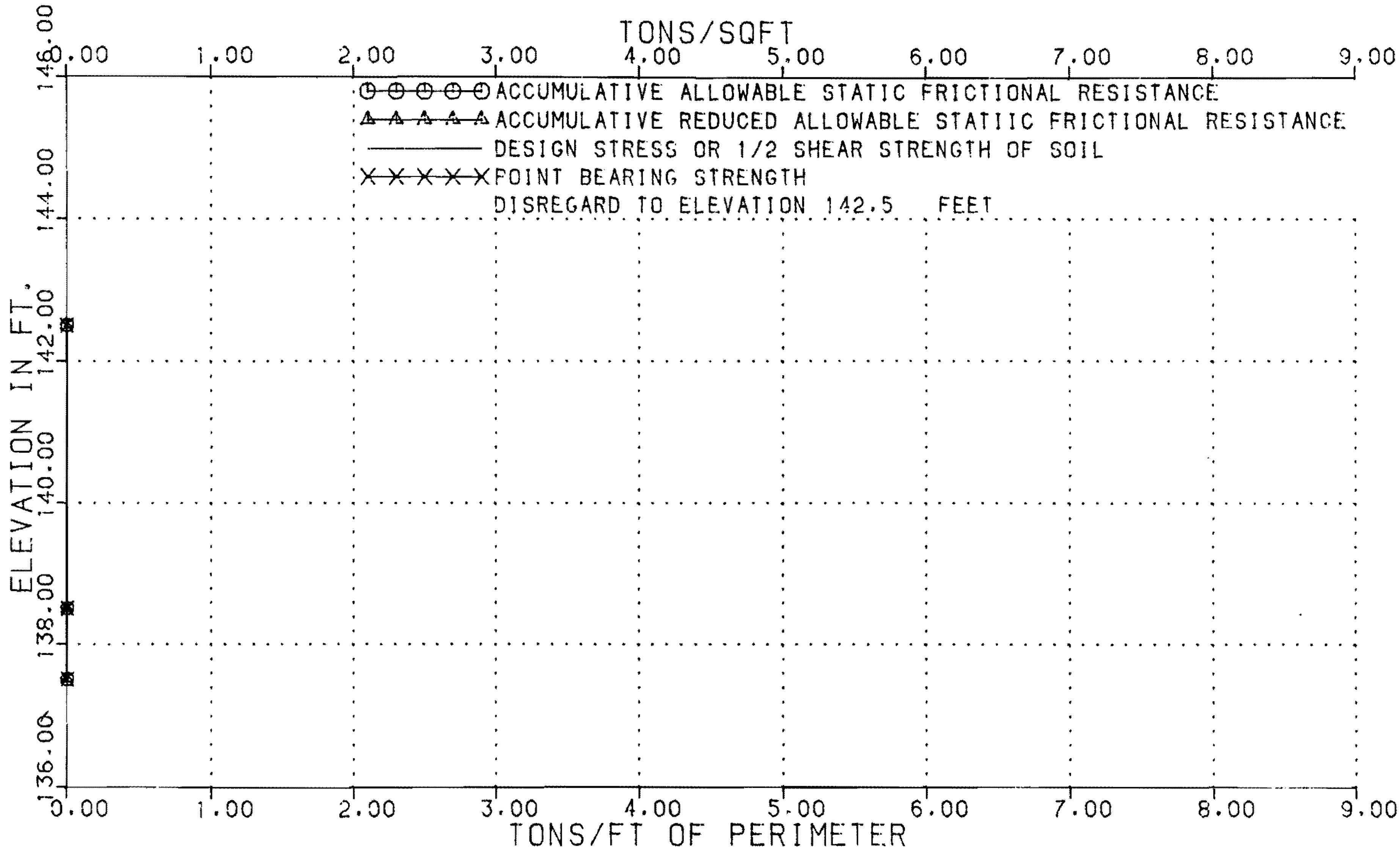
ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.5 0				CLAY, SANDY, MULTICOLORED, SOFT, MOIST.	C
136.5				SAND, F-MED, TAN, LOOSE, MOIST.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

E58

FOUNDING MATERIAL PROPERTIES



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W91	DATE	04/17/90
CONTROL	0050-06-033	STATION	1970+20.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W91

GROUND ELEV. 142.5

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.5
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W92
STATION- 1969+60.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.5

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT- ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1											SAND, (CEMENT STABILIZED SAND).
2											
3					1			16			CLAY,SANDY,GR-TAN,SOFT,MOIST,
4					2	0- 6	135	16			CALCAREOUS.
5					4	0- 18	137	15			- FER.
6											
7					6	0- 14	133	17			- STIFF.
8					7	0- 21	137	17			
9					9	0- 19	138	15			
10					10	0- 21	136	21			
11					11	0- 11	135	16	37		- FIRM.
12					14	0- 20	126	20			- STIFF.
13											
14					16	0- 17	118	27			
15					17			22			

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W92

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE PEAK	AVG	TAT RATIO
1	142	127	15.0	7.5	CL			1182	133	0.0	5.0	1.9	5.3

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.5 FEET

HOLE NUMBER W92

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.5	127.5	0.30	4.43	4.43	0.7	3.10	3.10	3.62	0.00	0.0

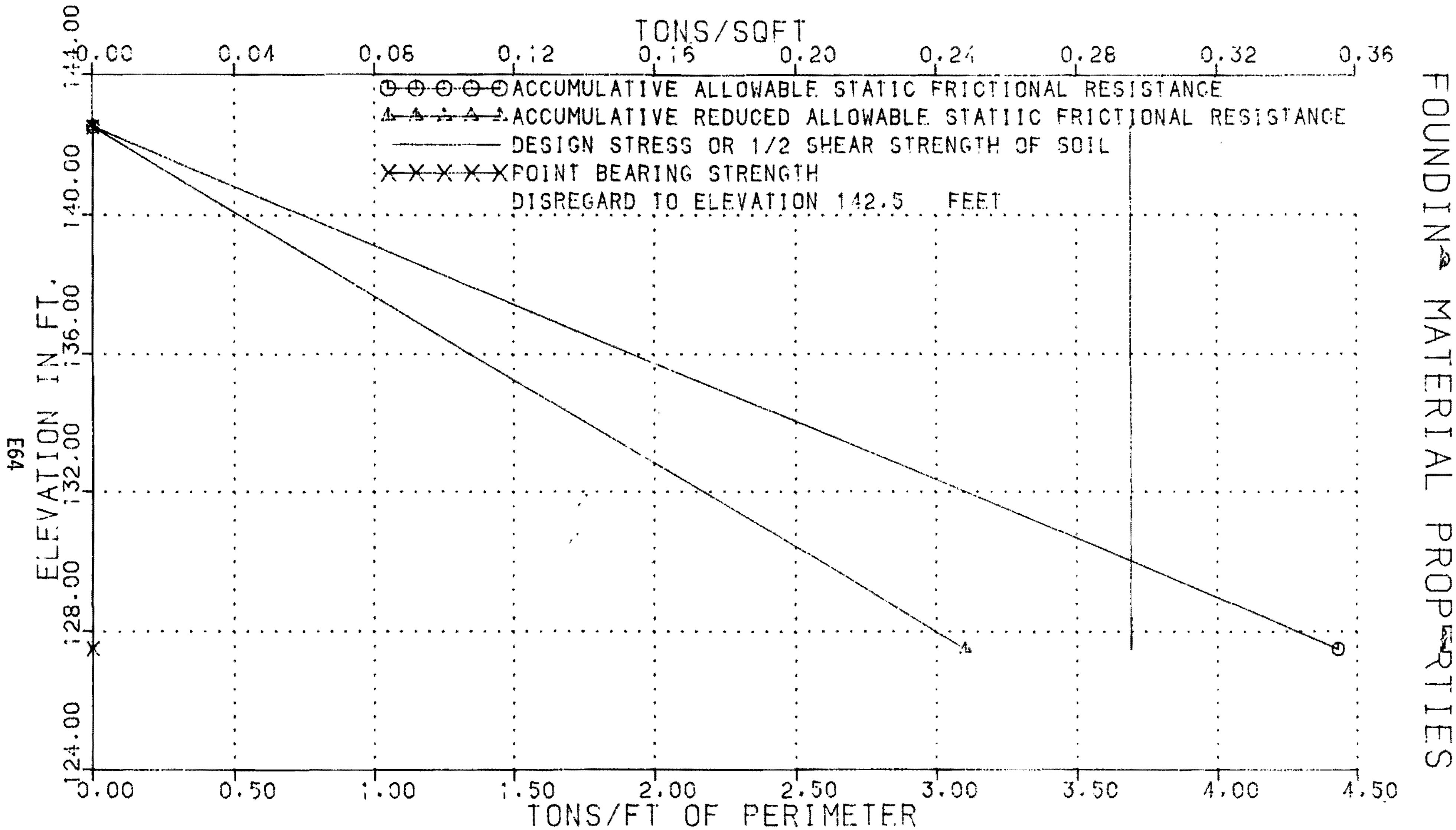
DRILLING LOG

COUNTY HARRIS | STRUCTURE US-290 CYPRESS BYPASS RET. WALL. | THD DIST 12
HIGHWAY NO US-290 | HOLE NO W92 | DATE 04/17/90
CONTROL 0050-06-033 | STATION 969+60.00 | GRD. ELEV. 142.5
IPE | LOCATION 00.00' | GRD. WATER ELEV. 142.5

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.5 0				CLAY,SANDY,BRN-GR,SOFT,MOIST,CALCS.	
10					

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W92	DATE	04/17/90
CONTROL	0050-06-033	STATION	1969+60.00	BY	PAZ
IPE		LOCATION	00.00		

TEST HOLE NO. W92

GROUND ELEV. 142.5

CLAY,SANDY,BRN-GR,SOFT,MOIST,CALCS.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.6
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W93
STATION- 1969+75.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.6

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1											SAND, (CEMENT STABILIZED SAND).
2											
3											CLAY,SANDY,GR-TAN,SOFT,MOIST,FER.
4					1	0- 28	137	15			
5											
6					2	0- 18	135	17			
7											
8					5	0- 9	132	19			
9					6	0- 14	133	19			

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W93

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	143	141	2.0	1.0	OTHE								
2	141	134	7.0	5.5	CL			1334	135	0.0	4.7	3.1	4.2

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.6 FEET

HOLE NUMBER W93

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.6	140.6	0.00	0.00	0.00	0.7	0.00	0.00	4.29	0.00	0.0
2	140.6	133.6	0.35	2.34	2.34	0.7	1.63	1.63	3.62	0.00	0.0

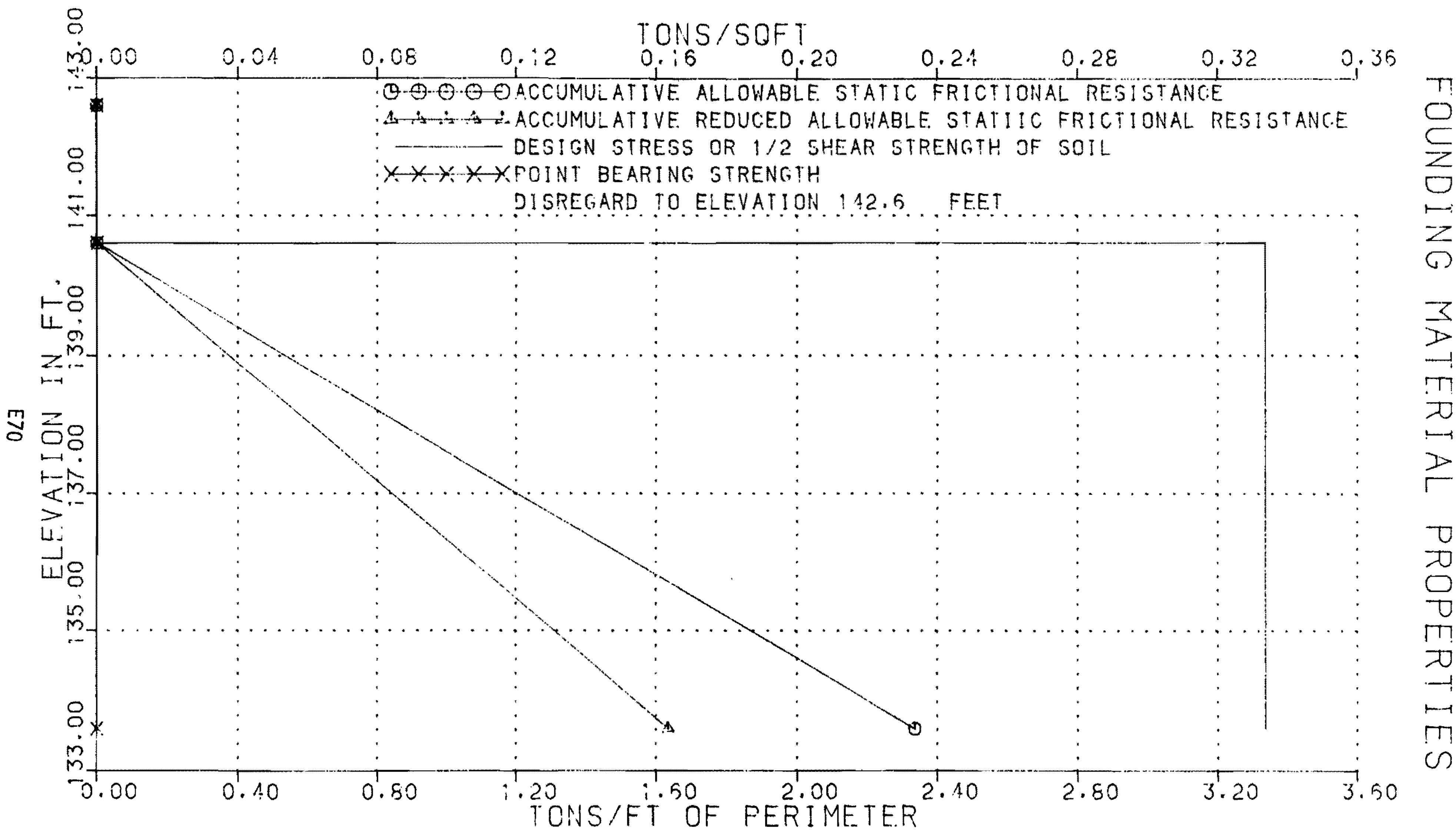
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W93	DATE	04/17/90
CONTROL	0050-05-033	STATION	1969+75.00	GRD. ELEV.	142.5
TPE		LOCATION	00.00'	GRD. WATER ELEV.	142.5

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.5 0				SAND. (CEMENT STABILIZED SAND).	C
140.5				CLAY-SANDY-GR-TAN-SOFT-MOIST.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W93	DATE	04/17/90
CONTROL	0050-06-033	STATION	1969+75.00	BY	PAZ
IPE		LOCATION	00.00		

TEST HOLE NO. W93

GROUND ELEV. 142.6

SAND. (CEMENT STABILIZED SAND).

CLAY, SANDY, GR-TAN, SOFT, MOIST.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.1
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W94
STATION- 1972+20.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.1

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1											SAND, (CEMENT STABILIZED SAND).
2											- NO RECOVERY 0-7'
3											
4											
5											
6											
7					1	0- 7	124	20	32		CLAY,SANDY,RD.BRN,FIRM,MOIST.
8											
9					2	0- 11	127	20	48		- RD.BRN-GR,MOTTLED.
10					3			31			
11					4	0- 12	122	36			
12											
13					6	0- 16	129	25			- STIFF.
14											
15											

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W94

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	142	135	7.0	3.5	OTHE								
2	135	127	8.0	11.0	CL			630	125	0.0	2.2	1.3	4.7

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.1 FEET

HOLE NUMBER W94

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.1	135.1	0.00	0.00	0.00	0.7	0.00	0.00	4.29	0.00	0.0
2	135.1	127.1	0.21	1.66	1.66	0.7	1.16	1.16	3.62	0.00	0.0

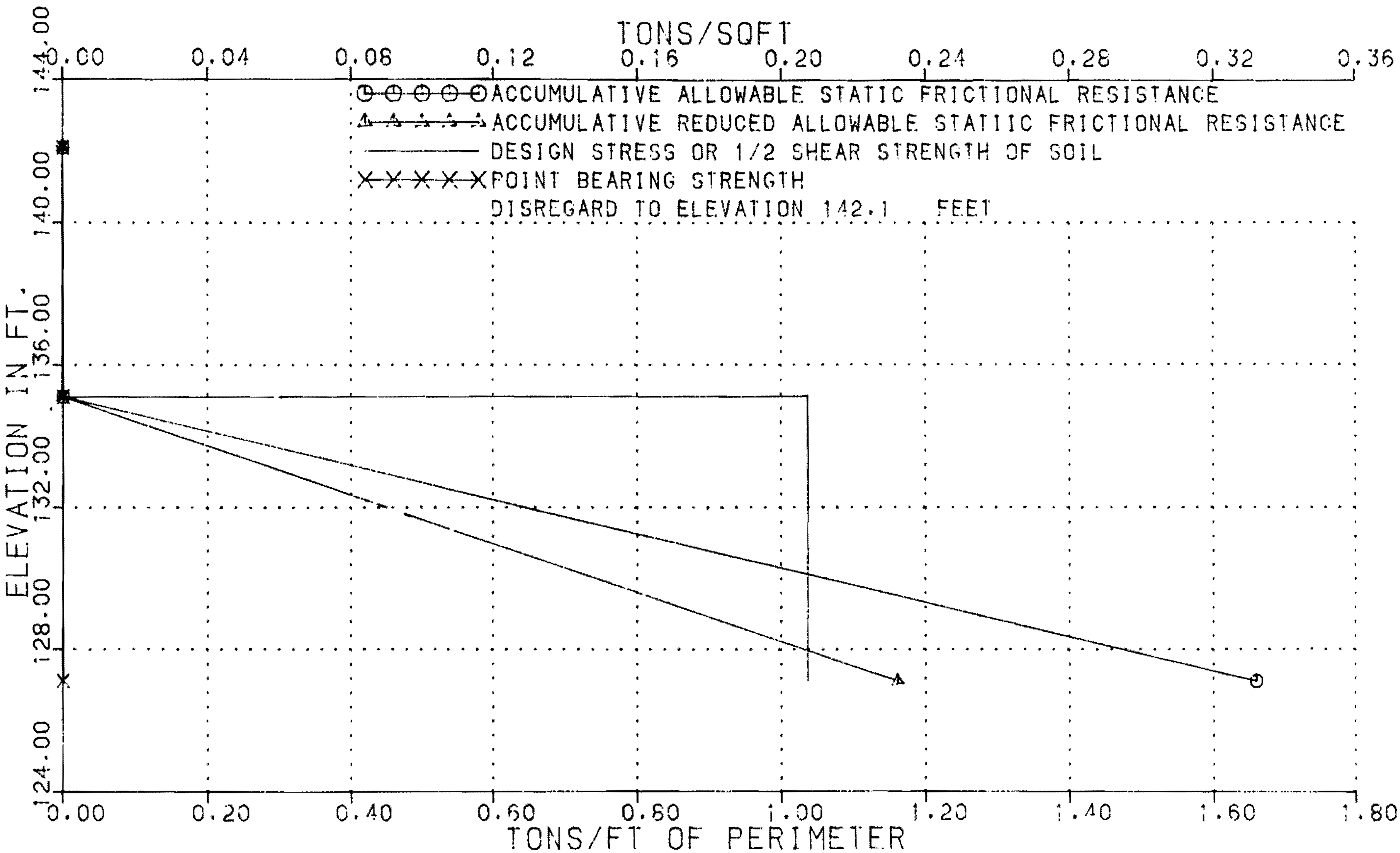
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W94	DATE	04/17/90
CONTROL	0050-06-033	STATION	1972+20.00	GRD. ELEV.	142.1
IPE		LOCATION	00.00'	GRD.WATER ELEV.	142.1

ELEV FT.	DEPTH	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.1	0				SAND. NO REC.TO 7'. STBL.CEMENT-3'	
135.1					CLAY.SANDY.RED BRN.FIRM.MOIST.	
	10					

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



FOUNDIN MATERIAL PROP RTIES

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W94	DATE	04/17/90
CONTROL	0050-05-033	STATION	972+20.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W94

GROUND ELEV. 142.1

SAND. NO REC. TO 7'. STBL.CEMENT-3'

CLAY,SANDY,RED BRN.FIRM,MOIST.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 142.3
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W95
STATION- 1972+25.00
OFFSET- 00 00'
CODED BY- PAZ
GRD. WATER ELEV.- 142.3

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											
1											SAND, (CEMENT STABILIZED SAND).
2											
3					1		129	17	30		CLAY,SANDY,GR-BLK,SOFT,MOIST.
4											
5					2	0- 11	133	24			- FIRM.
6											
7											
8					3	0- 7	128	26	49		
9											

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W95

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	142	140	2.0	1.0	OTHE								
2	140	133	7.0	5.5	CL			630	130	0.0			

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.3 FEET

HOLE NUMBER W95

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.3	140.3	0.00	0.00	0.00	0.7	0.00	0.00	4.29	0.00	0.0
2	140.3	133.3	0.16	1.10	1.10	0.7	0.77	0.77	3.62	0.00	0.0

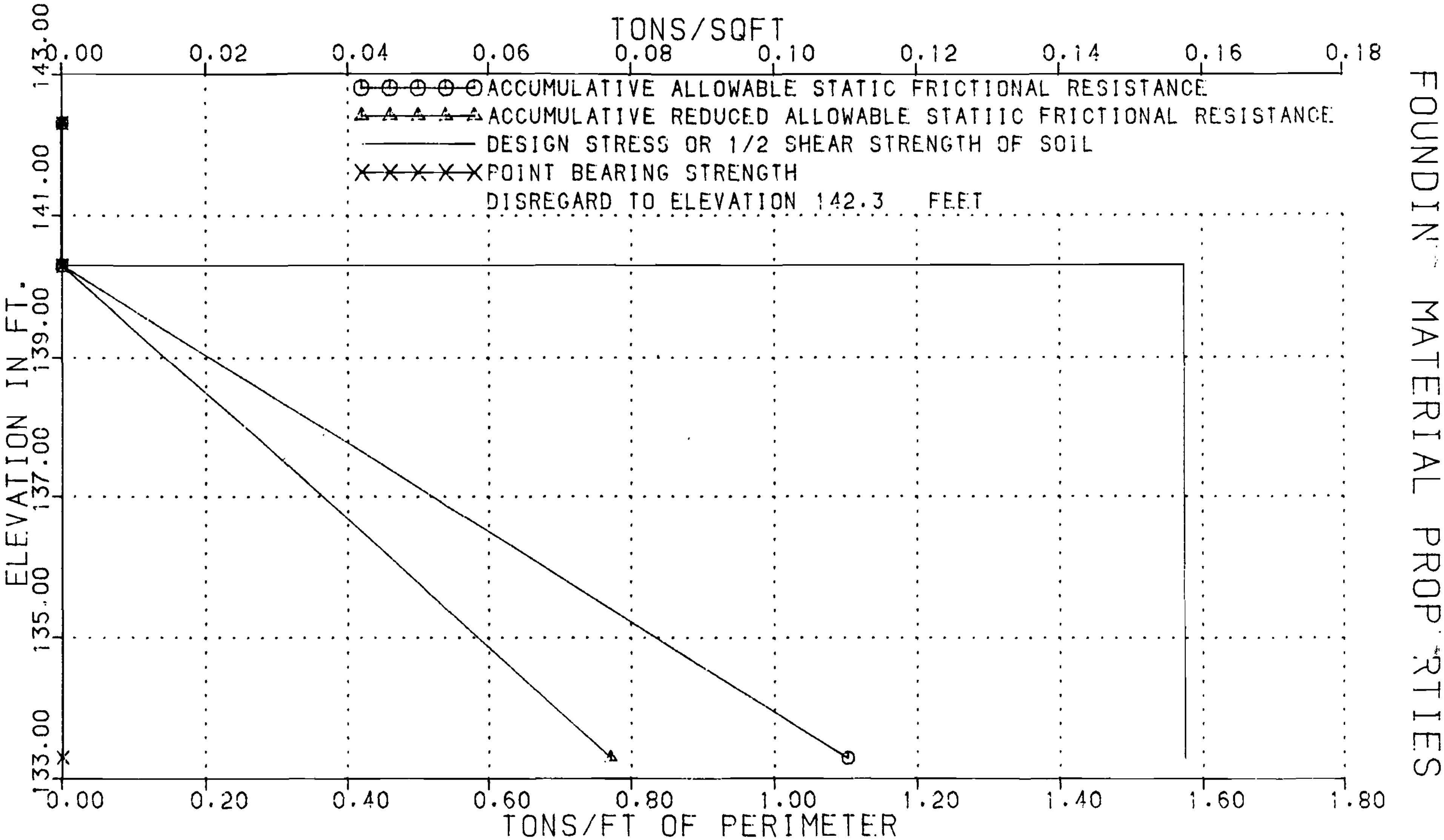
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL. THD DIST 12
HIGHWAY NO	US-290	HOLE NO	W95	DATE 04/17/90
CONTROL	0050-05-033	STATION	1972+25.00	GRD. ELEV. 142.3
IPE		LOCATION	00 00'	GRD.WATER ELEV. 142.3

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
142.3 0				SAND. (CEMENT STABILIZED SAND).	
140.3				CLAY.SANDY.GR-BLK.SOFT.MOIST.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



E82

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W95	DATE	04/17/90
CONTROL	0050-05-033	STATION	1972+25.00	BY	PAZ
IPE		LOCATION	00 00'		

TEST HOLE NO. W95

GROUND ELEV. 142.3

SAND. (CEMENT STABILIZED SAND).

CLAY, SANDY, GR-BLK, SOFT, MOIST.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 139.9
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W96
STATION- 1975+00.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 139.9

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									
1			1		124	10	32		CLAY,SANDY,RD.BRN-GR,SOFT,MOIST,
2			2						CALCAREOUS.
3			3		126	14			
4									
5									
6			4		123	24	45		
7									
8			5	0- 4	123	27	47		
9									
10			7	0- 4	121	29	52		
11									
12			8	0- 2	119	28			- V.SOFT.
13									SAND,RD.BRN,LOOSE,WET.
14			11	0- 21	136	11	18		- SL.COMPACT.
15			12	0- 21	136	12	16		

DRILLER- MIKE BAHM

LOGGER- LOGGER AL FERRALL E
TITLE- ENGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W96

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN-TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	140	128	12.0	6.0	CL			299	122	0.0			
2	128	125	3.0	13.5	SC			850	127	0.0			

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 139.9 FEET

HOLE NUMBER W96

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	139.9	127.9	0.07	0.90	0.90	0.7	0.63	0.63	3.62	0.00	0.0
2	127.9	124.9	0.21	0.64	1.53	0.7	0.45	1.07	4.29	0.00	0.0

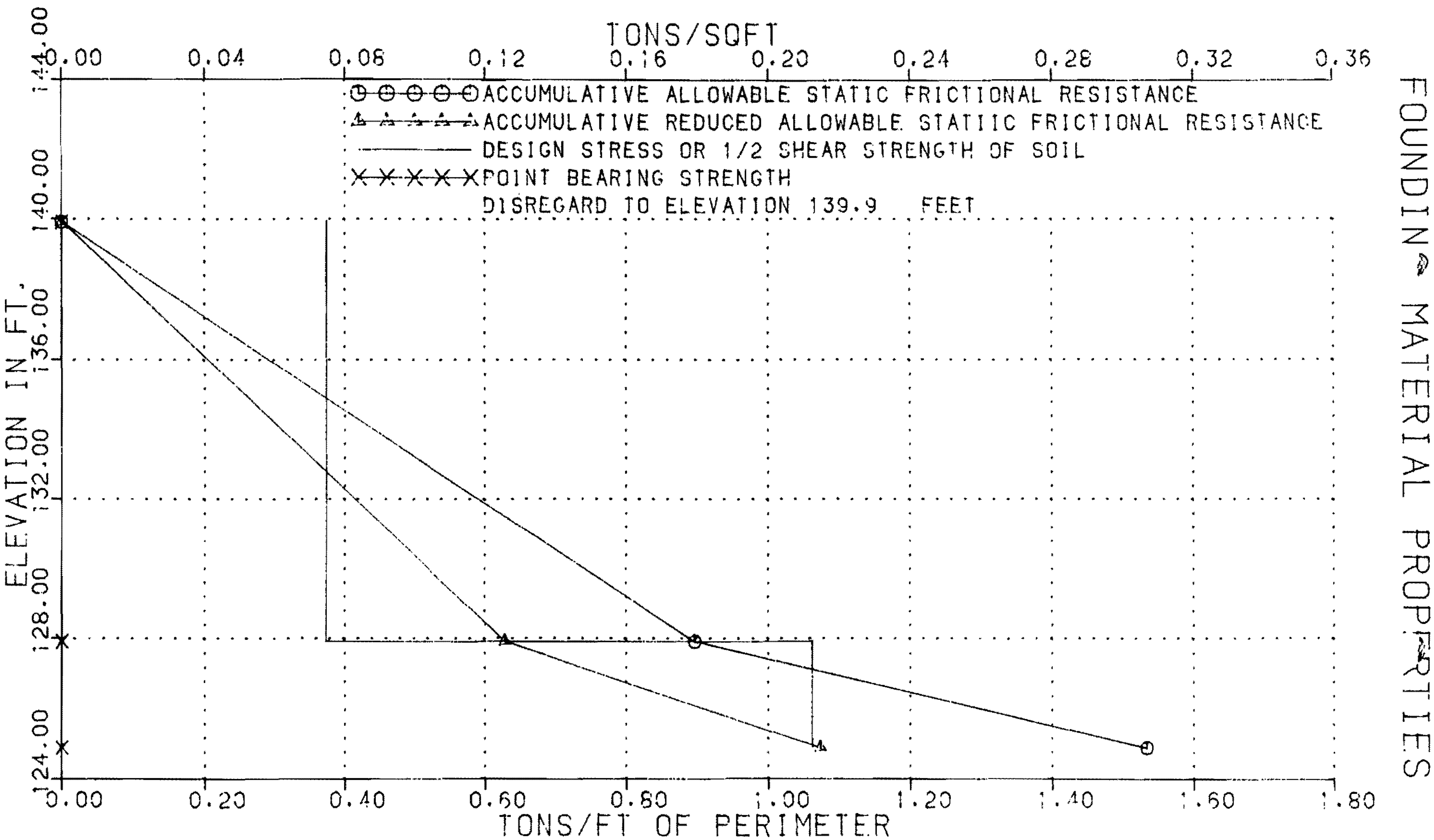
DRILLING LOG

COUNTY HARRIS
STRUCTURE US-290 CYPRESS BYPASS RET. WALL.
THD DIST 12
HIGHWAY NO US-290
HOLE NO W96
DATE 04/17/90
CONTROL 0050-06-033
STATION 1975+00.00
GRD. ELEV. 139.9
TYPE
LOCATION 00.00'
GRD.WATER ELEV. 139.9

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
139.9 0				0 CLAY.SANDY.RD.BRN-GR.SOFT.MOI.CALCS	
10				10	
127.9				SAND.RD.BRN.LOOSE.WET.	

*REMARKS:

DRILLER MIKE BAHM
LOGGER LOGGER AL FERRALL E
TITLE ENGR. TECH III



E88

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W96	DATE	04/17/90
CONTROL	0050-06-033	STATION	1975+00.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W96

GROUND ELEV. 139.9

CLAY,SANDY,RD.BRN-GR,SOFT,MOI,CALCS

SAND,RD.BRN,LOOSE,WET.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 139.7
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W97
STATION- 1977+00.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 139.7

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0	0									
1				2	0- 31	124	15	30		CLAY,SANDY,MULTICOLORED,V.STIFF,MOI
2				3		120	10			CALCAREOUS.
3										
4										
5				4	0- 7	130	22	34		- FIRM.
6				6	0- 28	136	19	42		- STIFF.
8				8	0- 17	126	31	56		CLAY,RD.BRN-GR,STIFF,MOIST,CALCS.
9										
10										
11				10	0- 32	130	26	67		- V.STIFF.
12				11	0- 35	135	27			
13										CLAY,SANDY,RD.BRN-GR,V.STIFF,MOIST.
14				12		131	22	42		
15										

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W97

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	140	133	7.0	3.5	CL			1579	130	0.0	7.6	5.1	3.4
2	133	128	5.0	9.5	CH			1760	128	0.0			
3	128	125	3.0	13.5	CL			2549	135	0.0			

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 139.7 FEET

HOLE NUMBER W97

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	139.7	132.7	0.39	2.76	2.76	0.7	1.93	1.93	3.62	0.00	0.0
2	132.7	127.7	0.44	2.20	4.96	0.7	1.54	3.47	2.57	0.00	0.0
3	127.7	124.7	0.64	1.91	6.88	0.7	1.34	4.81	3.62	0.00	0.0

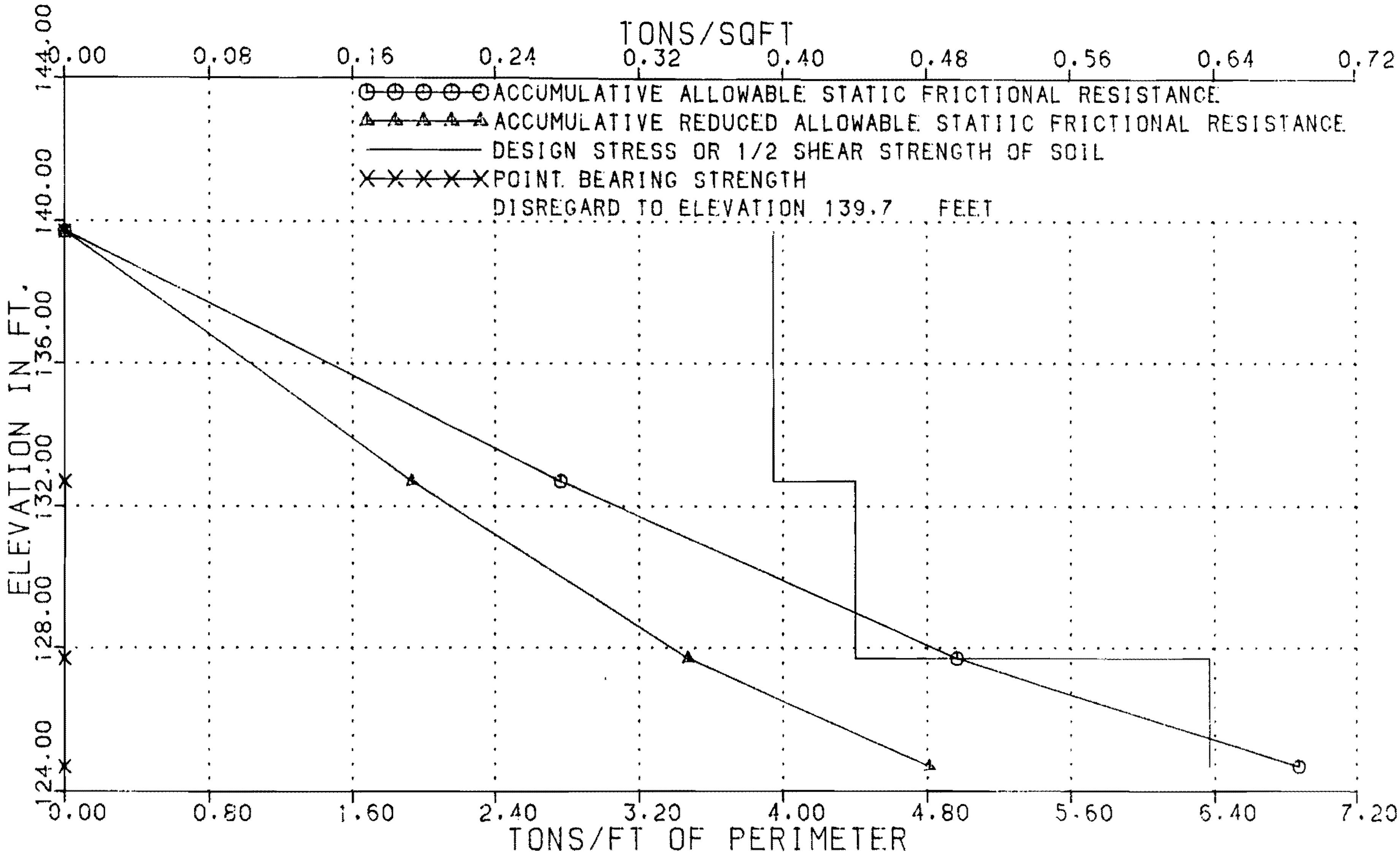
DRILLING LOG

COUNTY HARRIS STRUCTURE US-290 CYPRESS BYPASS RET. WALL. THD DIST 12
HIGHWAY NO US-290 HOLE NO W97 DATE 04/17/90
CONTROL 0050-06-033 STATION 1977+00.00 GRD. ELEV. 139.7
IPE LOCATION 00.00' GRD. WATER ELEV. 139.7

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
139.7 0				CLAY, SANDY, MULTICOLORED, SOFT, MOIST.	
132.7				CLAY, RD. BRN-GR, STIFF, MOIST, CALCS.	
10					
127.7				CLAY, SANDY, RD. BRN-GR, V. STIFF, MOIST.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III



FOUNDIN' MATERIAL PROP'RTIES

COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W97	DATE	04/17/90
CONTROL	0050-06-033	STATION	1977+00.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W97

GROUND ELEV. 139.7

CLAY, SANDY, MULTICOLORED, SOFT, MOIST.

CLAY, RD. BRN-GR, STIFF, MOIST, CALCS.

CLAY, SANDY, RD. BRN-GR, V. STIFF, MOIST.

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US-290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/17/90
GRD. ELEV.- 139.0
DISTRICT- 12

STRUCTURE- US-290 CYPRESS BYPASS RET. WALL.
HOLE NO.- W98
STATION- 1979+00.00
OFFSET- 00.00'
CODED BY- PAZ
GRD. WATER ELEV.- 139.0

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									
1			1	0- 17	138	14	68		CLAY,SANDY,MULTICOLORED,STIFF,MOIST
2									CALCAREOUS,FER.
3									
4			3		122	23	31		
5									
6									
7			5	0- 3	124	22	41		
8									
9									
10			7	0- 10	116	16	39		
11			8	0- 19	127	27	33		
12									SAND,LT.BRN-GR,LOOSE,MOIST.
13			10	0- 13	139	15	21		
14			12	0- 11	142	14	24		- WET.
15			13	0- 12	142	14			

DRILLER- MIKE BAHM
LOGGER- LOGGER AL FERRALL E
TITLE- NGR. TECH III

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER W98

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	139	128	11.0	5.5	CL			744	126	0.0	3.6	2.4	4.2
2	128	124	4.0	13.0	OTHE			1034	136	0.0	2.4	1.6	3.7

PROB (CONTD)
2165 (NOTE: NO SOIL IS DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 139.0 FEET

HOLE NUMBER W98

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	139.0	128.0	0.19	2.05	2.05	0.7	1.43	1.43	3.62	0.00	0.0
2	128.0	124.0	0.26	1.03	3.08	0.7	0.72	2.16	4.29	0.00	0.0

DRILLING LOG

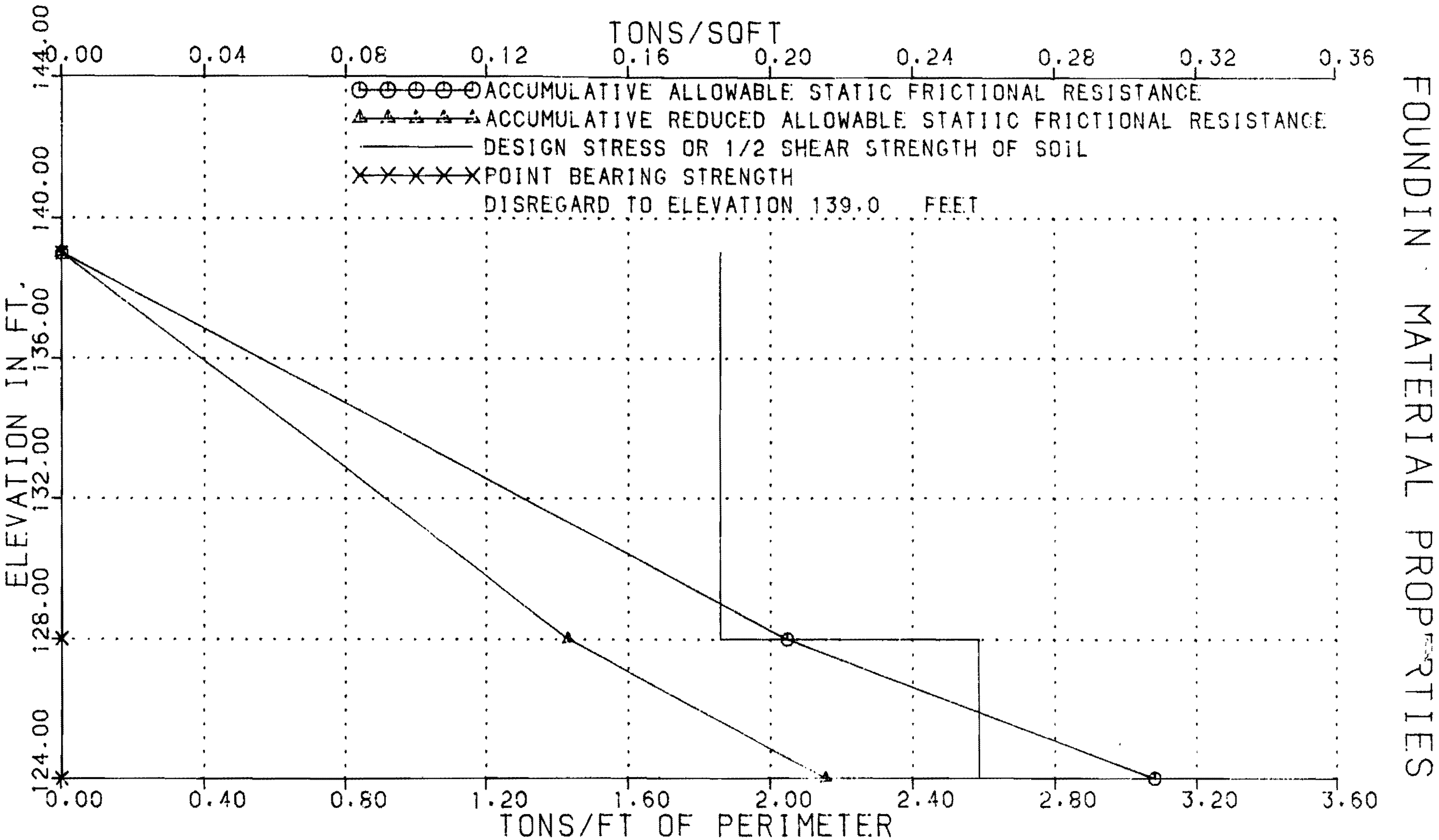
COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W98	DATE	04/17/90
CONTROL	0050-06-033	STATION	1979+00.00	GRD. ELEV.	139.0
IPE		LOCATION	00.00'	GRD. WATER ELEV.	139.0

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
139.0 0				CLAY, SANDY, MULTICOLORED, STIFF, MOI.	
10					
126.0				SAND, LT. BRN-GR, LOOSE, MOIST.	

*REMARKS:

DRILLER MIKE BAHM LOGGER LOGGER AL FERRALL E TITLE NGR. TECH III

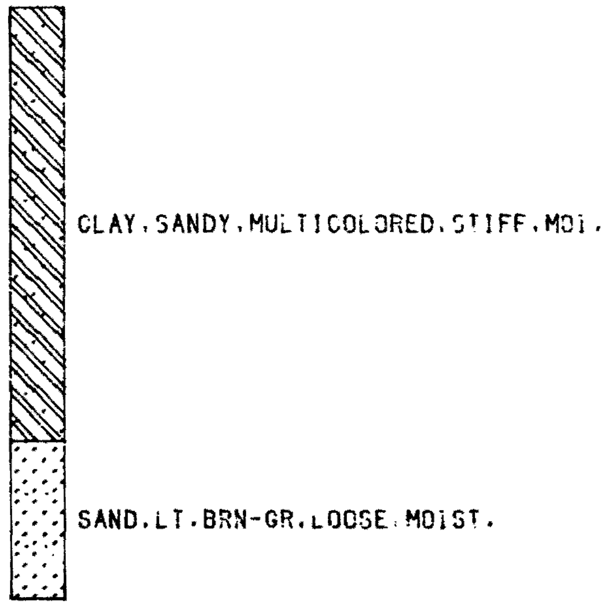
E100



COUNTY	HARRIS	STRUCTURE	US-290 CYPRESS BYPASS RET. WALL.	THD DIST	12
HIGHWAY NO	US-290	HOLE NO	W98	DATE	04/17/90
CONTROL	0050-05-033	STATION	1979+00.00	BY	PAZ
IPE		LOCATION	00.00'		

TEST HOLE NO. W96

GROUND ELEV. 139.0



TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

FAST BRIDGE FOUNDATION AND SOIL TEST PROG - 224168 VER 1.6 OCT 84

PROB (CONTD)

1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

DRILLING REPORT

COUNTY-	HARRIS	STRUCTURE-	OVERPASS
HIGHWAY NO.-	US 290	HOLE NO.-	1
CONTROL-	0050-06-033	STATION-	1969+07.13
PROJECT NO.-		OFFSET-	3.28 FT. RT
DATE-	04/01/85	CODED BY-	SFY
GRD. ELEV.-	144.0	GRD. WATER ELEV.-	144.0
DISTRICT- 12			

DEPTH FT.	PEN 1ST 6IN	PEN. 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									FISH-TAILED TO 3 FT.
1									
2									
3									
4			1			14	24		OLIVE GY.,SL.SILTY,SANDY CL. W/CALC
5			2		131	14			-W/BLACK NODULES
6	4	4							
7			3	0- 6	122	16	30		FIRM,GRAY,YELLOWISH BN,SILTY CLAY
8			4	0- 12	131	16			-W/FERROUS NODULES
9			5	0- 12	132	19	37		
10			6	0- 18	134	20			
11	7	6							
12			7		134	15	31		-LIGHT GRAY
13			8	0- 8	129	25			-W/CALCS
14			9	0- 14	130	14	50		-W/BLACK NODULES
15			10	0- 26	131	18	64		-W/CALC.
16	10	11							
17			11	0- 36	126	30			-W/CALC.
18			12	0- 11	131	31			-W/CALC.
19			13		145	14	21		SLIGHTLY COMPACT,RED,GY,SILTY SAND
20			14		134	15	20		
21	14	11							
22									NO RECOVERY
23									
24			15	0- 8	140	13	20		COMPACT,GRAY,BRNISH YEL,SANDY CLAY
25			16	0- 10	147	16	30		FIRM BLACK GRAY SILTY CLAY
26	20	37							
27									SAND-HARD-COULD NOT PUSH
28									
29									
30									
31	50/6.00	50/5.50							USING-WASHING-PEN
32									WASHED-OUT
33									
35									

36'	16	17						
37			17	0- 37	131	20		HARD,GRAY,RED,SLIGHT SILTY CLAY
38			18	0- 50	132	19	56	
39			19	0- 39	133	19		-LIGHT GY,REDDISH YEL,W/BL. NOD
40			20	0- 32	134	18	48	-SAME-
41	15	17						
42			21			19		FIRM,LT.GRAY,BROWN,SANDY CLAY
43			22	0- 9	132	19	32	
44								SAND-HARD-COULD NOT PUSH
45								
46	46	40						USING-WASHING-PEN
47								WASHED-OUT
48								
49								
50								
51	50/3.00	50/2.25						
52								
53								
54								
55								
56	50/3.25	50/2.75						
57								
58								
59								
60								
61	12	12						
62								
63								
64								
65								
66	21	23						

RILLER- W. WILLMAN/M. BAHM LOGGER- R. K. CURSON
TITLE-

PROB (CONTD)
1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER 1

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN-TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	144	141	3.0	1.5	OTHE								
2	141	139	2.0	4.0	OTHE								
3	139	126	13.0	11.5	CL	14.0	60	1190	129	0.0	9.6	3.8	5.6
4	126	121	5.0	20.5	OTHE	25.0	80	821	131	0.0			
5	121	118	3.0	24.5	OTHE			659	143	0.0			
6	118	108	10.0	31.0	OTHE	80.7	80						
7	108	103	5.0	38.5	CH	33.0	50	2860	132	0.0	5.1	2.6	4.9
8	103	101	2.0	42.0	CL	32.0	60						
9	101	88	13.0	49.5	OTHE	157.3	80	677	132	0.0			
10	88	81	7.0	59.5	OTHE	112.0	80						
11	81	77	4.0	65.0	OTHE	44.0	80						

PROB (CONTD)
1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.0 FEET

HOLE NUMBER 1

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	144.0	141.0	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
2	141.0	139.0	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
3	139.0	126.0	0.30	3.87	3.87	0.5	1.93	1.93	3.62	0.00	1.3
4	126.0	121.0	0.21	1.03	4.89	0.5	0.51	2.45	4.29	0.00	0.7
5	121.0	118.0	0.16	0.49	5.39	0.5	0.25	2.69	4.29	0.00	0.0
6	118.0	108.0	1.01	10.08	15.47	0.5	5.04	7.74	4.29	0.00	0.0
7	108.0	103.0	0.72	3.58	19.05	0.5	1.79	9.52	2.57	0.00	1.1
8	103.0	101.0	0.53	1.07	20.11	0.5	0.53	10.06	3.62	0.00	0.0
9	101.0	88.0	0.17	2.20	22.31	1.0	2.20	12.26	4.29	0.00	0.2
10	88.0	81.0	0.85	5.95	28.26	1.0	5.95	18.21	4.29	0.00	0.0
11	81.0	77.0	0.55	2.20	30.46	0.5	1.10	19.31	4.29	0.00	0.0

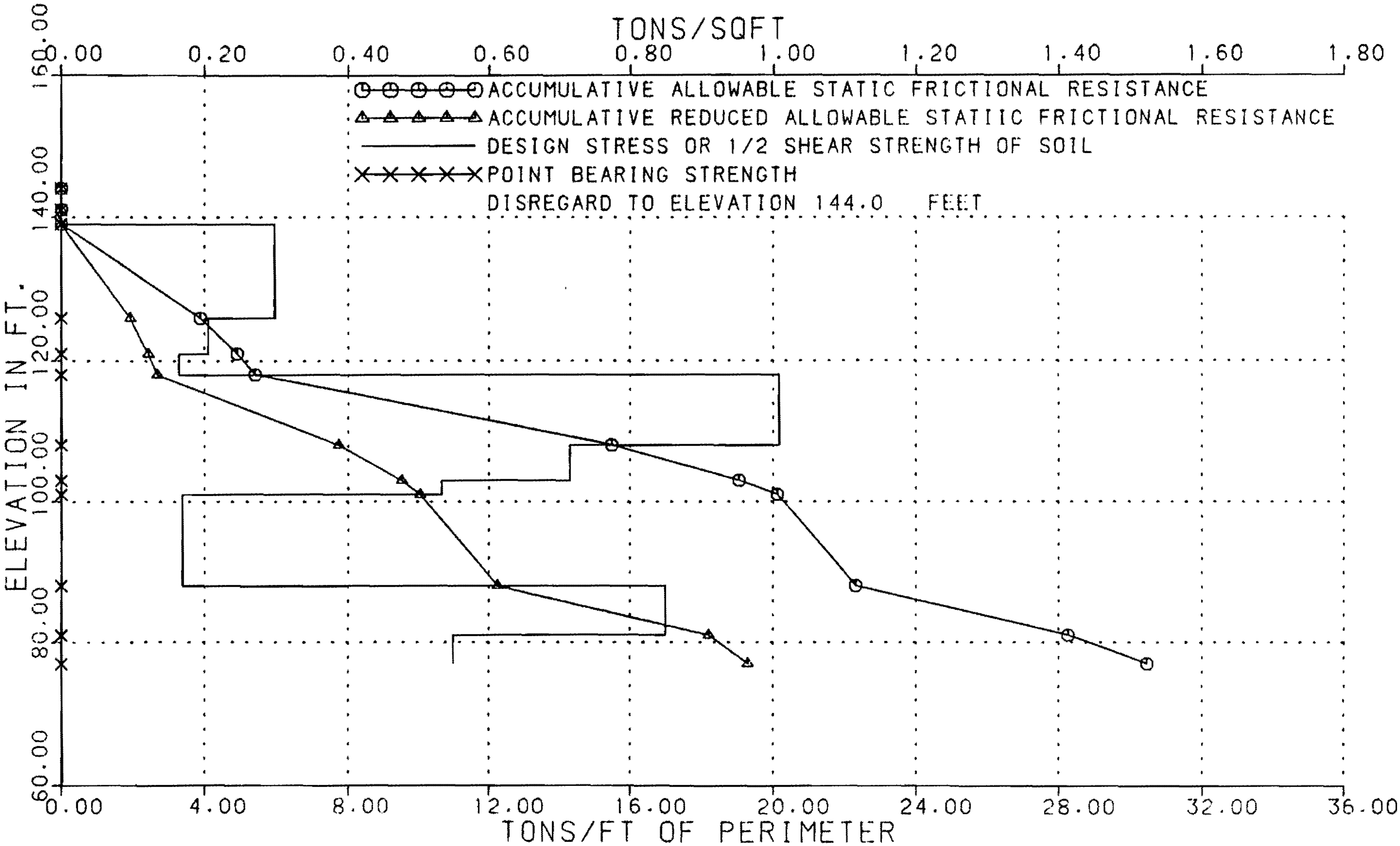
DRILLING LOG

COUNTY HARRIS | STRUCTURE BEAST | THD DIST 12
HIGHWAY NO US 290 | HOLE NO 1 | DATE 04/01/85
CONTROL 0050-06-033 | STATION 1969+07.13 | GRD. ELEV. 144.0
IPE | LOCATION 3.28 FT. RT | GRD.WATER ELEV. 144.0

ELEV FT.	DEPTH	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
144.0	0				FISH-TAILED TO 3 FT.	
141.0					OLIVE GRAY.SL. SI..SANDY CLAY	
139.0			4 (6.0")	4 (6.0")	FIRM..GRAY.YELLOWISH BN..SILTY CLAY	
	10		7 (6.0")	6 (6.0")		
			10 (6.0")	11 (6.0")		
126.0					SLIGHTLY COMPACT.RD.GRAY.SILTY SAND	
	20		14 (6.0")	11 (6.0")		
121.0			20 (6.0")	37 (6.0")	COMPACT.GRAY.BRNISH YEL..SANDY CLAY	
118.0					DENSE SAND	
	30		50 (6.0")	50 (5.5")		
			16 (6.0")	17 (6.0")		
108.0					HARD.GRAY.RED.SLIGHT SILTY CLAY	
	40		15 (6.0")	17 (6.0")		
103.0					FIRM.LT. GY..BN..SANDY CLAY	
101.0					DENSE SAND	
			46 (6.0")	40 (6.0")		
	50		50 (3.0")	50 (2.2")		
			50 (3.2")	50 (2.7")		
88.0					SLIGHTLY COMPACT SAND	
	60		12 (6.0")	12 (6.0")		
81.0			21 (6.0")	23 (6.0")	COMPACT SAND	

*REMARKS:

DRILLER W. WILLMAN/M. BAHM LOGGER R. K. CURSON TITLE



E107

FOUNDING MATERIAL PROPERTIES

COUNTY	HARRIS	STRUCTURE	BFAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	1	DATE	04/01/85
CONTROL	0050-06-033	STATION	1969+07.13	BY	SFY
IPE		LOCATION	3.28 FT. RT		

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

FAST BRIDGE FOUNDATION AND SOIL TEST PROG - 224168 VER 1.6 OCT 84

PROB (CONTD)
1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US 290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 04/01/85
GRD. ELEV.- 144.4
DISTRICT- 12

STRUCTURE- OVERPASS
HOLE NO.- 2
STATION- 1966+68.68
OFFSET- 10.6 FT. RT
CODED BY- SFY
GRD. WATER ELEV.- 144.4

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									NO RECOVERY
1									
2									
3			1		142	16	25		SL. COMP.,GY,SL.CL.,SA-SI MIX W/ORG
4			2	0- 5	137	18			
5	5	5							
6									
7			3	0- 4	138	14	29		
8									
9									
10	6	7							
11			4	0- 4	125	18			FIRM,TAN,SILTY CLAY
12			5	0- 15	133	28	48		
13			6	0- 14	128	23	52		
14			7	0- 11	115	26			-W/CALC. NODS & SILT STONE LAYERS
15	10	10							
16									SLIGHT COMPACT,BROWN SILTY SAND
17			8	0- 17	130	25	63		-BECOMING MORE SILTY
18			9	0- 12	132	13	39		
19			10			16			
20	12	20							
21									
22									COMPACT,BROWN,SILTY SAND
23			11			16			-WATER BEARING
24			12			16			
25	20	26							
26									
27									
28									
29			13			17	23		
30	12	16							
31									NO RECOVERY
32									
33									
34									

36			14	0- 40	139	19		HARD,LT.GY.,TAN,SILTY CLAY W/BL.NOD
37			15	0- 57	137	18	49	
38			16	0- 54	138	18		
39			17	0- 43	138	19	46	
40	15	16						
41			18	0- 42	132	21		HARD,LT. GY,TAN,V. SL. SILTY CLAY
42			19	0- 51	130	24	57	-W/BLACK NODULES
43			20	0- 39	133	22	57	
44			21	0- 37	132	19		
45	15	40						
46								COMPACT,TAN,FINE & MD. SAND W/SI PK
47								-WATER BEARING
48			22			23	25	
49			23			22		
50	50/4.00	48/6.00						
51								
52								
53			24			23	21	-WITHOUT SILT POCKETS
54								
55	47	48						
56								NO RECOVERY
57								
58								
59								
60	40	26						
61								
62								
63								
64								
65	10	13						

'RILLER- W. WILLMAN/M. BAHM   LOGGER- R. K. CURSON
TITLE-

PROB (CONTD)
1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER 2

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	144	142	2.0	1.0	OTHE								
2	142	139	3.0	3.5	OTHE			360	137	0.0			
3	139	134	5.0	7.5	CL	10.0	60	317	138	0.0			
4	134	128	6.0	13.0	CL	16.5	60	805	125	0.0	3.4	1.7	4.4
5	128	122	6.0	19.0	OTHE	32.0	80	1040	131	0.0			
6	122	114	8.0	26.0	OTHE	46.0	80						
7	114	109	5.0	32.5	OTHE	28.0	80						
8	109	103	6.0	38.0	CL	30.0	60	3492	138	0.0	4.3	3.4	3.5
9	103	99	4.0	43.0	CH			3049	132	0.0	4.3	2.1	4.5
10	99	89	10.0	50.0	OTHE	86.3	80						
11	89	78	11.0	60.5	OTHE	61.3	80						

PROB (CONTD)
1099 CYPRESS BYPASS(SKINNER:HOLE 1&2,SPRING CYPRESS:3&4,CYPRS ROSEHILL:5&6)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.4 FEET

HOLE NUMBER 2

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	144.4	142.4	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
2	142.4	139.4	0.09	0.27	0.27	0.5	0.13	0.13	4.29	0.00	0.0
3	139.4	134.4	0.08	0.40	0.67	0.5	0.20	0.33	3.62	0.00	0.5
4	134.4	128.4	0.20	1.21	1.87	0.5	0.60	0.94	3.62	0.00	0.7
5	128.4	122.4	0.26	1.56	3.43	0.5	0.78	1.72	4.29	0.00	0.7
6	122.4	114.4	0.57	4.60	8.03	0.5	2.30	4.02	4.29	0.00	0.0
7	114.4	109.4	0.35	1.75	9.78	0.5	0.87	4.89	4.29	0.00	0.0
8	109.4	103.4	0.87	5.24	15.02	0.5	2.62	7.51	3.62	0.00	1.7
9	103.4	99.4	0.76	3.05	18.07	0.5	1.52	9.04	2.57	0.00	0.0
10	99.4	89.4	1.08	10.79	28.86	0.5	5.39	14.43	4.29	0.00	0.0
11	89.4	78.4	0.77	8.43	37.29	0.5	4.22	18.65	4.29	0.00	0.0

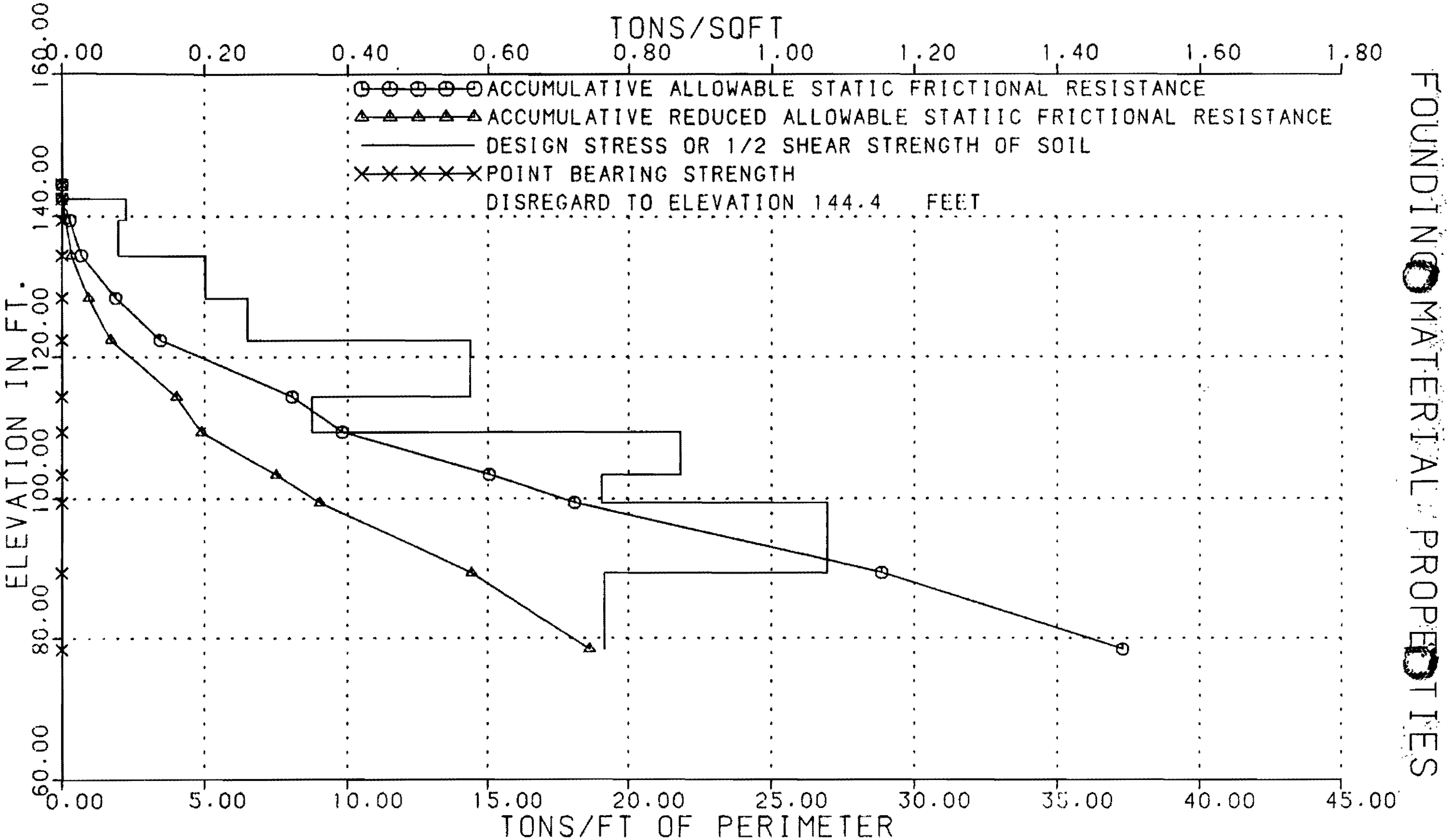
DRILLING LOG

COUNTY HARRIS | STRUCTURE BFAST | THD DIST 12
HIGHWAY NO US 290 | HOLE NO 2 | DATE 04/01/65
CONTROL 0050-06-083 | STATION 1966+66.66 | GRD. ELEV. 144.4
IPE | LOCATION 10.6 FT. RT | GRD.WATER ELEV. 144.4

ELEV FT.	LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	DEPTH	METHOD OF CORING
144.4 0					0	
142.4				NO RECOVERY TO 2 FT		
139.4		5 (6.0")	5 (6.0")	SL. COMP..CY.SL.CL..SA-SI MIX W/ORG		
		6 (6.0")	7 (6.0")	SOFT.TAN.SILTY CLAY W/FER. NOD.	10	
134.4 10		10 (6.0")	10 (6.0")	FIRM.TAN.SILTY CLAY		
128.4		12 (6.0")	20 (6.0")	SLIGHT COMPACT.BROWN SILTY SAND	20	
20						
122.4		20 (6.0")	26 (6.0")	COMPACT.WATER BEARING.BN.SILTY SAND		
		12 (6.0")	16 (6.0")		30	
114.4 30		13 (6.0")	16 (6.0")	NO RECOVERY		
109.4		15 (6.0")	16 (6.0")	HARD.LT.GY.TAN.SILTY CLAY W/BL. NOD	40	
40						
103.4		15 (6.0")	40 (6.0")	HARD.LT.GY.TAN.V.SL.SI.CL. W/BL.NOD		
99.4		50 (4.0")	46 (6.0")	COMPACT.TAN.FINE & MD.SAND W/SI PKT	50	
50		47 (6.0")	46 (6.0")			
89.4		40 (6.0")	26 (6.0")	NO RECOVERY	60	
60		10 (6.0")	13 (6.0")			

*REMARKS:

DRILLER W. WILLMAN/M. BAHM LOGGER R. K. CURSON TITLE



E113

COUNTY	HARRIS	STRUCTURE	BFAST	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	2	DATE	04/01/85
CONTROL	0050-06-033	STATION	1966+68.68	BY	SFY
IPE		LOCATION	10.6 FT. RT		

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US 290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 05/01/86
GRD. ELEV.- 143.0
DISTRICT- 12

STRUCTURE- BRIDGE @ SKINNER ROAD
HOLE NO.- SK3
STATION- 1971+01
OFFSET- 19.6 FT RT
CODED BY- SJH
GRD. WATER ELEV.- 143.0

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									NO CORE,BLACK,CLAYEY SAND TO 4 FT.
1									
2									
3									
4									
5	5	3							
6									PEN-CUTTINGS-NO-CORE,SAME AS BELOW
7			1			15			LIGHT GRAY,TAN,CLAYEY SAND
8			2	0- 19	137	15	28		-STIFF
9			3	0- 15	140	16			-STIFF
10	3	3							
11									PEN-CUTTINGS-NO-CORE,SAME AS BELOW
12			4		130	22			GRAY,TAN,BROWN CLAY
13			5	0- 24	134	18	54		-VERY STIFFY
14			6	0- 33	135	16			-HARD
15	6	7							
16									PEN-CUTTINGS-NO-CORE,SAME AS BELOW
17			7	0- 25	132	23	53		VERY STIFF,BROWN,GRAY CLAY W/CALCS.
18			8			16			BROWN,GRAY SAND W/V. SL. CLAY
19			9		134	11	21		
20	10	11							
21									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
22			10		139	15			GRAY SAND W/VERY SLIGHT CLAY
23			11		142	13	16		
24			12		140	15			
25	23	32							
26									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
27			13		135	14			GRAY SAND W/VERY SLIGHT CLAY
28			14		138	14	25		
29			15			14			
30	50/3.35	29/6.00							
31			16	0- 53	140	16			HARD,LIGHT GRAY CLAY
32			17	0- 59	139	17	46		-HARD
33			18	0- 60	138	16			-HARD
34			19	0- 58	138	17	50		-HARD
35	15	16							

37			21	0- 32	138	16	49	-HARD
38			22	0- 59	137	18		-HARD
39			23	0- 41	136	18	36	-HARD
40	20	33						
41								PEN-CUT,NO CORE,MAT'L SAME AS BELOW
42			24			17		GRAY,TAN,SILTY SAND
43			25			18	25	
44			26			19		
45	50/3.00	50/2.35						
46								NO RECOVERY,DENSE SAND
47								
48								
49								
50	50/5.00	50/6.00						
51								PEN-CUTTINGS-NO-CORE,SAME AS BELOW
52			27			19	20	GRAY,SILTY SAND
53			28			18		BROWN,SILTY SAND
54			29			17		
55	27	18						
56								PEN-CUT,NO-CORE,MAT'L SAME AS BELOW
57			30			21		GRAY,SILTY SAND
58			31			21		
59			32			29	41	BROWN,SILTY,SANDY CLAY
60	29	18						
61								PEN-CUTTINGS-NO-CORE,SAME AS BELOW
62			33			18		LIGHT BROWN SAND
63			34			18		
64			35	0- 24	125	32	62	VERY STIFF,BROWN CLAY
65	12	14						
66			36	0- 60	139	12		HARD,GRAY,TAN CLAY
67			37	0- 60	139	11	33	HARD,GRAY,TAN,SANDY CLAY
68			38	0- 60	139	12		-HARD
69			39	0- 60	141	12	34	-HARD
70	25	23						
71			40	0- 51	135	18		-HARD
72			41	0- 47	136	18	39	-HARD
73			42	0- 52	139	17		-HARD
74			43	0- 30	138	13	23	HARD,GRAY,TAN,CLAYEY SAND
75	46/6.00	54/4.50						

DRILLER- MIKE BAHM LOGGER- R. CURSON,JOHN HOES

TITLE-

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER SK3

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE PEAK	AVG	TAT RATIO
1	143	138	5.0	2.5	SC								
2	138	133	5.0	7.5	SC	8.0	70	1246	138	0.0			
3	133	126	7.0	13.5	CH	9.5	50	2052	134	0.0			
4	126	113	13.0	23.5	OTHE	38.0	80	1793	132	0.0			
5	113	108	5.0	32.5	CH	101.4	50	4142	139	0.0	2.3	1.1	4.3
6	108	103	5.0	37.5	CL	31.0	60	3425	137	0.0	7.7	5.5	3.4
7	103	99	4.0	42.0	OTHE	53.0	80						
8	99	93	6.0	47.0	OTHE	224.3	80						
9	93	85	8.0	54.0	OTHE	77.0	80						
10	85	83	2.0	59.0	CL								
11	83	80	3.0	61.5	OTHE	47.0	80						
12	80	77	3.0	64.5	CH	26.0	50	1742	125	0.0			
13	77	70	7.0	69.5	CL	48.0	60	4058	138	0.0	4.5	2.5	3.7
14	70	67	3.0	74.5	SC	114.3	70	2956	138	0.0			

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 143.0 FEET

HOLE NUMBER SK3

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUMULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUMULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	143.0	138.0	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
2	138.0	133.0	0.31	1.56	1.56	0.5	0.78	0.78	4.29	0.00	2.7
3	133.0	126.0	0.51	3.59	5.15	0.5	1.80	2.57	2.57	0.00	2.7
4	126.0	113.0	0.45	5.83	10.97	0.5	2.91	5.49	4.29	0.00	0.9
5	113.0	108.0	1.04	5.18	16.15	1.0	5.18	10.66	2.57	0.00	1.2
6	108.0	103.0	0.86	4.28	20.43	0.5	2.14	12.81	3.62	0.00	1.7
7	103.0	99.0	0.66	2.65	23.08	0.5	1.32	14.13	4.29	0.00	0.0
8	99.0	93.0	1.25	7.50	30.58	1.0	7.50	21.63	4.29	0.00	0.0
9	93.0	85.0	0.96	7.70	38.29	0.5	3.85	25.48	4.29	0.00	0.0
10	85.0	83.0	0.00	0.00	38.29	0.5	0.00	25.48	3.62	0.00	0.0
11	83.0	80.0	0.59	1.76	40.05	0.5	0.88	26.36	4.29	0.00	0.0
12	80.0	77.0	0.44	1.31	41.36	0.5	0.65	27.02	2.57	0.00	0.8
13	77.0	70.0	1.01	7.10	48.46	0.5	3.55	30.57	3.62	0.00	1.3
14	70.0	67.0	0.74	2.22	50.68	1.0	2.22	32.79	4.29	0.00	0.9

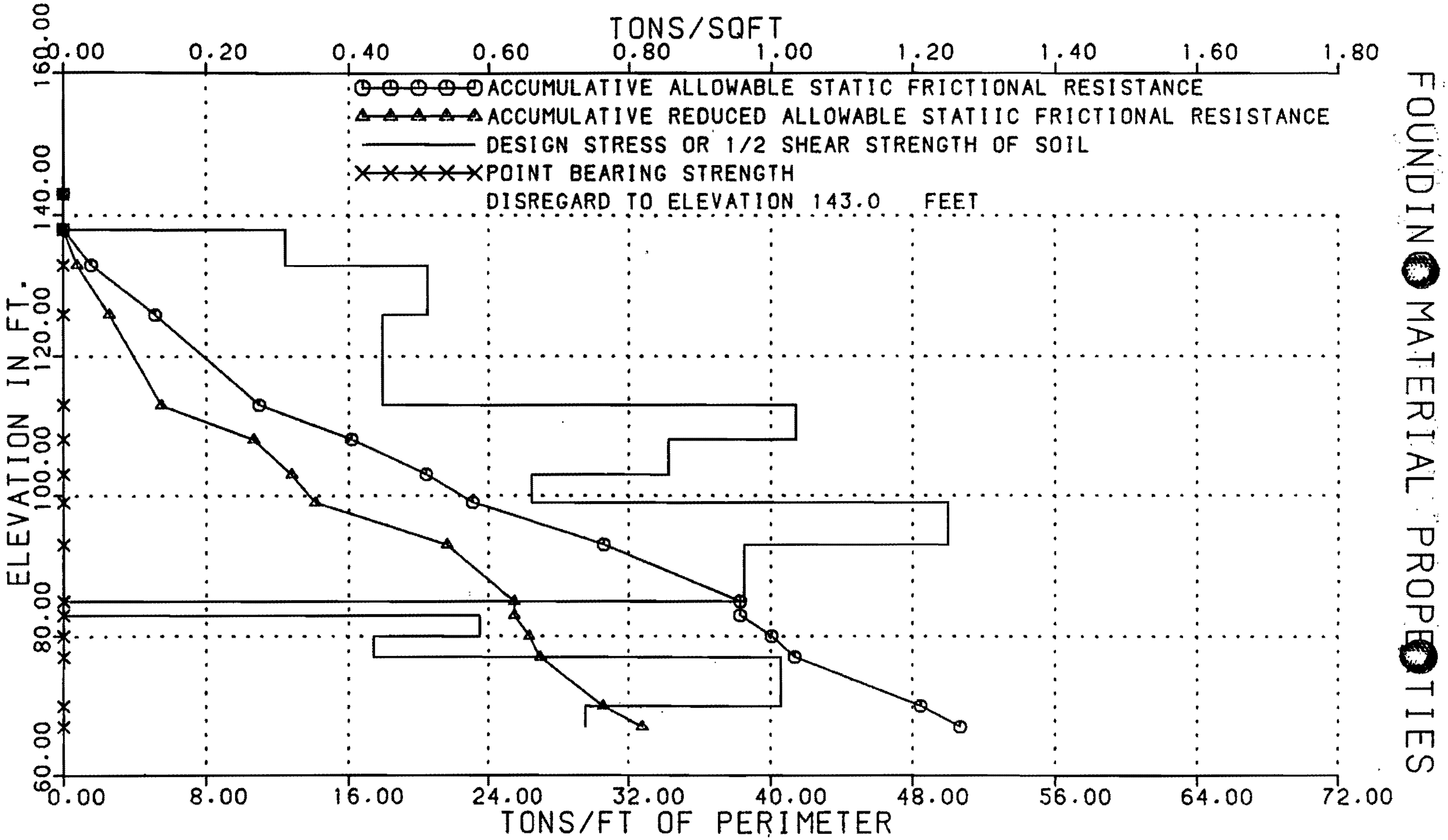
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	BRIDGE • SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK3	DATE	05/01/86
CONTROL	0050-06-033	STATION	1971+01	GRD. ELEV.	143.0
IPE		LOCATION	19.6 FT RT	GRD.WATER ELEV.	143.0

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
						0	
143.0	0				CLAYEY SAND,NO CORE		
			5 (6.0")	3 (6.0")			
138.0					STIFF,LIGHT GRAY,TAN,CLAYEY SAND		
			3 (6.0")	3 (6.0")		10	
133.0	10				VERY STIFF,GRAY,BROWN CLAY		
			6 (6.0")	7 (6.0")			
126.0					GRAY SAND		
	20		10 (6.0")	11 (6.0")		20	
			23 (6.0")	32 (6.0")			
			50 (3.3")	29 (6.0")		30	
113.0	30				HARD,LIGHT GRAY CLAY		
			15 (6.0")	16 (6.0")			
108.0					HARD,GRAY,SANDY CLAY		
			20 (6.0")	33 (6.0")		40	
103.0	40				GRAY,TAN,SILTY SAND		
99.0			50 (3.0")	50 (2.3")	NO RECOVERY,DENSE SAND		
			50 (5.0")	50 (6.0")		50	
93.0	50				GRAY,SILTY SAND		
			27 (6.0")	18 (5.0")			
85.0			29 (6.0")	18 (6.0")	BROWN,SILTY,SANDY CLAY	60	
83.0	60				LIGHT BROWN SAND		
80.0			12 (6.0")	14 (6.0")	HARD,GRAY,BROWN CLAY		
77.0					HARD,GRAY,TAN,SANDY CLAY		
	70		25 (6.0")	23 (6.0")		70	
70.0			46 (6.0")	54 (4.5")	HARD,GRAY,TAN,CLAYEY SAND		

•REMARKS: GWE OBSD. • END OF DRLG.

DRILLER MIKE BAHM LOGGER R. CURSON,JOHN HOES TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK3	DATE	05/01/86
CONTROL	0050-06-033	STATION	1971+01	BY	SJH
IPE		LOCATION	19.6 FT RT		

E119

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US 290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 05/01/86
GRD. ELEV.- 142.3
DISTRICT- 12

STRUCTURE- BRIDGE @ SKINNER ROAD
HOLE NO.- SK4
STATION- 1972+76
OFFSET- 21.7 FT RT
CODED BY- SJH
GRD. WATER ELEV.- 142.3

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0									NO RECOVERY TO 4 FT.
1									
2									
3									
4									
5	6	4							
6									PEN-CUT,NO-CORE,MAT'L SAME AS BELOW
7									
8			1	0- 23	138	15	33		VERY STIFF,GRAY,TAN,SANDY CLAY
9			2	0- 24	140	14			-VERY STIFF
10	3	3							
11									PEN-CUT,NO-CORE,MAT'L SAME AS BELOW
12			3			28			GRAY,BROWN CLAY
13			4	0- 18	131	33	84		-STIFF W/SOME CALCS.
14			5	0- 31	133	22			-HARD
15	6	6							
16			6		137	15			GRAY,TAN SAND W/VERY SLIGHT CLAY
17			7		136	15	18		
18			8		135	18			
19			9		142	14	19		
20	16	12							
21			10	0- 10	138	13			FIRM,GRAY,TAN,SILTY,CLAYEY SAND
22			11		140	14	25		
23			12	0- 11	140	15			
24			13	0- 9	138	15	25		
25	12	19							
26			14			13			LIGHT GRAY SAND
27			15		139	13			
28			16		135	16	21		
29			17		132	16			
30	31	15							
31			18			15			LIGHT GRAY,SANDY CLAY
32			19	0- 46	141	16	46		-HARD
33			20	0- 58	140	15			-HARD
34			21	0- 51	139	17	40		-HARD
35	16	18							

37			23	0- 35	134	21	27	-HARD
38			24	0- [illegible]9	135	19		-STIF
39			25		134	17	32	
40	35	26						
41								PEN-CUT,NO-CORE,MAT'L SAME AS BELOW
42			26			21		GRAY,SILTY SAND
43			27			19	23	
44			28			19		
45	27	31						
46								NO RECOVERY,SAND
47								
48								
49								
50	21	19						
51								NO RECOVERY,SAND
52								
53								
54								
55	33	23						
56								NO RECOVERY,SAND
57								
58								
59								
60	50/5.00	45/6.00						
61								PEN-CUTTINGS-NO-CORE,SAME AS BELOW
62			29	0- 23	125	25		VERY STIFF,GRAY,BROWN CLAY
63			30	0- 20	129	23	56	-STIFF
64			31	0- 32	139	16		-HARD
65	18	20						
66								PEN-CUT,NO-CORE,MAT'L SAME AS BELOW
67			32	0- 60		11		HARD,GRAY,TAN,SANDY CLAY
68			33	0- 60		12	44	-HARD
69			34	0- 59	141	15		-HARD
70	28	30						
71			35	0- 20	137	16		-STIFF
72			36	0- 31	136	19		-HARD
73			37	0- 21	135	19	26	VERY STIFF,GRAY,TAN,CLAYEY SAND
74			38		135	19		
75	44	56						

DRILLER- MIKE BAHM LOGGER- R. CURSON,JOHN HOES
TITLE-

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER SK4

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	142	137	5.0	2.5	OTHE								
2	137	132	5.0	7.5	CL	10.0	60	1717	139	0.0			
3	132	127	5.0	12.5	CH	6.0	50	1768	132	0.0			
4	127	122	5.0	17.5	OTHE	12.0	80						
5	122	117	5.0	22.5	SC	28.0	70	720	139	0.0	0.4	0.3	3.7
6	117	112	5.0	27.5	OTHE	31.0	80						
7	112	107	5.0	32.5	CL	46.0	60	3720	140	0.0	3.1	2.0	3.9
8	107	102	5.0	37.5	SC	34.0	70	2448	135	0.0	7.4	4.9	4.1
9	102	97	5.0	42.5	OTHE	61.0	80						
10	97	82	15.0	52.5	OTHE	51.3	80						
11	82	77	5.0	62.5	CH	103.6	50	1795	131	0.0	3.6	2.4	3.6
12	77	70	7.0	68.5	CL	48.0	60	3611	139	0.0	14.9	7.5	3.8
13	70	66	4.0	74.0	SC	100.0	70	1890	135	0.0			

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 142.3 FEET

HOLE NUMBER SK4

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	142.3	137.3	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
2	137.3	132.3	0.43	2.15	2.15	0.5	1.07	1.07	3.62	0.00	2.6
3	132.3	127.3	0.44	2.21	4.36	0.5	1.10	2.18	2.57	0.00	3.7
4	127.3	122.3	0.15	0.75	5.11	0.5	0.37	2.55	4.29	0.00	0.0
5	122.3	117.3	0.18	0.90	6.01	0.5	0.45	3.00	4.29	0.00	0.5
6	117.3	112.3	0.39	1.94	7.94	0.5	0.97	3.97	4.29	0.00	0.0
7	112.3	107.3	0.93	4.65	12.59	0.5	2.33	6.30	3.62	0.00	1.2
8	107.3	102.3	0.61	3.06	15.65	0.5	1.53	7.83	4.29	0.00	1.3
9	102.3	97.3	0.76	3.81	19.47	0.5	1.91	9.73	4.29	0.00	0.0
10	97.3	82.3	0.64	9.62	29.09	0.5	4.81	14.55	4.29	0.00	0.0
11	82.3	77.3	0.45	2.24	31.34	1.0	2.24	16.79	2.57	0.00	0.5
12	77.3	70.3	0.90	6.32	37.66	0.5	3.16	19.95	3.62	0.00	1.1
13	70.3	66.3	0.47	1.89	39.55	0.5	0.95	20.90	4.29	0.00	0.3

DRILLING LOG

COUNTY HARRIS
HIGHWAY NO US 290
CONTROL 0050-06-033
IPE

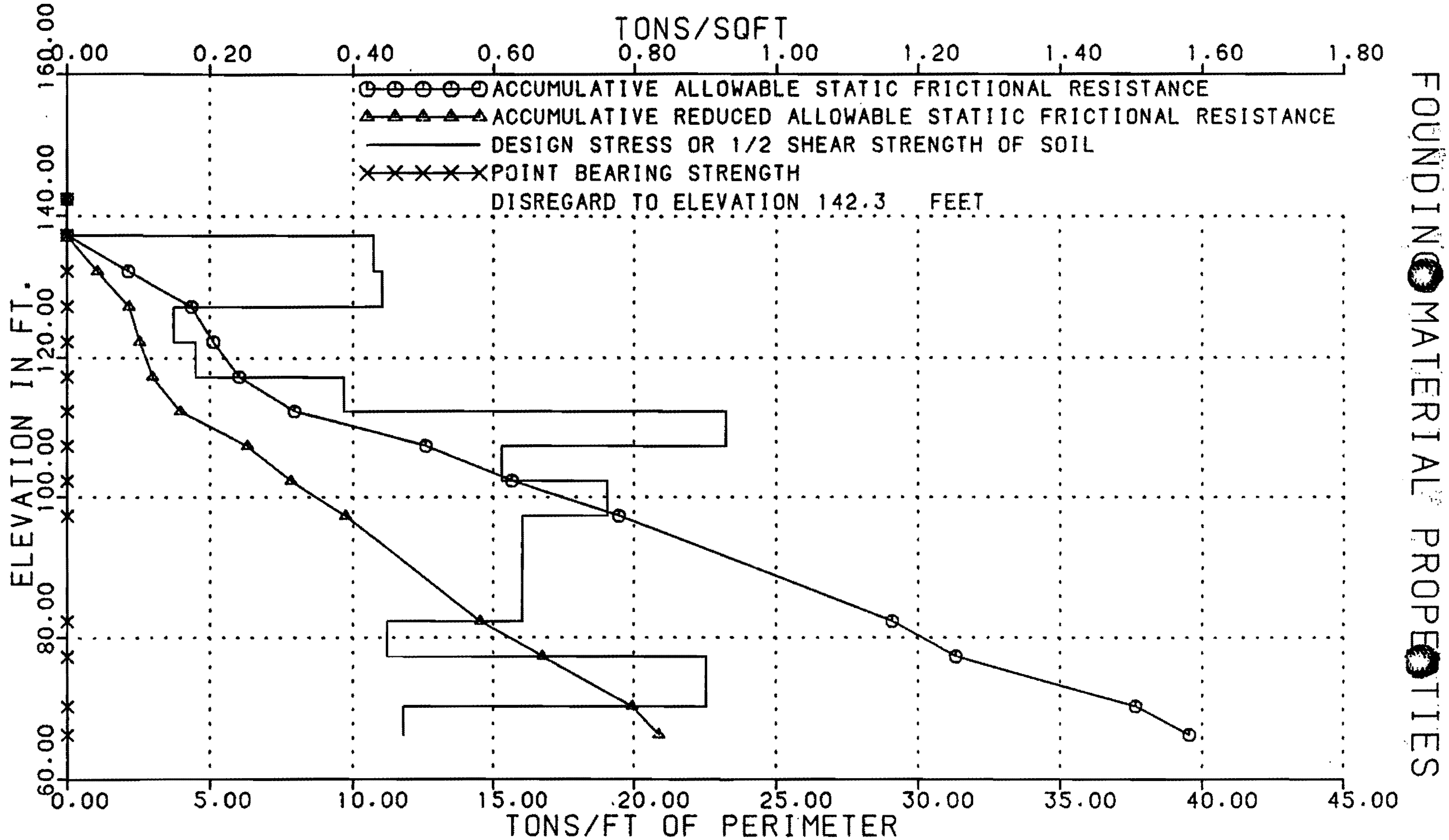
STRUCTURE BRIDGE • SKINNER ROAD
HOLE NO SK4
STATION 1972+76
LOCATION 21.7 FT RT

THD DIST 12
DATE 05/01/86
GRD. ELEV. 142.3
GRD.WATER ELEV. 142.3

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
						0	
142.3	0				NO RECOVERY		
			6 (6.0")	4 (6.0")			
137.3					VERY STIFF.GRAY.TAN.SANDY CLAY		
			3 (6.0")	3 (6.0")		10	
132.3	10				VERY STIFF.BROWN.GRAY CLAY W/CALCS.		
			6 (6.0")	5 (6.0")			
127.3					GRAY.TAN.SILTY SAND		
			16 (6.0")	12 (6.0")		20	
122.3	20				FIRM.GRAY.TAN.SILTY.CLAYEY SAND		
			12 (6.0")	19 (6.0")			
117.3					LIGHT GRAY SAND		
			31 (6.0")	15 (6.0")		30	
112.3	30				HARD.GRAY.TAN.SANDY CLAY		
			16 (6.0")	18 (6.0")			
107.3					HARD.LIGHT BROWN.SILTY.CLAYEY SAND		
			35 (6.0")	26 (6.0")		40	
102.3	40				GRAY.SILTY SAND		
			27 (6.0")	31 (6.0")			
97.3					NO RECOVERY.SAND		
	50		21 (6.0")	19 (6.0")		50	
			33 (6.0")	23 (6.0")			
			50 (5.0")	45 (6.0")		60	
82.3	60				VERY STIFF.GRAY.BROWN CLAY		
			18 (6.0")	20 (6.0")			
77.3					HARD.GRAY.TAN.SANDY CLAY		
	70		28 (6.0")	30 (6.0")		70	
70.3					STIFF.LIGHT GRAY.TAN.CLAYEY SAND		
			44 (6.0")	56 (6.0")			

•REMARKS: GWE OBSD. • END OF DRLG.

DRILLER MIKE BAHM LOGGER R. CURSON.JOHN HOES TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK4	DATE	05/01/86
CONTROL	0050-06-033	STATION	1972+76	BY	SJH
IPE		LOCATION	21.7 FT RT		

E125

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY-	HARRIS	STRUCTURE-	BRIDGE @ SKINNER ROAD
HIGHWAY NO.-	US 290	HOLE NO.-	SK5
CONTROL-	0050-06-033	STATION-	1964+30
PROJECT NO.-		OFFSET-	16.1 FT RT
DATE-	05/01/86	CODED BY-	SJH
GRD. ELEV.-	145.1	GRD. WATER ELEV.-	145.1
DISTRICT- 12			

DEPTH	FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE	NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0											NO RECOVERY TO 4 FT.
1											
2											
3											
4											
5		5	3								
6											PEN-CUT,NO CORE,MAT'L SAME AS BELOW
7											
8					1			16			GRAY,TAN CLAY W/SOME CALCS.
9					2	0- 17	131	20	56		STIFF
10		9	5								
11											PEN-CUT,NO CORE,MAT'L SAME AS BELOW
12					3		129	24			GRAY,TAN CLAY W/SM.CALCS.+NOD.
13					4	0- 27	129	18	54		-VERY STIFF
14					5	0- 31	131	20			-HARD
15		4	6								
16											PEN-CUT,NO CORE,MAT'L SAME AS BELOW
17											
18					6		142	14			GRAY,BROWN,CLAYEY SAND W/SM.CALCS.
19					7	0- 10	137	16	26		-FIRM,W/SD.LAYER
20		6	7								
21					8		135	16			-LIGHT GRAY,TAN
22					9	0- 11	139	14	26		-FIRM
23					10	0- 11	139	14			-FIRM
24					11		136	14			
25		12	21								
26											PEN-CUTTINGS-NO-CORE,SAME AS BELOW
27											
28					12		133	17			LT. GRAY,SILTY,CLAYEY SAND
29					13		130	17	27		
30		16	38								
31											DRILLED-OUT,SAND
32											
33											
34											
35		13	16								

37								
38								
39								
40	13	13						
41			14	0- 28	136	18		VERY STIFF,LIGHT GRAY,CLAYEY SAND
42			15	0- 18	137	16	31	-STIFF
43			16	0- 10	138	16		-FIRM
44			17			16	27	
45	50/5.50	50/6.00						
46								DRILLED-OUT,DENSE SAND
47								
48								
49								
50	45	32						
51								DRILLED-OUT,DENSE SAND
52								
53								
54								
55	50/3.50	50/3.00						
56								
57								DRILLED-OUT,SAND
58								
59								
60	18	14						
61								DRILLED-OUT,SAND
62								
63								
64								
65	8	12						
66								DRILLED-OUT,SAND
67								
68								
69								
70	20	24						
71								DRILLED-OUT,SAND
72								
73								
74								
75	12	10						

DRILLER- MIKE BAHM LOGGER- R. CURSON,JOHN HOES
TITLE-

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER SK5

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	145	140	5.0	2.5	OTHE								
2	140	130	10.0	10.0	CH	11.0	50	1802	130	0.0	3.9	2.6	3.6
3	130	120	10.0	20.0	SC	11.5	70	754	138	0.0	0.4	0.3	3.2
4	120	115	5.0	27.5	SC	33.0	70						
5	115	105	10.0	35.0	OTHE	41.5	80						
6	105	101	4.0	42.0	SC	26.0	70	1334	137	0.0	4.6	3.1	4.1
7	101	89	12.0	50.0	OTHE	122.0	80						
8	89	75	14.0	63.0	OTHE	26.0	80						
9	75	69	6.0	73.0	OTHE	33.0	80						

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 145.1 FEET

HOLE NUMBER SK5

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM- ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM- ULATIVE	BEAR FACTOR NC	POINT BEARING STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	145.1	140.1	0.00	0.00	0.00	0.5	0.00	0.00	4.29	0.00	0.0
2	140.1	130.1	0.45	4.51	4.51	0.5	2.25	2.25	2.57	0.00	2.0
3	130.1	120.1	0.19	1.88	6.39	0.5	0.94	3.20	4.29	0.00	1.1
4	120.1	115.1	0.47	2.36	8.75	0.5	1.18	4.37	4.29	0.00	0.0
5	115.1	105.1	0.52	5.19	13.94	0.5	2.59	6.97	4.29	0.00	0.0
6	105.1	101.1	0.33	1.33	15.27	0.5	0.67	7.63	4.29	0.00	0.9
7	101.1	89.1	0.85	10.20	25.47	1.0	10.20	17.83	4.29	0.00	0.0
8	89.1	75.1	0.32	4.55	30.02	0.5	2.27	20.11	4.29	0.00	0.0
9	75.1	69.1	0.41	2.47	32.49	0.5	1.24	21.35	4.29	0.00	0.0

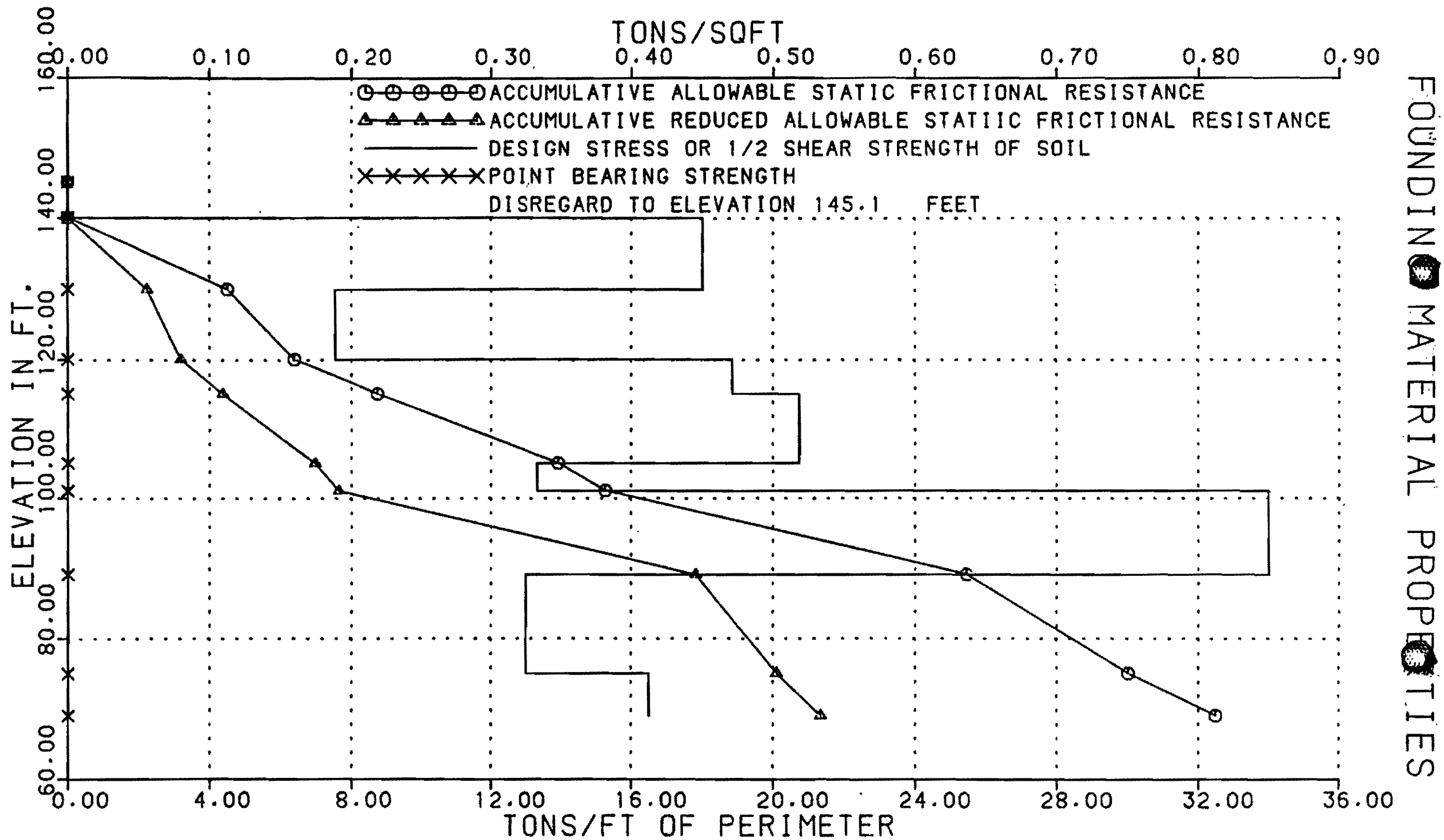
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK5	DATE	05/01/86
CONTROL	0050-06-033	STATION	1964+30	GRD. ELEV.	145.1
IPE		LOCATION	16.1 FT RT	GRD.WATER ELEV.	145.1

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	METHOD OF CORING
145.1	0				NO RECOVERY	
			5 (6.0")	3 (6.0")		
140.1					VERY STIFF,GRAY,TAN CLAY W/SM.CALCS	
	10		9 (6.0")	5 (6.0")		
			4 (6.0")	6 (6.0")		
130.1					FIRM,GRAY,BROWN,CLAYEY SAND	
	20		6 (6.0")	7 (6.0")		
			12 (6.0")	21 (6.0")		
120.1					LIGHT GRAY,SILTY,CLAYEY SAND	
115.1	30		16 (6.0")	38 (6.0")		
					DRILLED-OUT,SAND	
			13 (6.0")	16 (6.0")		
105.1	40		13 (6.0")	13 (6.0")		
					STIFF,LIGHT GRAY,CLAYEY SAND	
101.1			50 (3.5")	50 (6.0")	DRILLED-OUT,DENSE SAND	
	50		45 (6.0")	32 (6.0")		
			50 (3.5")	50 (3.0")		
89.1					DRILLED-OUT,SAND	
	60		18 (6.0")	14 (6.0")		
			8 (6.0")	12 (6.0")		
75.1	70		20 (6.0")	24 (6.0")		
					DRILLED-OUT,SAND	
			12 (6.0")	10 (6.0")		

*REMARKS: GWE OBSD. @ END OF DRLG.

DRILLER MIKE BAHM LOGGER R. CURSON,JOHN HOES TITLE



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK5	DATE	05/01/86
CONTROL	0050-06-033	STATION	1964+30	BY	SJH
IPE		LOCATION	16.1 FT RT		

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PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US 290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 05/01/86
GRD. ELEV.- 144.7
DISTRICT- 12

STRUCTURE- BRIDGE @ SKINNER ROAD
HOLE NO.- SK6
STATION- 1962+46
OFFSET- 38.0 FT RT
CODED BY- SJH
GRD. WATER ELEV.- 144.7

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0			1			20			GRAY,SILTY CLAY
1			1			20			
2			1			20			
3			2	0- 5	122	18			
4			2	0- 5	122	18			
5	8	10							
6									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
7			3			12			GRAY,TAN,SANDY CLAY W/SD.LAYER
8			4	0- 60		12	40		-HARD
9			5	0- 61		11			-HARD
10	8	8							
11									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
12			6	0- 17	126	30			STIFF,GRAY,TAN CLAY
13			7	0- 21	124	37	70		-VERY STIFF
14			8	0- 14	135	16			STIFF,GRAY,BROWN,SANDY CLAY
15	7	10							
16			9			21			
17			10	0- 25	132	19	36		-VERY STIFF
18			11	0- 30	134	19			-HARD
19			12	0- 26	139	18	32		-VERY STIFF
20	7	10							
21			13	0- 20	137	18			-STIFF
22			14	0- 23	138	18	40		-VERY STIFF
23			15	0- 23	137	18			VERY STIFF,GRAY,BROWN,CLAYEY SAND
24			16	0- 23	140	17	22		
25	12	26							
26									NO RECOVERY,SAND
27									
28									
29									
30	40	27							
31									DRILLED-OUT,SAND
32									
33									
34									
35	16	18							

37								
38								
39								
40	15	17						
41								DRILLED-OUT
42								
43								
44								
45	16	25						
46			17			18		LIGHT GRAY,FINE SAND
47			18			18	16	
48			19			16		
49			20			18	21	
50	43/6.00	57/5.00						
51								DRILLED OUT,DENSE SAND
52								
53								
54								
55	50/4.50	50/3.50						
56								DRILLED OUT,DENSE SAND
57								DRILLED-OUT,SAND
58								
59								
60	9	10						
61								DRILLED OUT,SAND
62								
63								
64								
65	12	12						
66								PEN-CUT,NO CORE,MAT'L SAME AS BELOW
67								
68			21	0- 31	140	16		HARD,GRAY,TAN,SANDY CLAY
69			22	0- 48	141	14	34	-HARD
70	21	21						
71			23		139	17	26	LIGHT GRAY,CLAYEY SAND
72			24	0- 29	137	18		-HARD
73			25	0- 20	137	17	29	-STIFF
74			26	0- 10	139	17		-FIRM
75	50/4.75	50/6.00						

DRILLER- MIKE BAHM　　　LOGGER- R. CURSON,JOHN HOES
TITLE-

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER SK6

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICK NESS FEET	CEN- TROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	145	140	5.0	2.5	CL			367	122	0.0			
2	140	135	5.0	7.5	CL	18.0	60	4349	0	0.0			
3	135	132	3.0	11.5	CH	16.0	50	1231	126	0.0			
4	132	123	9.0	17.5	CL	17.0	60	1648	133	0.0	4.3	2.2	5.1
5	123	121	2.0	23.0	SC			1685	137	0.0			
6	121	110	11.0	29.5	OTHE	52.5	80	1634	140	0.0			
7	110	100	10.0	40.0	OTHE	33.0	80						
8	100	96	4.0	47.0	OTHE	41.0	80						
9	96	89	7.0	52.5	OTHE	129.5	80						
10	89	80	9.0	60.5	OTHE	19.0	80						
11	80	75	5.0	67.5	CL	24.0	60	2858	140	0.0			
12	75	71	4.0	72.0	SC	42.0	70	1764	137	0.0			
13	71	69	2.0	75.0	OTHE	111.6	80	706	139	0.0			

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 144.7 FEET

HOLE NUMBER SK6

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	144.7	139.7	0.09	0.46	0.46	0.5	0.23	0.23	3.62	0.00	0.0
2	139.7	134.7	1.09	5.44	5.90	0.5	2.72	2.95	3.62	0.00	3.6
3	134.7	131.7	0.31	0.92	6.82	0.5	0.46	3.41	2.57	0.00	1.0
4	131.7	122.7	0.41	3.71	10.53	0.5	1.85	5.26	3.62	0.00	1.5
5	122.7	120.7	0.42	0.84	11.37	0.5	0.42	5.68	4.29	0.00	0.0
6	120.7	109.7	0.41	4.49	15.86	0.5	2.25	7.93	4.29	0.00	0.6
7	109.7	99.7	0.41	4.12	19.99	0.5	2.06	9.99	4.29	0.00	0.0
8	99.7	95.7	0.51	2.05	22.04	0.5	1.02	11.02	4.29	0.00	0.0
9	95.7	88.7	0.85	5.95	27.99	1.0	5.95	16.97	4.29	0.00	0.0
10	88.7	79.7	0.24	2.14	30.13	0.5	1.07	18.04	4.29	0.00	0.0
11	79.7	74.7	0.71	3.57	33.70	0.5	1.79	19.82	3.62	0.00	1.8
12	74.7	70.7	0.44	1.76	35.46	0.5	0.88	20.71	4.29	0.00	0.7
13	70.7	68.7	0.18	0.35	35.82	1.0	0.35	21.06	4.29	0.00	0.2

DRILLING LOG

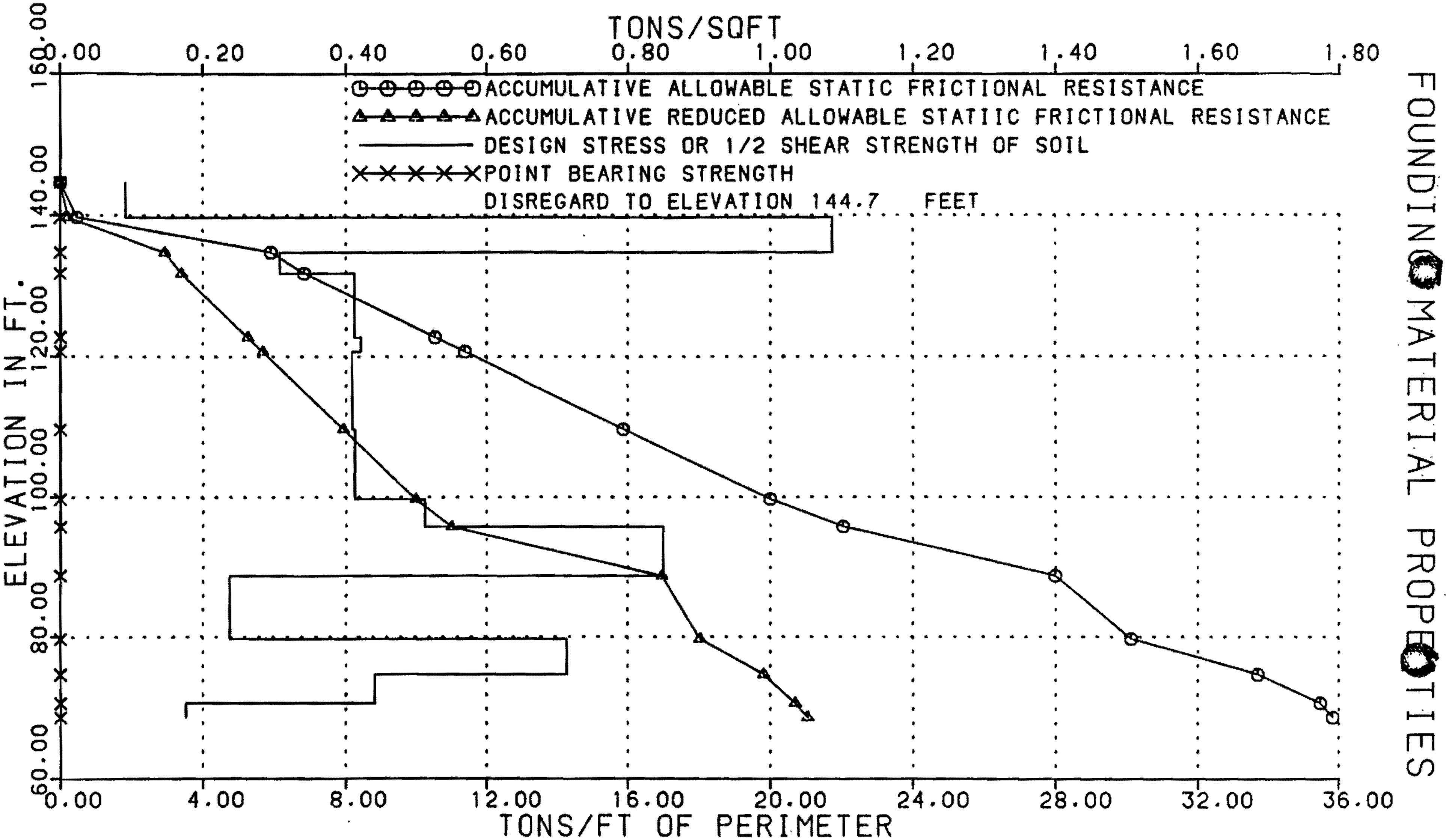
COUNTY	HARRIS	STRUCTURE	BRIDGE • SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK6	DATE	05/01/86
CONTROL	0050-06-033	STATION	1962+46	GRD. ELEV.	144.7
IPE		LOCATION	38.0 FT RT	GRD.WATER ELEV.	144.7

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL		METHOD OF CORING
144.7	0					0	
			8 (6.0")	10 (6.0")	GRAY,SILTY CLAY		
139.7			8 (6.0")	8 (6.0")	HARD,GRAY,TAN,SANDY CLAY	10	
134.7	10				STIFF,GRAY,TAN CLAY		
131.7			7 (6.0")	10 (6.0")	VERY STIFF,GRAY,BROWN,SANDY CLAY		
	20		7 (6.0")	10 (6.0")		20	
122.7					VERY STIFF,GRAY,TAN,CLAYEY SAND		
120.7			12 (6.0")	26 (6.0")	NO-RECOVERY,SAND		
	30		40 (6.0")	27 (6.0")		30	
			16 (6.0")	18 (6.0")			
109.7					DRILLED-OUT,SAND		
	40		15 (6.0")	17 (6.0")		40	
			16 (6.0")	25 (6.0")			
99.7					LIGHT GRAY,FINE SAND W/V. SL.CLAY		
95.7	50		43 (6.0")	37 (5.0")	DRILLED-OUT,DENSE SAND	50	
			50 (4.5")	50 (3.5")			
86.7					DRILLED-OUT,SAND		
	60		9 (6.0")	10 (6.0")		60	
			12 (6.0")	12 (6.0")			
79.7					HARD,GRAY,TAN,SANDY CLAY		
74.7	70		21 (6.0")	21 (6.0")		70	
					STIFF,LIGHT GRAY,CLAYEY SAND		
70.7			50 (4.7")	50 (5.0")	DENSE SAND		

•REMARKS:

DRILLER MIKE BAHM LOGGER R. CURSON,JOHN HOES TITLE

E137



COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK6	DATE	05/01/86
CONTROL	0050-06-033	STATION	1962+46	BY	SJH
IPE		LOCATION	38.0 FT RT		

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

DRILLING REPORT

COUNTY- HARRIS
HIGHWAY NO.- US 290
CONTROL- 0050-06-033
PROJECT NO.-
DATE- 05/01/86
GRD. ELEV.- 145.2
DISTRICT- 12

STRUCTURE- BRIDGE @ SKINNER ROAD
HOLE NO.- SK7
STATION- 1960+55
OFFSET- 31.0 FT RT
CODED BY- SJH
GRD. WATER ELEV.- 145.2

DEPTH FT.	PEN 1ST 6IN	PEN 2ND 6IN	SAMPLE NO.	TAT LAT-ULT PSI	WET DEN PCF	MOI %	LL %	PI %	DESCRIPTION OF MATERIAL AND REMARKS
0			1			14			DARK GRAY,SANDY CLAY
1			1			14			
2			1			14			
3			1			14			
4			1			14			
5	6	7							
6									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
7			2		141	15			GRAY,TAN,SILTY,SANDY CLAY W/CALCS.
8			3	0- 40	140	14	34		-HARD
9			4	0- 32	138	17			-HARD
10	3	5							
11									PEN-CUTTINGS,CLAY
12									
13			5	0- 8	128	24			FIRM,GRAY,TAN CLAY W/SOME CALCS.
14			6			16	51		
15	5	8							
16			7	0- 10	132	18			FIRM,GRAY,BROWN,SANDY CLAY
17			8	0- 31	135	19	45		-HARD
18			9	0- 35	136	18			-HARD
19			10	0- 26	136	18	36		-VERY STIFF
20	12	19							
21			11		139	13			LIGHT GRAY,TAN,SANDY CLAY
22			12	0- 20	137	16			-STIFF
23			13	0- 19	139	17	37		-STIFF
24			14	0- 24	139	16			-VERY STIFF
25	17	21							
26									PEN-CUT,NO CORE,MAT'L SAME AS BELOW
27			15			20			GRAY,TAN,SILTY,CLAYEY SAND
28			16			18	27		
29			17			18			LIGHT GRAY,SILTY SAND
30	25	18							
31									DRILLED-OUT,SAND
32									
33									
34									
35	17	20							

37			19	0- 19	138	16		-STIFF
38			20	0- [illegible]6	138	17	42	-HARD
39			21	0- 41	139	16		-HARD
40	32	44						
41								PEN-CUT,NO RECOVERY,SAME AS BELOW
42								
43			22			19		GRAY SAND W/SOME SILT
44			23			16	23	
45	32	37						
46								DRILLED-OUT,NO RECOVERY
47								
48								
49								
50	48/6.00	52/5.75						
51								DRILLED-OUT,DENSE SAND
52								
53								
54								
55	43	50						
56								
57								DRILLED-OUT,SAND
58								
59								
60	10	13						
61								PEN-CUT,NO CORE,MAT'L SAME AS BELOW
62			24			18		BROWN,GRAY,SILTY CLAY W/SM.CALCS.
63			25	0- 36	124	18	43	-HARD
64			26			14		
65	16	19						
66			27	0- 38	136	14		HARD,BROWN,GRAY,SILTY CLAY
67			28	0- 38	138	14	36	-HARD
68			29	0- 52	139	14		-HARD
69			30	0- 43	137	18	44	-HARD
70	18	20						
71			31	0- 33	136	18		HARD,GRAY,SILTY,CLAYEY SAND
72			32	0- 26	136	17	29	-VERY STIFF
73			33	0- 22	137	18		-VERY STIFF
74			34		132	19	30	
75	34	25						

CRILLER- MIKE BAHM LOGGER- R. CURSON,JOHN HOES

TITLE-

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS

HOLE NUMBER SK7

STRAT NUM.	ELEVATION -FEET- FROM	TO	STRAT THICKNESS FEET	CENTROID DEPTH FEET	UNIF. SOIL CLASS	PEN TEST BLOWS/ FOOT	TEST CONST	SOIL COHES LBS/ SQFT	WET DENS LBS/ CUFT	INT. FRIC ANG.	VARIANCE TAT PEAK	AVG	RATIO
1	145	140	5.0	2.5	CL								
2	140	136	4.0	7.0	CL	13.0	60	2873	140	0.0			
3	136	131	5.0	11.5	CH	8.0	50	1454	133	0.0			
4	131	125	6.0	17.0	CL	13.0	60	1838	135	0.0	7.7	3.8	4.6
5	125	120	5.0	22.5	CL	31.0	60	1534	138	0.0	1.6	1.1	3.4
6	120	117	3.0	26.5	SC	38.0	70						
7	117	110	7.0	31.5	OTHE	43.0	80						
8	110	105	5.0	37.5	CL	37.0	60	2923	138	0.0	10.6	5.3	4.8
9	105	100	5.0	42.5	OTHE	76.0	80						
10	100	89	11.0	50.5	OTHE	88.0	80						
11	89	85	4.0	58.0	OTHE								
12	85	75	10.0	65.0	CL	29.0	60	2992	135	0.0	5.4	2.4	4.6
13	75	69	6.0	73.0	SC	48.5	70	1934	136	0.0	3.1	2.0	3.9

PROB (CONTD)
1217 US 290:CYPRESS BYPASS (NOTE:NO SOIL DISREGARDED FOR STRENGTH ANALYSIS)

SOIL STRENGTH ANALYSIS RESULTS
DISREGARD TO ELEVATION IS 145.2 FEET

HOLE NUMBER SK7

STRATA NUMBER	ELEVATION -FEET- FROM	TO	DESIGN STRESS OF SOIL TONS/SQFT PER STRAT	ALLOWABLE STATIC FRICTIONAL RESISTANCE (DS) TONS PER FT OF PERIMETER PER STRATUM	ACCUM-ULATIVE	SR	REDUCED ALLOWABLE STATIC FRICTIONAL RESISTANCE PER STRATUM	ACCUM-ULATIVE	POINT BEARING BEAR FACTOR NC	STRENGTH SNC TONS/SQFT	TAT TO PEN RAT
1	145.2	140.2	0.00	0.00	0.00	0.5	0.00	0.00	3.62	0.00	0.0
2	140.2	136.2	0.72	2.87	2.87	0.5	1.44	1.44	3.62	0.00	3.3
3	136.2	131.2	0.36	1.82	4.69	0.5	0.91	2.35	2.57	0.00	2.3
4	131.2	125.2	0.46	2.76	7.45	0.5	1.38	3.72	3.62	0.00	2.1
5	125.2	120.2	0.38	1.92	9.37	0.5	0.96	4.68	3.62	0.00	0.7
6	120.2	117.2	0.54	1.63	10.99	0.5	0.81	5.50	4.29	0.00	0.0
7	117.2	110.2	0.54	3.76	14.76	0.5	1.88	7.38	4.29	0.00	0.0
8	110.2	105.2	0.73	3.65	18.41	0.5	1.83	9.21	3.62	0.00	1.2
9	105.2	100.2	0.95	4.75	23.16	0.5	2.37	11.58	4.29	0.00	0.0
10	100.2	89.2	1.10	12.11	35.27	0.5	6.05	17.63	4.29	0.00	0.0
11	89.2	85.2	0.00	0.00	35.27	0.5	0.00	17.63	4.29	0.00	0.0
12	85.2	75.2	0.75	7.48	42.75	0.5	3.74	21.37	3.62	0.00	1.5
13	75.2	69.2	0.48	2.90	45.65	0.5	1.45	22.83	4.29	0.00	0.7

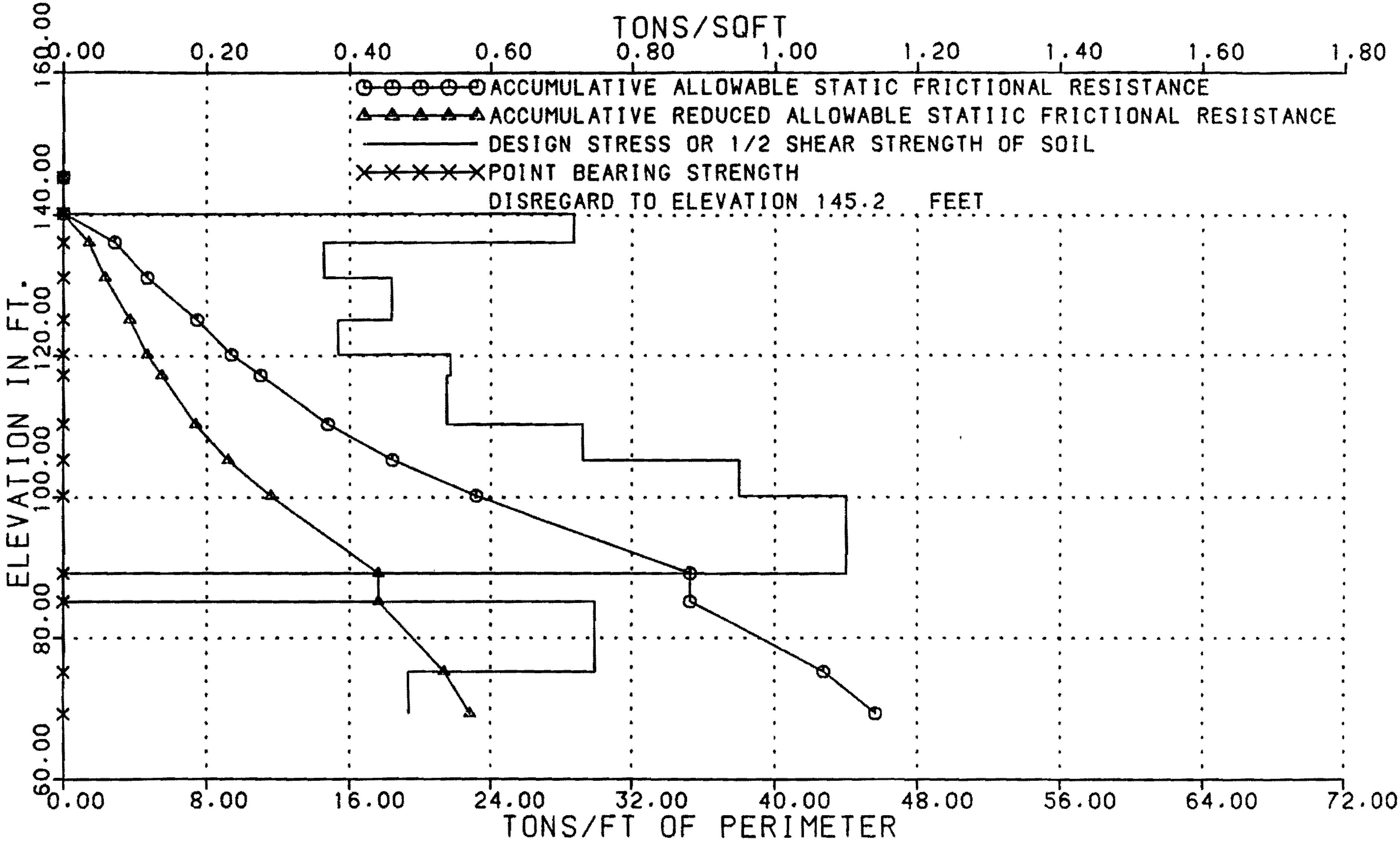
DRILLING LOG

COUNTY	HARRIS	STRUCTURE	BRIDGE * SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK7	DATE	05/01/86
CONTROL	0050-06-033	STATION	1960+55	GRD. ELEV.	145.2
IPE		LOCATION	31.0 FT RT	GRD.WATER ELEV.	145.2

ELEV FT.		LOG	THD PEN. TEST NO. OF BLOWS 1ST 6"	2ND 6"	DESCRIPTION OF MATERIAL	DEPTH	METHOD OF CORING
145.2	0				GRAY.SANDY CLAY	0	
			6 (6.0")	7 (6.0")			
140.2					HARD.GRAY.TAN.SILTY.SANDY CLAY		
136.2	10		3 (6.0")	5 (6.0")	FIRM.GRAY.TAN CLAY W/SM. CALCS.	10	
131.2			5 (6.0")	8 (5.0")	VERY STIFF.BROWN.GRAY.SANDY CLAY		
125.2	20		12 (6.0")	19 (6.0")		20	
					STIFF.LIGHT GRAY.TAN.SANDY CLAY		
120.2			17 (6.0")	21 (6.0")			
					GRAY.TAN.SILTY.CLAYEY SAND		
117.2	30		25 (6.0")	18 (6.0")	DRILLED-OUT.SAND	30	
			17 (6.0")	20 (6.0")			
110.2					HARD.GRAY.SILTY CLAY		
105.2	40		32 (6.0")	44 (6.0")		40	
					GRAY.DENSE SAND W/SM. SILT		
100.2			32 (6.0")	37 (6.0")			
					DRILLED-OUT.DENSE SAND		
	50		48 (6.0")	52 (5.7")		50	
			43 (6.0")	50 (6.0")			
89.2					NO RECOVERY.SAND		
85.2	60		10 (6.0")	13 (6.0")		60	
					HARD.BROWN.GRAY.SILTY CLAY		
			16 (6.0")	19 (6.0")			
75.2	70		18 (6.0")	20 (6.0")		70	
					VERY STIFF.GRAY.SILTY.SANDY CLAY		
			34 (6.0")	25 (6.0")			

*REMARKS:

DRILLER MIKE BAHM LOGGER R. CURSON.JOHN HOES TITLE



FOUNDIN MATERIAL PROP TIES

E143

COUNTY	HARRIS	STRUCTURE	BRIDGE @ SKINNER ROAD	THD DIST	12
HIGHWAY NO	US 290	HOLE NO	SK7	DATE	05/01/86
CONTROL	0050-06-033	STATION	1960+55	BY	SJH
IPE		LOCATION	31.0 FT RT		