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Madrid Report –
Waste Clay Report

Madrid Engineering Group, Inc.

Waste Phosphatic Clay Liner Evaluation Report

Southeast County Landfill
Hillsborough County, Florida

Prepared for SCS Engineers

The Earth is our Business℠

NOT USED

Prepared by:

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APPENDICES

1.0 Introduction

Madrid Engineering Group, Inc. (MEG) has completed a geotechnical investigation and evaluation of the waste phosphatic clay liner beneath the Southeast county landfill in Hillsborough County, Florida (Figure 1). The purpose of this report is to provide recommendations regarding the condition of the clay liner as it relates to the timeline for placing additional debris in the landfill. The site is a municipal landfill situated on formerly mined land with waste phosphatic clay deposits originally ranging from 4 to 18 feet in thickness (Ardaman, 1983). Based on the thickness, compressibility and strength of the clay, geotechnical modeling indicated the need to stage the filling sequence of the landfill. Follow-up testing and monitoring of the clay "liner" was recommended in order to refine the filling rate and sequence.

In November 2000, MEG was contracted by SCS Engineers, Inc. (SCS), the County's landfill engineers, to conduct the evaluation of the clay liner. SCS provided the scope, current site elevations and previous reports of geotechnical studies at the site as a basis for the current project.

Madrid Engineering Group Inc. proposed to obtain the majority of this information using the cone penetrometer test (CPT) method as it is a standardized test (ASTM D5778), and obtains rapid and continuous measurements of tip resistance and side friction, which are then correlated to strength and soil type information. In addition, the ability of the probe to obtain pore pressure response (U) is directly related to the permeability of the clays. The probe can also indicate the absolute pore pressure under dissipated, steady-state conditions, which indicates the height of the saturated water column over the probe if hydrostatic conditions are assumed.

In addition to the CPTU tests, hollow stem augers were advanced to the clay layer and undisturbed Shelby tube samples were obtained for laboratory testing.

2.0 Field Investigation

The field investigation began in November 2000 and was originally scoped as three separate tasks including piezocone investigation, piezoprobe (pore pressure) monitoring and undisturbed sample (Shelby tube) collection. The locations of the probes are shown on Figure 2, and are designated PC-1 through PC-6.

After discovery of very hard drilling conditions, the scope was modified to incorporate pre-drilling through the debris as needed, and to combine the piezocone program and the pore pressure monitoring by using a multi-level, combination piezocone/pore pressure monitoring instrument. This instrument will be described in further detail below.

2.1 Piezocone Investigation

The piezocone investigation began at location PC-4 on December 13, 2000. The purpose of the piezocone program was to identify the top and bottom elevations of the sand and clay liner and to gather strength information from each unit. Initially, MEG and our subcontractor, the In-Situ Group, Inc. attempted to complete the piezocone sounding at this location without augering the borehole. We found the landfill debris that had been placed over the waste phosphatic clay was very dense or hard in certain layers, and that "pre-pushing" the borehole or pre-augering would indeed be necessary. Several attempts to "pre-push" were made at PC-4. Each attempt encountered obstructions between 15 and 20 feet below ground surface (bgs).

Based on additional information and direction from SCS, the piezocone investigation was continued at locations PC-5 and then PC-6. These areas were only filled with trash that had been incinerated and were therefore considered easier to push. On December 20, 2000, piezocone soundings were successfully completed at locations PC-5 and PC-6, after the boreholes had been pre-pushed using hardened steel rods and point.

At this time during the investigation, the methodology for obtaining the top of clay elevations and strength data scope was modified to combine the piezocone and pore pressure monitoring. This change was necessitated by the very dense nature of the debris at locations PC-1, PC-2, PC-3 and PC-4. Prior to the change, it would have been necessary to auger two boreholes at each location and a third necessary for undisturbed sample collection at select locations.

A multi-level combination piezocone/pore pressure instrument was manufactured by In Situ Group specifically for this testing program. It consists of a typical piezocone measuring tip, sleeve and pore pressure sensor at the bottom of the instrument, and additional pore pressure sensor located 4.5 feet above the lower sensor on the side of the probe rod. This setup allowed MEG to complete the piezocone program in conjunction with the piezoprobe (pore pressure) monitoring, and to maximize productivity by obtaining simultaneous readings from the two elevations at the same time.

Complete piezocone logs are included in **Appendix A** of this report. These logs indicate many parameters with respect to the depth below ground surface, including point stress and local friction, friction ratio, pore pressure information, inferred soil type and strength (friction angle for granular soils, cohesion for fine-grained soils), and overconsolidation ratio.

Elevations for the top of the sand unit and clay liner are summarized below in **Table 2.1**. Also included is the thickness of the clay liner at each location.

Table 2.1
Piezocone Testing Summary

Location	Ground Elevation[1] (feet)	Top of Sand Elevation (feet)	Top of Clay Elevation (feet)	Bottom of Clay Elevation (feet)	Clay Thickness (feet)
PC-1	171.47	121.5	116.5	105.5	11.0
PC-2	184.82	119.3	114.8	110.3	4.5
PC-3	156.46	124.5	119.5	110.5	9.0
PC-4	142.29	120.8	116.3	106.3	10.0
PC-5	142.29	116.3	111.3	94.3	17.0
PC-6	146.76	115.3	112.8	104.8	8.0

[1]Elevations provide to MEG by SCS

2.2 Pore Pressure Monitoring

Pore pressure measurements were obtained at various depths in the piezoprobe, both at the upper (sleeve) and lower (tip) sensors, to obtain pore water response to the pushes. The cone advancement induces excess pore pressures in the vicinity of the probe due to the push itself, and the rate of pore pressure dissipation was obtained. Pore pressure dissipation curves at various depths are included with the CPT logs in Appendix A.

The dissipation data were obtained up to 300,000 seconds (3.5 days) elapsed time, due to the very low permeability of the waste phosphatic clay. The dissipation curves were used to graphically estimate the t_{50}, or time for 50 percent consolidation. From the t_{50}, the coefficient of consolidation c_v and the permeability k were estimated, using Figures 5.39 and 5.42 of Cone Penetration Testing in Geotechnical Practice (Lunne, Robertson, and Powell, 1997 Spon Press). These data are summarized below.

Sample ID and Depth (ft)	Cv (cm^2/min)	k (cm/sec)
PP-1 54.9	$8x10^{-2}$	$1x10^{-6}$
PP-1 56.1	$4x10^{-2}$	$9x10^{-9}$
PP-1 59.4	$7x10^{-3}$	$4x10^{-4}$
PP-1 61.5	$1x10^{-2}$	$1x10^{-8}$
PP-1 66.0	$1x10^{-1}$	$2x10^{-7}$
PP-2 67.7	$7x10^{-3}$	$6x10^{-9}$
PP-2 72.2	$3x10^{-3}$	$2x10^{-3}$
PP-2 74.4	$8x10^{-3}$	$7x10^{-9}$

PP-3	35.9	$4x10^{-1}$	$8x10^{-8}$
PP-3	40.1	$2x10^{-2}$	$1x10^{-8}$
PP-3	40.4	$1x10^{-2}$	$3x10^{-9}$
PP-3	44.6	$6x10^{-3}$	$3x10^{-9}$
PP-4	24.0	N/A – above water table	
PP-4	28.5	$8x10^{-2}$	$4x10^{-8}$
PP-4	29.1	$7x10^{-3}$	$2x10^{-9}$
PP-4	33.6	$7x10^{-2}$	$7x10^{-8}$
PP-5	30.9	N/A – at base of sand	
PP-5	35.4	$8x10^{-3}$	$3x10^{-9}$
PP-5	40.0	7	$1x10^{-3}$
PP-5	43.0	N/A - not fully dissipated	
PP-5	44.5	$2x10^{-3}$	$1x10^{-9}$
PP-5	47.5	$9x10^{-1}$	$1x10^{-4}$
PP-6	32.3	N/A – above water table	
PP-6	35.6	$2x10^{-1}$	$2x10^{-6}$
PP-6	36.8	$8x10^{-3}$	$2x10^{-9}$
PP-6	38.5	N/A - not fully dissipated	
PP-6	40.1	$3x10^{-2}$	$1x10^{-8}$
PP-6	43.0	No pore press. response	

In addition to the time rate and permeability values, the final pore pressure dissipation data is an indication of the steady state pore pressure at a particular location and depth. The interpretation of this data is of critical importance to understanding the physical conditions at the transducers. The following are important methods used in interpreting this data.

- Pore pressures are plotted as a function of *excess pore pressure*, therefore the steady state pore pressure should indicate *zero excess* pore pressure as caused by the advancement of the cone. This means that if ongoing consolidation is taking place, excess pore pressure from consolidation will not be shown by these dissipation curves – they are essentially independent of each other.
- The groundwater table elevation was used be used to adjust the tip (lower pore pressure indicator) dissipation curve up or down until the excess tip pressure is zeroed out.
- The difference between the "groundwater table depth" and the "tip depth" is an indication of the total head at the tip. In other words, if a piezometer were installed at the tip depth, water in the piezometer would eventually rise or fall to the groundwater depth.
- Hydrostatic conditions were assumed to apply within the sand layer above the waste clay. The readings within the sand and/or near the top of the clay were used to determine the depth to groundwater, i.e. the height of ponded water above the clays.

- The "groundwater elevation" listed in the graph is relative to the tip (bottom) pressure indicator only. The upper sensor is located 4.5 feet above the tip, so the final dissipation curves should indicate the upper sensor has 4.5 feet less of total pressure, if the pore pressure is under hydrostatic conditions between the sensors.
- Alternatively, if the sensors do not indicate the physical difference of 4.5 feet of head, then addition pressures are at work, and the sensors are under differing total pressure heads. This may be a result of excess pore pressure due to consolidation, or the influence of boundary conditions such as bottom drainage at the base of the clay deposit. The interpretation of pore pressure data under this circumstance requires additional data, such as the March 2001 groundwater elevation contours shown in Figure 3.

Each CPTU sounding and pore pressure dissipation curve was analyzed closely to form an interpretation of the data. The individual interpretations of the Static Pore Pressure Decay curves are described below in detail.

<u>PC-1 at tip depth of 56.1 feet bgs</u>

- At PC-1, with tip depth of 56.1, the upper pore pressure (green curve) does not have less pressure than the lower (tip) probe. Therefore the upper probe is providing a false reading. The upper curve is almost non-responsive, indicating that the probe was above the water table. In fact, the lower curve, corresponding to the cone tip transducer, has been normalized to zero excess pore pressure by adjusting the ground water depth (GWD) to 53 feet. Since the top of clay is at a depth of 55 feet bgs, there is 2.0 feet of standing water at this location. Finally, a comparison of the groundwater contour map of the site (March 2001) to the elevation of the groundwater as measured herein is 119.8 (interpolated) vs. 118.5, which is very close and is a further indication that the readings are precise.
- For the PC-1 tip transducer, it is further noted that the time for dissipation was in excess of 100,000 seconds, and the test was run to 300,000 seconds. This corresponds to a permeability of 9×10^{-9} cm/s.

<u>PC-1 at tip depth 59.4 feet bgs</u>

- The next dissipation curve, PC-1 at tip depth 59.4 feet, indicates a response for both lower and upper pore pressure transducers, i.e., both are under water. The upper transducer is 4.5 feet above the tip, at a depth of 54.9 feet. Since the groundwater depth is 53, the upper transducer is at 1.9 feet of water – note that the separation between the two curves at final dissipation is approximately 2 feet, which is further confirmation that the readings are correct. The upper transducer is also right at the interface between the sand cover and the waste phosphatic clay, as indicated by the relatively fast time for dissipation (indicating a permeability on the order of 4×10^{-4} cm/s. The tip pore pressure has been normalized on the curve to have zero pore pressure at long term (i.e, 100,000 seconds or more). However, in order to normalize, the ground water depth has been adjusted to 50 feet, instead of the depth of

closer to 3'

Is this just assumed, or was the curve zeroed out

Something does not make sense with the adjustment — not measuring GWT

53 previously indicated, meaning that a piezometer installed at this depth would rise above the top of the standing water table – i.e., excess pore pressure. The total head at the tip is 59.4-50 = 9.4 feet, or approximately 3 feet above hydrostatic.

<u>PC-1 at tip depth 66 feet bgs</u>

Total GW elevation is suspect

- PC-1 at tip depth of 66 feet is at or near the bottom of the clay layer, as indicated on the Piezocone Strata log for this sounding. The tip pore pressure decay indicates that it is no longer in clay, as the t50 occurs at just 8.3 minutes, which is slower than the upper transducer by an order of magnitude. In order to normalize the tip pressure curve to zero excess pore pressure at long term, it is noted that the ground water depth is now at 65 feet, instead of the 53 indicated at the top of the clay. This indicates a head loss across the bottom of the clay. The total head in this sample is just 1 foot of head, indicating that the bottom of the clay layer has mostly dissipated, and that there is almost no excess pore pressure at the base of the clay unit. Suspect

  I measure 50' @ no scale 25' @ 50 scale
- The upper transducer at a depth of 61.5 bgs, is in the clay unit and has a higher t50, therefore is less permeable than the soils at the tip. At 300,000 seconds elapsed time, this curve is approximately 22 feet above the tip curve, instead of 4.5 feet below the tip curve. This is a strong indication of excess pore pressure at this transducer. The total head is 18.5 feet if the curve were normalized to zero on this graph. The excess pore pressure above the hydrostatic model is approximately 9 feet.
- An estimate of the degree of consolidation can be made by comparing the total anticipated stress of the landfill on the clay, at the point where the most excess pore pressure is at a depth of 61.5 feet bgs. The change in effective stress at this elevation, assuming the landfill debris has a total density of 74 pcf (SCS, personal communication) and the clay has a submerged density of 28 pcf, is 4124 psf, or 66 feet of water. The excess pressure above hydrostatic is 9 feet of water, therefore the consolidation has 9/66x100 = 14% remaining and is 86% complete. Since last loading

  is correct what about GWT @ 119'

My read says 5' difference

<u>PC-2 at tip depth 72.2 feet bgs</u>

- PC-2 with tip depth 72.2 feet in the clays, and upper transducer at 67.7 feet in the sand, indicates a ground water depth of 60 feet on the graph. It is noted that the steady state condition indicates 4.5 feet between upper and lower transducers (i.e., hydrostatic between them). The upper is in very dense sands and indicates a low permeability. The tip transducer is clearly in clays, due to the delayed pressure response from pushing to the depth, and the long time for final decay. Since the top of the clay is at depth 70 feet bgs, the water table at 60 feet indicates 10 feet of ponded water. Since a functioning underdrain/seepage collection system exists to prevent such a condition, it is unclear as to why these readings occur. It is possible that perched lenses exist at this site and have created a localized hydraulic head in the dense sand above the clay. It was noted that the tip rods were very wet when removed from this hole after completion of pore pressure readings.

<u>PC-2 at tip depth 74.5 feet bgs</u>

- A tip depth at 74.5 means that the upper transducer is at elevation 70.0, i.e., at the sand-clay interface. Since there is almost no pressure response in the upper transducer, it indicates a very fast draining layer or lense at the top of the clay. The tip transducer indicates clay with $7x10^{-9}$ cm/s permeability. The groundwater depth is still indicated at 60 feet, therefore hydrostatic conditions may exist at PC-2. There is not a significant head loss at the bottom of the clay (depth of 74.4 bgs) as in PC-1.
- As there is no excess pore pressure, this location has completely consolidated from the stress of the overlying landfill. This is not surprising, as the clays were only 4.5 feet thick at this location, based on the CPT log. The time rate of consolidation varies as the square of the thickness, so thinner layers of clay will consolidate much faster than thicker layers.
- A groundwater depth of 60 feet bgs corresponds to an elevation of 124.82, which is approximately 7 feet higher than the contour map generated for the same time period.

<u>PC-3 at tip depth 35.5 feet bgs</u>

- PC-3 at 35.5 tip depth indicates no response in the upper transducer which is at a depth of 31.0 , i.e., the water table is at or below 31.0 feet bgs.
- The lower transducer at the tip has a very small change starting negative and ending with just 2 feet of excess pore pressure on the negative side. Since the tip is at 35.5, the ground water depth should be adjusted from 29 to 29-2 = 31 feet bgs. However, since there was no pore pressure response at 31 feet bgs, the water table may in fact be at a lower depth.

<u>PC-3 at tip depth 40.4 feet bgs</u>

- The upper transducer is almost exactly at the same depth as the previous tip. The lower transducer, which is within the waste clay, ends readings at 90,000 seconds, and is not complete based on other tests we did that shows full dissipation within the clays generally takes approximately 300,000 seconds. Therefore, its reading does not come completely to the zero excess pore pressure line – it is still dissipating. With excess pore pressure, the ground water depth used to normalize the curve is too high. We therefore conclude that this curve cannot be used to accurately determine the depth to groundwater.
- The upper transducer, on the other hand, is within the sand and dissipates within 2000 seconds. Again, the final dissipation occurs 2.5 feet below the zero excess, which is normalized to 29 feet groundwater depth. Therefore the groundwater is at 29-2.5 = 31.5 feet. This is below 31.0 feet (see previous readings, therefore, the water table is 31.5 feet bgs.
- The top of clay at 37 feet indicates 5.5 feet of ponded water.

PC-3 at tip depth 44.6 feet bgs

- PC-3 at tip depth 44.6 is about 1.5 feet above the base of the clay. Both the tip and the side transducers are in clay, as indicated by the shape of the curves and the time for dissipation. There is evidence of slight excess pore pressures, since the two curves are 2.5 feet apart at 300,000 seconds, instead of 4.5 feet apart. Both curves are classic consolidation curves, with the tip generating a delayed pore pressure response.
- The tip transducer is normalized to the zero excess, which shows the groundwater depth of 29 feet, i.e., above the water table deducted from previous readings, and therefore indicating excess pore pressure of about 2.5 feet. This indicates at that the tip depth of 44.6, the consolidation is almost complete. The upper transducer is only 2.5 feet apart from the lower transducer at the end of dissipation, also indicating a difference of excess pore pressure between the two.

PC-4 at tip depth 28.5 feet bgs

- PC-4 at tip depth 28.5 and upper transducer at 24 are in clay and sand, respectively. Both curves stop at 6500 seconds, and therefore have to be extrapolated to estimate ground water depth. The tip curve is normalized to a ground water depth of 25 feet, such that it would meet zero excess pore pressure at an elapsed time of about 100,000 seconds.
- The upper probe, however, at a depth of 24 feet, does elicit a pore pressure response, which is essentially complete even at 6500 seconds, owing to the quick response of the sands. The water table must therefore exist at a depth of 24 feet. This is 2 feet above the top of clays.

PC-4 at tip depth 33.6 feet bgs

- PP-4 at tip depth 33.6 feet was normalized to a ground water depth of 30 feet, which indicates a head loss of 5 feet from the previous curve. Both tip and side transducers are in clay, however, the final pore pressures are not 4.5 feet apart, again indicating that both are under different pressure regimes. The PC-4 upper transducer is 2 feet above tip at the end of the test, instead of 4.5 feet below. Since the total pressure head for the upper is 5.6 feet, this corresponds to hydrostatic conditions at elev. 29. However, the total pressure at the bottom is 3.6 feet (tip depth – ground water depth), therefore there is about a 2 foot head loss between these two readings, indicating bottom drainage similar to that exhibited in PP-1.
- There is no excess pore pressure in any of the readings assuming the above observations, indicating that the consolidation is essentially completed.
- The groundwater elevation contour map (SCS, March 2001) indicates an elevation of 109.5; the elevation at the base of the clays is 108.7 which matches very closely. Since the elevation of the water table, based on these readings, is 2 feet above the top of the clays, or elevation 118.3, one would conclude that the clays are acting at this location as an aquaclude and the SCS reading is the groundwater elevation of the regime below the clays.

PC-5 at tip depth 35.4 feet bgs

- PC-5 at tip depth 35.4 was normalized to the zero excess pore water, which indicates groundwater at a depth of 26 feet, relative to the tip. Therefore, at the tip, the total head is 35.4-26 = 9.4 feet.
- The upper transducer dissipates to 3.0 feet less head than the tip transducer, instead of 4.5 feet, indicating 1.5 feet of excess pore pressure between the two readings. Since there cannot be any excess in the upper, due to hydrostatic conditions assumed through the sand, all excess pore pressure due to consolidation must be at the tip, i.e., the clays are actively undergoing consolidation. Subtracting 1.5 feet off of the normalized groundwater depth indicates the true groundwater at 27.5 feet bgs.
- The water table is therefore 3.5 feet ponded above the top of clay at PC-5.

PC-5 at tip depth 44.5 feet bgs

- PP-5 at tip 44.5 feet was normalized to a groundwater depth of 28 feet bgs, for a total head of 44.5-28 = 16.5 feet, indicating ongoing consolidation.
- The upper transducer responds by dissipating to 3 feet less head than the tip transducer, indicating 1.5 feet of excess pore pressure between the two. Therefore the upper transducer has 16.5+1.5 = 18 feet of total head, and both transducers indicate excess pore pressure.

PC-5 at tip depth 47.5 feet bgs

- PP-5 at tip depth 47.5 was normalized by adjusting the ground water depth to 47 feet, which indicates a very large head loss near the bottom of the clay unit. The tip is is very hard sandy or high permeability foundation materials, as noted by the rapid dissipation. The upper transducer at 10,000 seconds is 18.5 feet higher than it should be for hydrostatic conditions, indicating 19 feet total head at this depth. This further indicates a large head loss between the two readings.

PC-6 at tip depth 36.8 feet bgs

- PP-6, with tip depth of 36.8, has a nice curve for the tip transducer. It is noted that the clay is only 7 feet thick at this location, and the tip is 2.8 feet into the clay. By normalizing the curve to zero excess at long term, the ground water depth would be estimated at 28.5 feet as shown in the graph. However, it is noted that the upper transducer, at a depth of 32.3, has no pressure response, and is therefore not in the water table. The logical adjustment is to assume the water table below a depth of 32.3, say at 33 feet. Since the top of the clay is at a depth of 34, there is approximately 1 foot of ponded water at this location.

PC-6 at tip depth 40.0 feet bgs

- PP-6 at tip depth 40 feet indicates the ground water depth is 33 feet, which matches well with the above assumption. The tip at 40 feet is in the clay, near the bottom of the deposit. The upper transducer is at the sand clay interface, and based on the dissipation time is mostly influenced by the sand. This curve is therefore normalized to the zero line. However, the curve is limited to 100,000 seconds (27 hours) and dissipation is not fully complete at the tip. The final pressures are less than 4.5 feet apart, indicating excess pore pressures.

<u>PC-6 at tip depth 44 feet bgs</u>

- PP-6 at tip depth of 43 is at the base of the clay and is in hard materials, per the CPT point stress. There is no pressure response, indicating no pore pressure. Also, to normalize it, the ground water depth is set to 43 feet, which is equal to the tip depth, i.e., there is no excess pore pressure at the tip. Note that the upper transducer dissipates to 9 feet higher than it should be indicating 9 feet of excess pore pressure and ongoing consolidation.

2.3 Undisturbed Sample Collection

Undisturbed (Shelby Tube) samples of the clay liner were collected from location PC-5 on December 28, 2000 and from PC-4 on December 29, 2000. JRS Geosciences, Lakeland, provided the auger drill and crew. Because of the difficulty in augering through the landfill debris, these two locations were selected for collection of all the undisturbed samples. At location PC-5, undisturbed samples were collected at depths of 30 to 32 feet, 32 to 34 feet, 35 to 37 feet, 37 to 39 feet and 39 to 41 feet. No recovery was made at location PC-5 at a depth of 28.5 to 30.5 feet.

At location PC-4, several unsuccessful attempts were made to collect Shelby tubes from depths of 27 to 34 feet. Each time an attempt was made at these depths, the tube was retrieved empty and on the outside of the tube was a mixture of sand, clay and landfill debris. One successful sample was retrieved from a depth of 35 to 37 feet. In this tube, natural ground (brown silty sand) was present at a depth of approximately 36.5 feet. The remainder of the tube was clay with landfill debris.

After collection of the Shelby tubes, each borehole was backfilled to the top of the clay using a mixture of bentonite and cement grout. The remainder of the borehole was backfilled with cuttings and landfill debris.

3.0 Subsurface Conditions

The site consists of waste debris and ash overlying a sand blanket that was placed over very soft waste phosphatic clays. Waste debris thickness varied considerably as anticipated, and the strength of the debris was also highly variable. The underlying sand blanket was extremely dense and hard in most borings, which was not anticipated. This may be the result of debris that has intermixed, or the result of oxidation that has begun to cement the sand. The correlated friction angle of the sand is easily in the 36 to 39 degree range.

The thickness of the clays vary, based on the CPT borings, as indicated below:

CPT Boring No.	Top and Bottom Depths (ft bgs)	Clay Thickness – Jan. 2001
PP-1	From 55 to 66	11 feet
PP-2	From 70 to 74.5	4.5 feet
PP-3	From 37 to 46	9 feet
PP-4	From 26 to 36	10 feet
PP-5	From 31 to 47	16 feet
PP-6	From 35 to 42	7 feet

The clays indicated undrained shear strength of 0.5 to 1.6 kg/sq cm, or very soft to firm clays (blow counts 4 to 8) in PP-1 through 4. In PP-5, the clay is much softer, ranging from 0.27 to 0.54 kg/sq cm (i.e., very soft). PP-6 has soft to stiff clays, ranging from 0.49 to 4 kg/sq cm.

Underlying the clays is very hard material, based on tip resistance. This is possibly limestone, or unmined natural soils. Strength and other parameters of all materials can be estimated from the logs.

4.0 Laboratory Testing

Laboratory testing was completed on selected samples from the Shelby tubes collected from locations PC-4 and PC-5. The testing program included percent moisture content (ASTM D2216), Atterberg Limits (ASTM D4318), grain size (ASTM D422), consolidation (ASTM D2435) and triaxial testing (ASTM D4767). Complete laboratory data sheets are included in **Appendix B**.

4.1 Moisture Content

Moisture content data is summarized below in **Table 4.1**. In general, the data shows solids content values ranging from between approximately 38 percent and 53 percent and moisture content values ranging from between approximately 90 percent and 164 percent.

Table 4.1
Moisture Content Data Summary

Sample Number	$W_C + S_W$ (grams)	$W_C + S_D$ (grams)	W_C (grams)	Solids Content (percent)	Moisture Content (percent)
PC-4 35 ft.	54.66	31.50	9.14	49.1%	104%
PC-5 32 ft	72.10	42.30	9.06	52.7%	90%
PC-5 34 ft.	73.90	33.30	8.53	37.9%	164%
PC-5 35 ft.	78.40	41.10	9.54	45.8%	118%
PC-5 37 ft.	73.31	34.10	9.60	38.5%	160%
PC-5 39 ft.	71.46	33.40	8.78	39.3%	155%
PC-5 41 ft.	74.50	39.00	9.09	45.7%	119%

4.2 Atterberg Limits

Plasticity tests were conducted on selected clay samples, and are summarized below in **Table 4.2**. In general, the data shows the highly plastic nature of the clays with plasticity indices ranging from 102 to 144.

Table 4.2
Atterberg Limits Data Summary

Sample Number	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index
PC-5, 32 ft.	127	22	105
PC-5, 34 ft.	170	26	144
PC-5, 35 ft.	127	25	102
PC-5, 37 ft.	150	28	122
PC-5, 39 ft.	138	25	113
PC-5, 41 ft.	125	21	104

4.3 Grain Size Analysis

Grain size test results were not completed in this study. However, based on the previous studies completed at the site, the current moisture content and Atterburg Limits data, and our extensive experience with waste phosphatic clays, all of the samples analyzed classify according to the Unified Soils Classification System as highly plastic clays, CH. The clays were generally very uniform based on the CPT logs and were consistent in nature. Sand content in the clays, based on visual confirmation and previous testing, ranges from 0 to 3 percent. The sand is generally uniform fine sand tailings.

4.4 Consolidation Testing

Consolidation testing was completed under the direction of Manjriker Gunaratne, Ph.D., P.E., associate professor at the University of South Florida in Tampa, Florida. Three tests were performed on undisturbed samples from location PC-5, 30 to 32 feet, 37 to 39 feet and 39 to 41 feet. Based on the results of the consolidation tests, the average coefficient of consolidation (C_V) is calculated at 0.007 in^2/min. in the sample from 30 to 32 feet and from 37 to 39 feet. In the sample from 39 to 41 feet, C_V=0.0023 in^2/min. The calculated compression index (C_C) is 0.61 in the sample from 30 to 32 feet, 0.91 in the sample from 37 to 39 feet and 1.36 in the sample from 39 to 41 feet.

Initial void ratios were calculated from the consolidation samples, and ranged from 2.2 to 4.2. Testing further indicated that the preconsolidation pressure at PP-5 ranged from as high as 1400 psf, to as low as 740 psf. This would indicate that PP-5 is highly underconsolidated, and still has about 2/3 of the consolidation to complete.

4.5 Triaxial Testing

Consolidated undrained (quick) triaxial strength tests were completed on selected samples, at confining pressures simulating existing and future pressures. These pressures were within the range of 1800 psf to 1,000 psf. The samples were plotted on stress-strain Mohr's circle, and indicate a cohesion of 8.5 psi, and an apparent friction angle of 18.6 degrees. Results of the triaxial tests are indicated in Appendix B.

5.0 Discussion and Recommendations

The results of this investigation indicate several things of importance to the schedule and operations of the landfill, as follows:

- The consolidation of the underlying clays in Phase I, II and III are over 80 percent complete. Notwithstanding any other design or operational constraints that we may not be aware of , additional fill may be placed on these three Phases at this time.
- Boring PC-4 in Phase IV had approximately 10 feet of clay and based on piezocone readings is essentially consolidated at this time as well.
- Phase V and VI are still undergoing considerable consolidation. Consolidation tests indicate that most of the clay is at a preconsolidation pressure of 750 psf, versus over 2000 psf applied by the landfill debris, and is therefore only about 1/3 consolidated.
- Solids content tests from samples at PP-5 indicate clays ranging from 38 to 53 percent solids.

- The triaxial tests indicated a cohesion of 8.5 psi (1224 psf) and a slight frictional component as well. This cohesion indicates strength gain over time, as would be expected.
- The strength of the sand layer above the clays is extremely high. This may be the result of cementation, construction compaction, or the stress increase due to landfill placement. The strength of this layer is relatively consistent across the site and should be considered in future slope stability and wedge analyses.
- Groundwater readings indicated ponded water in the sand filter ranging from 1 to 10 feet above the clays, but generally less than 3 feet. As the sand blanket was applied uniformly across the site, and a functioning underdrain system is in place, the higher readings may be indicating other phenomena occurring, such as localized ponding, or lenses of perched or trapped water that connected to the bore hole during testing. We recommend the placement of additional permanent piezometers in the sand just above the clay layer in areas where readings were anomalous.

6.0 Basis for Recommendations

The recommendations provided are based in part on project information provided to us and only apply to the specific project and site discussed in this report. If the project information section in this report contains incorrect information or if additional information is available, Madrid Engineering Group, Inc. can be retained to review the corrected or additional information. We can modify our recommendations, if they are appropriate for the proposed project.

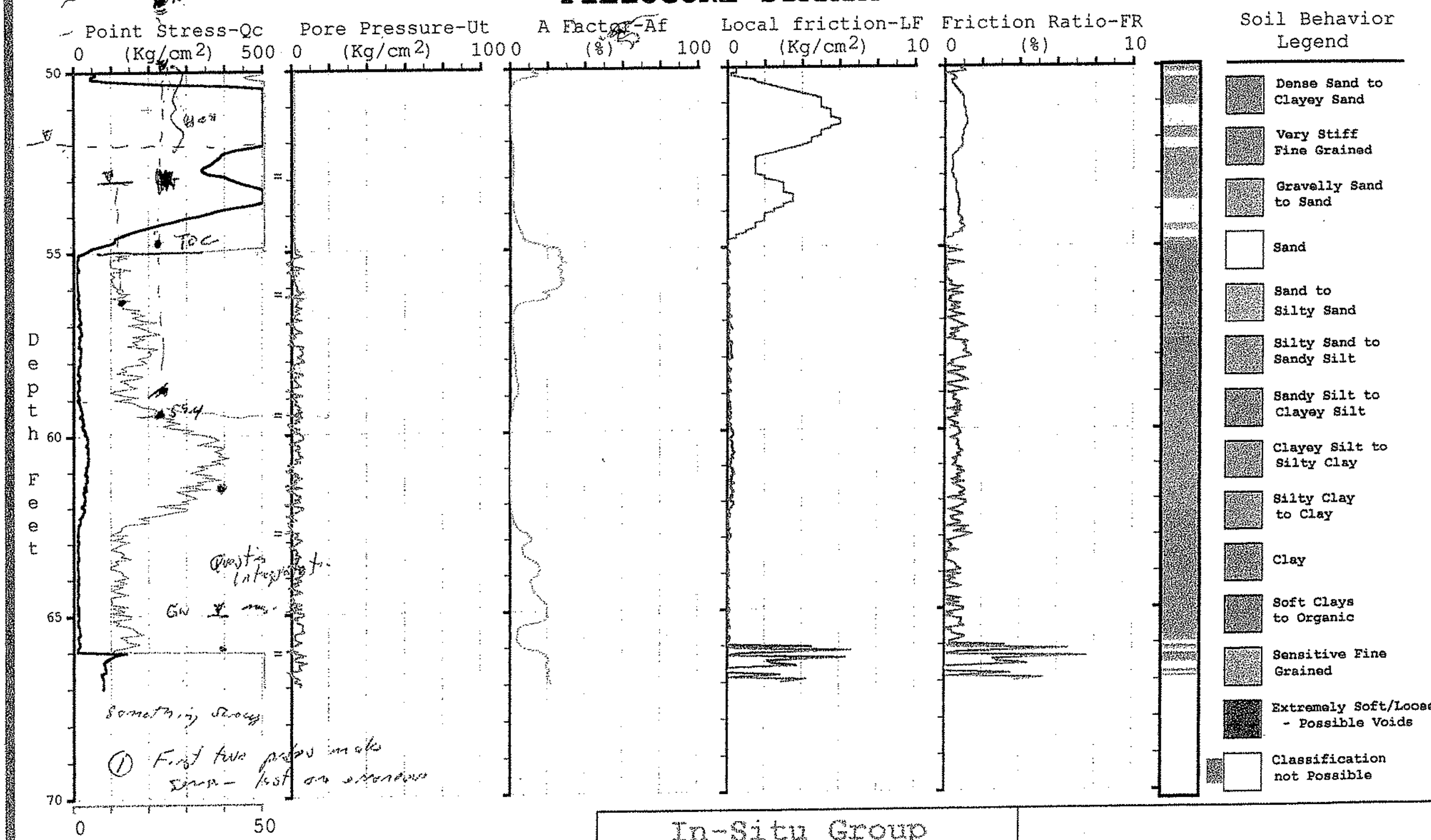
Regardless of the thoroughness of a geotechnical exploration, there is always a possibility that conditions between borings will be different from those at the specific areas explored and that conditions will not be as anticipated by the designers or contractors. The findings herein are based on the exploratory borings at the reference site and our professional judgment. The soil conditions described within this report are accurate with respect to the location and extent that the soil borings were completed. Because soils vary from place to place, and with depth, subsurface conditions different from those encountered in our exploration may exist. This investigation, was completed in accordance with generally accepted standards of engineering practice. No warranty regarding this investigation is intended, nor should any be inferred.

Appendix A

CPT Piezocone Strata Logs
Pore Pressure Dissipation Charts

REV: 98 - 3.10 - 0

PIEZOCONE STRATA



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-1'
Test Date 01-24-2001 11:22:07

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

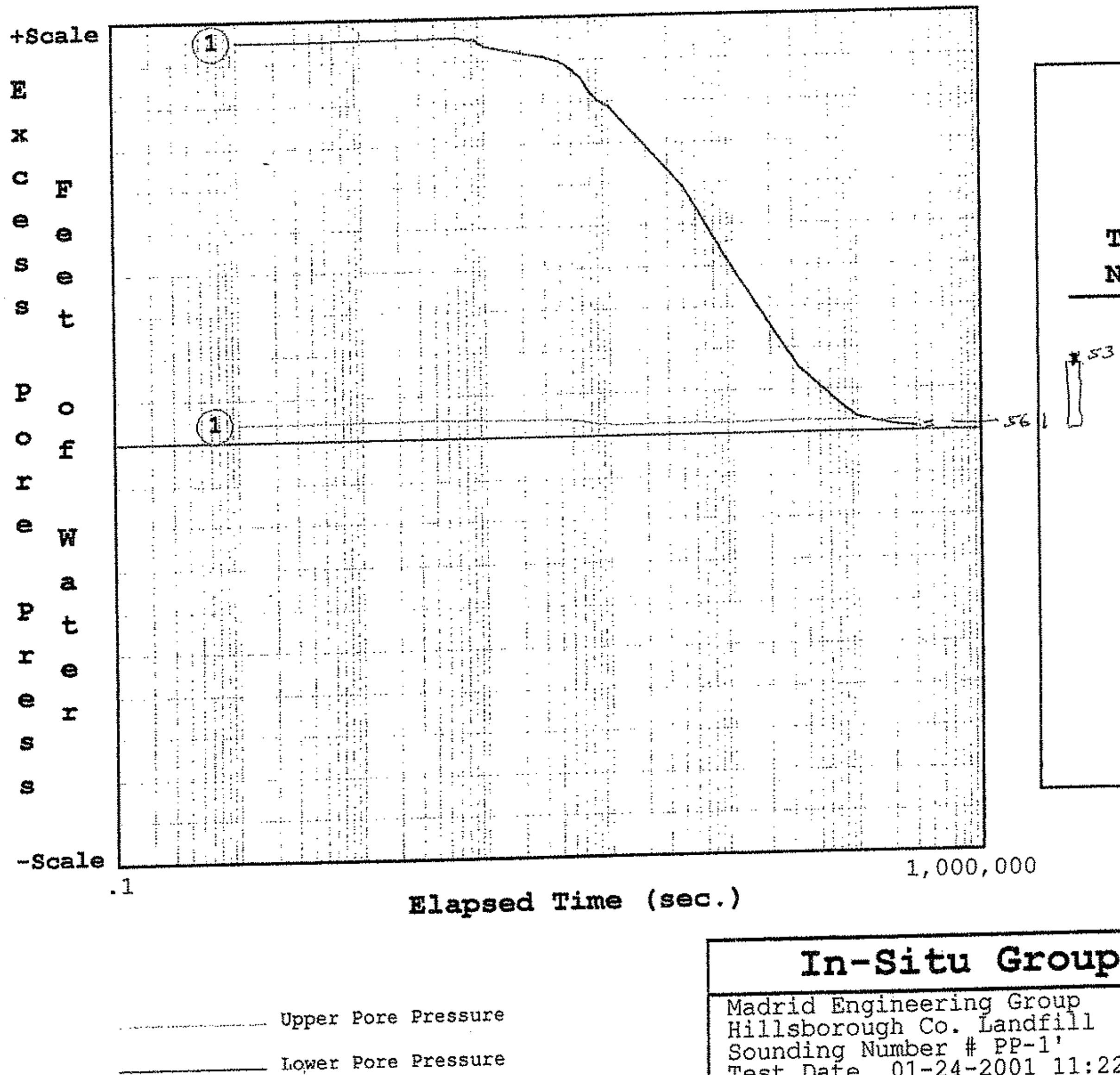
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
53	GRAVELLY SAND TO SAND	417.8	2	70	1.941	>85%	39-41	919	--	--	--	--
54	SAND	322.9	2.5	65	1.974	>85%	37-39	710	--	--	--	--
55	SAND TO SILTY SAND	48.7	.2	12	2.005	<35%	27-29	107	--	--	--	--
56	SANDY SILT TO CLAYEY SILT	11.7	0	5	2.035	<35%	<25	25	--	--	--	--
57	SILTY SAND TO SANDY SILT	17.7	.1	6	2.066	<35%	<25	38	--	--	--	--
58	SANDY SILT TO CLAYEY SILT	16.9	.1	7	2.097	<35%	<25	37	--	--	--	--
59	SANDY SILT TO CLAYEY SILT	16.8	.1	7	2.128	<35%	<25	36	--	--	--	--
60	SILTY SAND TO SANDY SILT	32.4	.2	11	2.159	<35%	25-27	71	--	--	--	--
61	SILTY SAND TO SANDY SILT	35.6	.2	12	2.189	<35%	25-27	78	--	--	--	--
62	SILTY SAND TO SANDY SILT	26.2	.2	9	2.22	<35%	<25	57	--	--	--	--
63	SANDY SILT TO CLAYEY SILT	12.4	0	5	2.251	<35%	<25	27	--	--	--	--
64	SANDY SILT TO CLAYEY SILT	12.1	0	5	2.282	<35%	<25	26	--	--	--	--
65	SANDY SILT TO CLAYEY SILT	12.7	0	5	2.312	<35%	<25	27	--	--	--	--
66	SILTY SAND TO SANDY SILT	59.9	1.3	20	2.343	<35%	27-29	131	--	--	--	--
67	SILTY SAND TO SANDY SILT	78.5	1.6	26	2.374	<35%	29-31	172	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-1'
Test Date 01-24-2001 11:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 53 Ft.

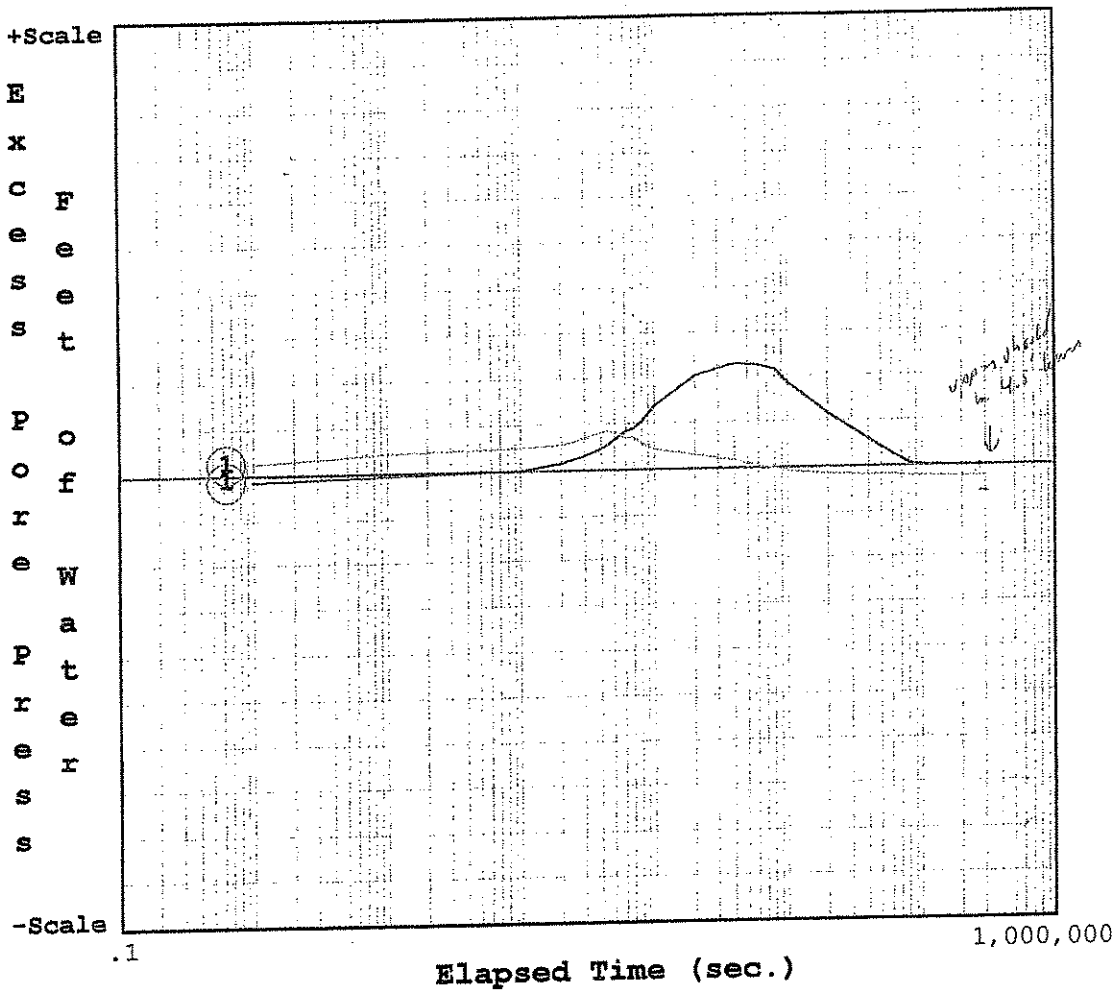
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	56.1	50

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-1'
Test Date 01-24-2001 11:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 50 Ft.

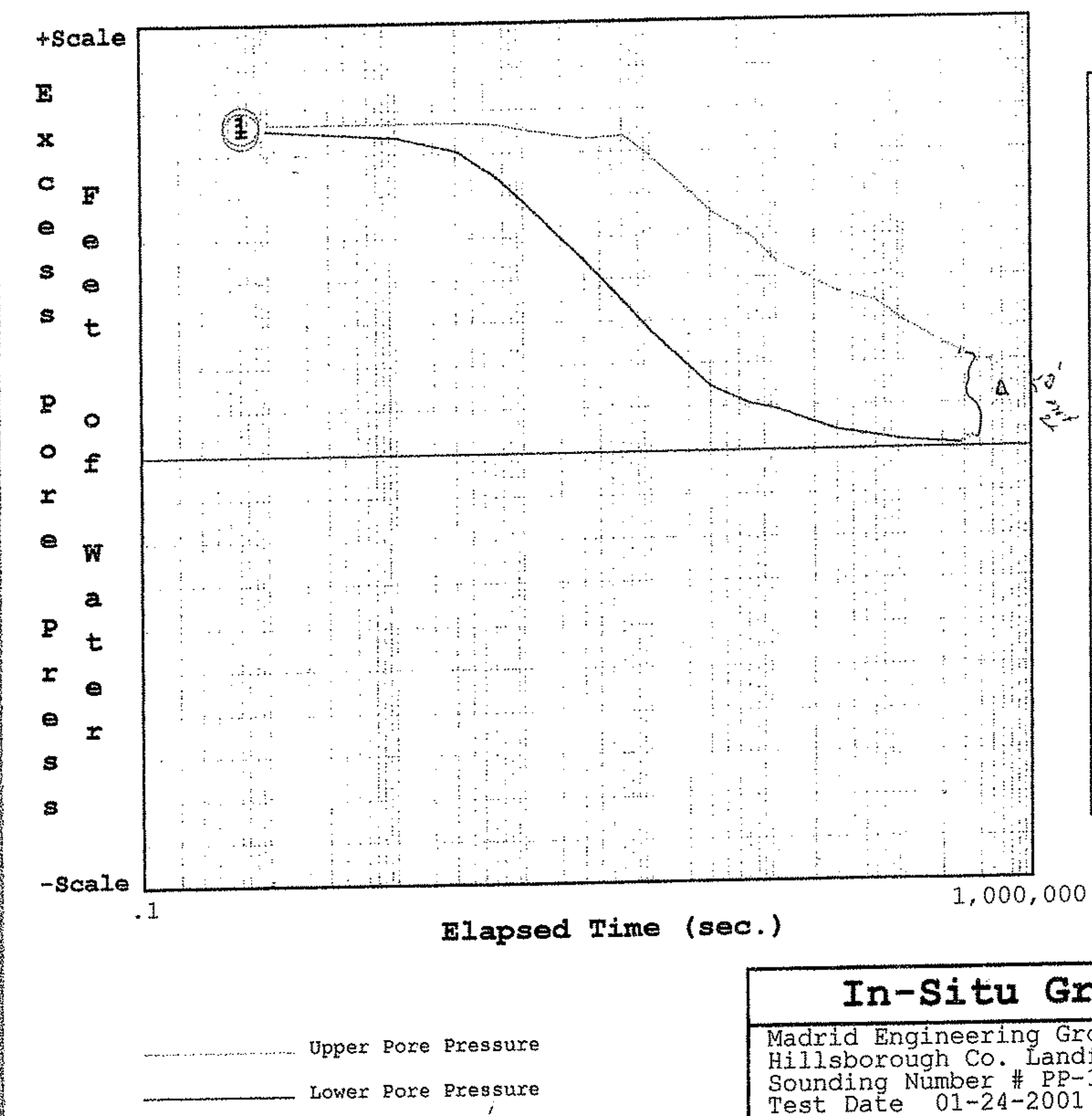
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	59.4	50

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-1'
Test Date 01-24-2001 11:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 65 Ft.		
Track Number	**Depth (Ft)**	**+/- Scale (Ft H2O)**
1	66	100

Assume and measures GWT 119.5

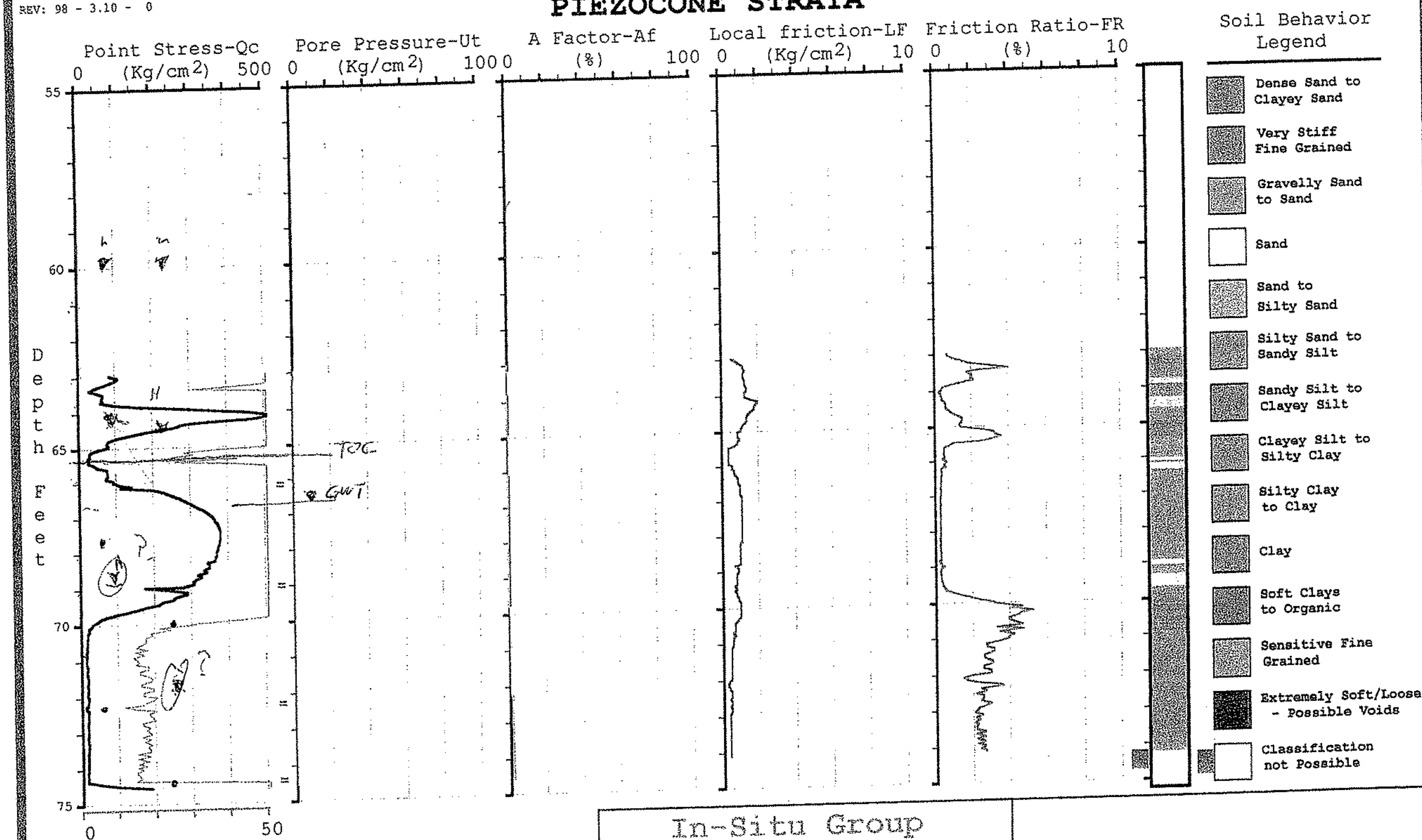
should tied to 50 scale

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-1'
Test Date 01-24-2001 11:22:07

REV: 98 - 3.10 - 0

PIEZOCONE STRATA



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-2
Test Date 01-29-2001 14:01:49

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

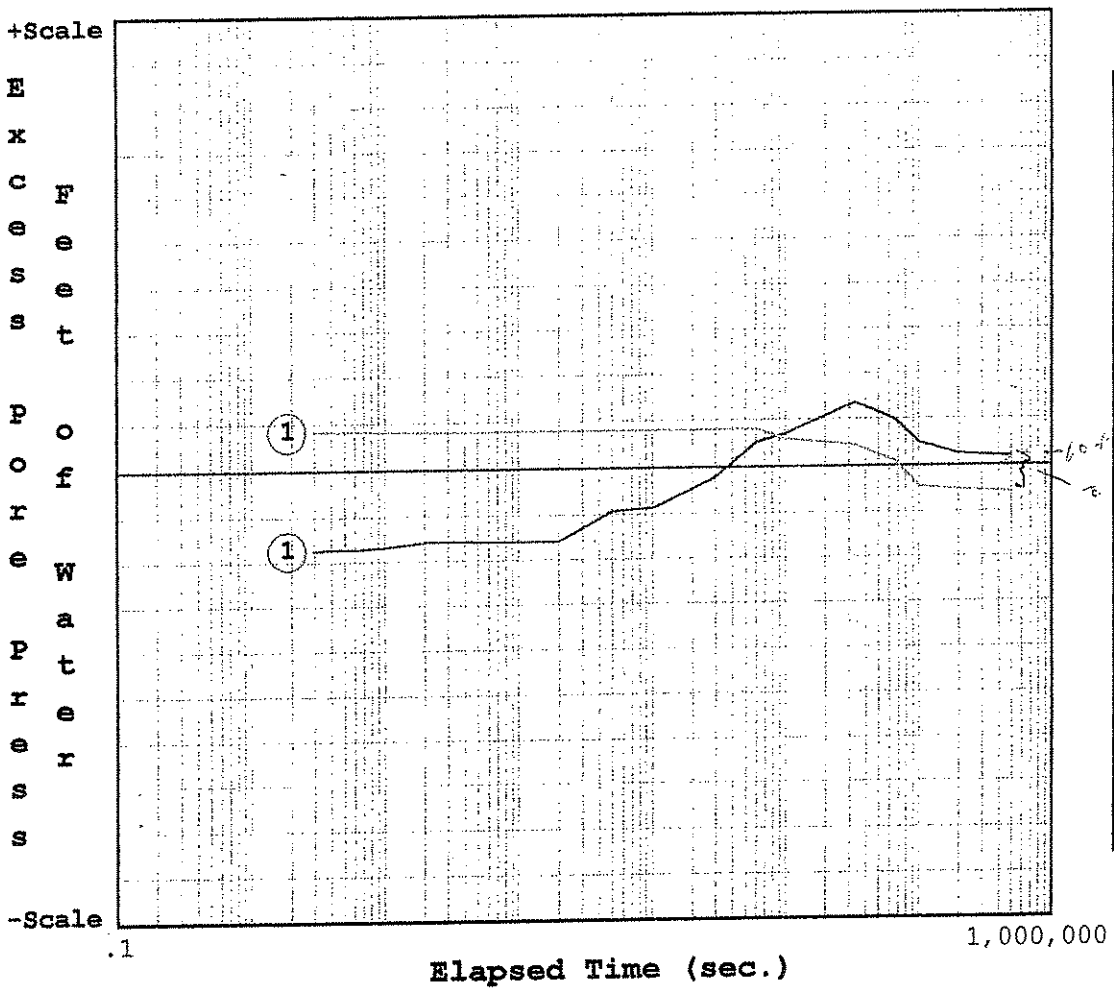
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
68	GRAVELLY SAND TO SAND	366.9	1	61	2.49	>85%	37-39	807	--	--	--	--
69	GRAVELLY SAND TO SAND	278.2	.8	46	2.527	58%-65%	35-37	612	--	--	--	--
70	SANDY SILT TO CLAYEY SILT	57.5	.8	23	2.558	<35%	27-29	126	--	--	--	--
71	SILTY CLAY to CLAY	16.4	.5	11	2.585	--	--	--	.74	2.8	.01	1-1.5
72	CLAYEY SILT TO SILTY CLAY	17.5	.4	9	2.612	--	--	--	.81	3.8	.02	1-1.5
73	CLAYEY SILT TO SILTY CLAY	18.1	.4	9	2.64	--	--	--	.84	4.2	.02	1-1.5
74	SANDY SILT TO CLAYEY SILT	24	.4	10	2.67	<35%	<25	52	--	--	--	--
75	--	188	--	--	--	--	--	--	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-2
Test Date 01-29-2001 14:01:49

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 60 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	72.2	50

Upper Pore Pressure

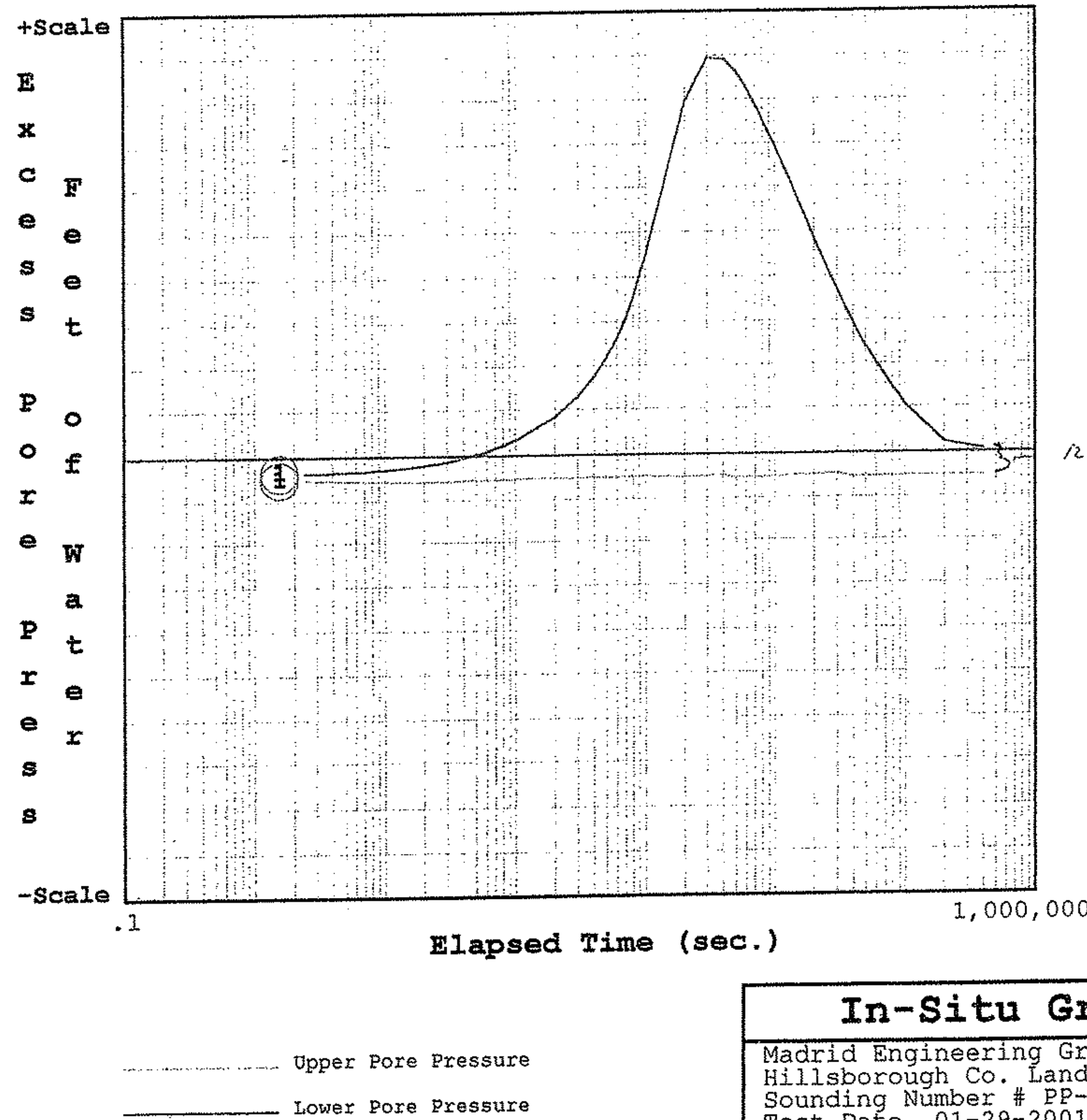
Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Co. LandfillDam
Sounding Number # PP-2
Test Date 01-29-2001 14:01:49

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 60 Ft.

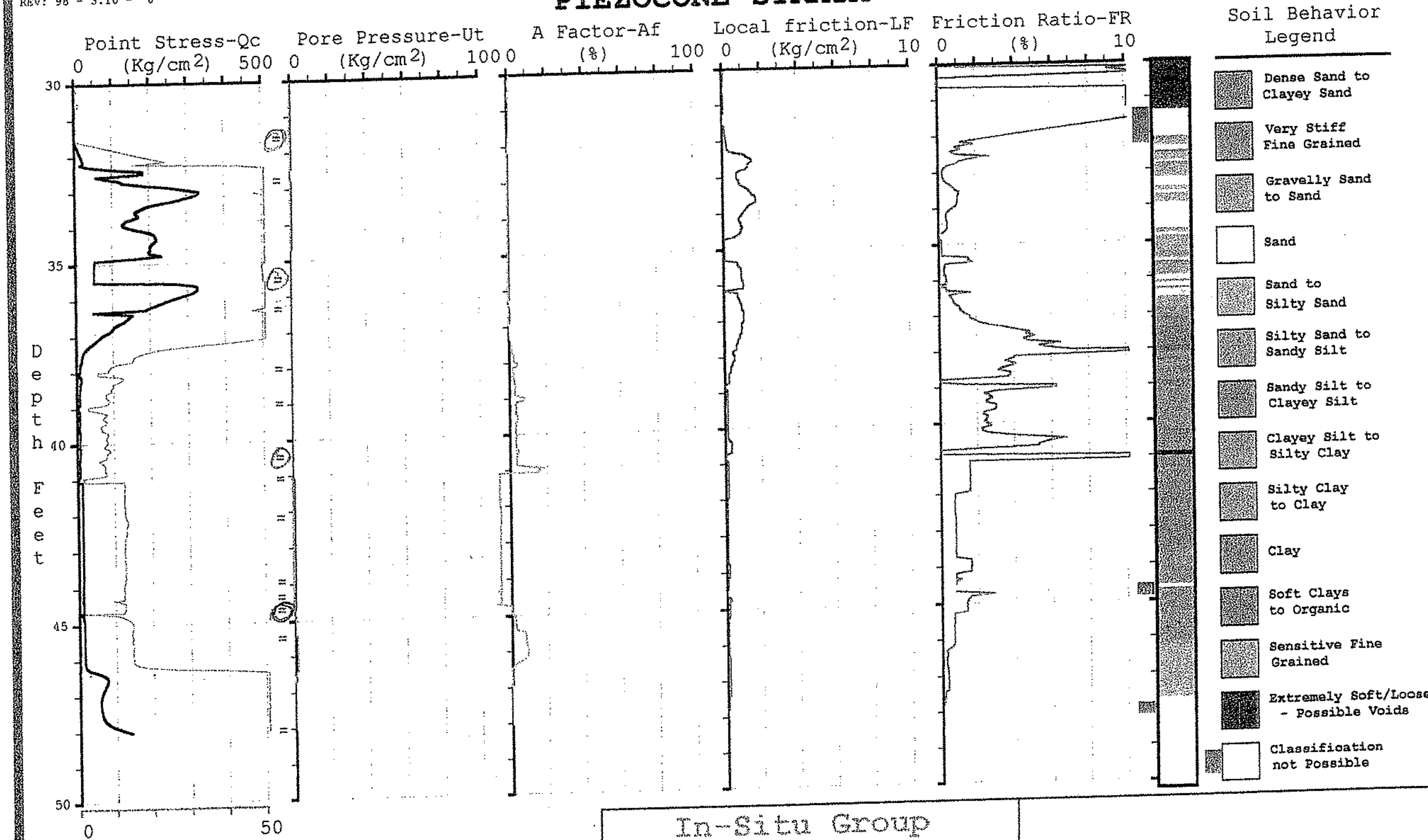
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	74.4	100

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-2
Test Date 01-29-2001 14:01:49

REV: 98 - 3.10 - 0

PIEZOCONE STRATA



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-3-
Test Date 02-11-2001 10:18:45

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

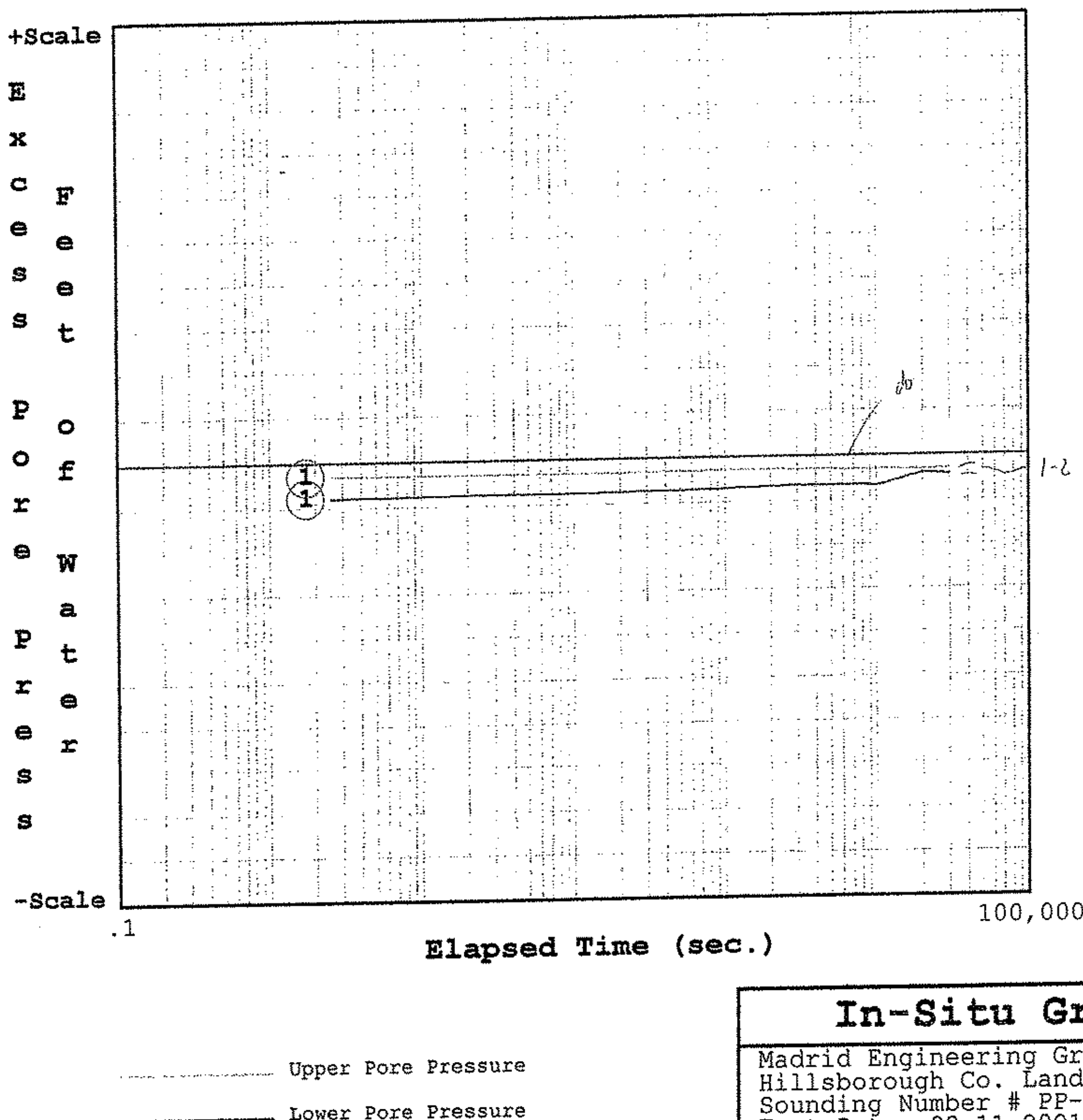
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
37	SILTY SAND TO SANDY SILT	71.7	.9	24	1.138	42%-50%	33-35	157	--	--	--	--
38	CLAYS	11	.5	11	1.163	--	--	--	.56	2	.02	1-1.5
39	SILTY CLAY to CLAY	7.2	.1	5	1.19	--	--	--	.32	3.5	.03	1-1.5
40	CLAYS	7.6	.2	8	1.215	--	--	--	.34	2.7	.03	1-1.5
41	SILTY CLAY to CLAY	8.7	.1	6	1.242	--	--	--	.4	2.8	.03	1-1.5
42	SANDY SILT TO CLAYEY SILT	12.9	.1	5	1.273	35-42%	<25	28	--	--	--	--
43	SANDY SILT TO CLAYEY SILT	12.6	.1	5	1.304	35-42%	<25	27	--	--	--	--
44	SANDY SILT TO CLAYEY SILT	12.2	.1	5	1.335	35-42%	<25	26	--	--	--	--
45	SANDY SILT TO CLAYEY SILT	12.7	.1	5	1.365	35-42%	<25	27	--	--	--	--
46	SILTY SAND TO SANDY SILT	24.3	.1	8	1.396	35-42%	25-27	53	--	--	--	--
47	SAND TO SILTY SAND	62.2	.1	16	1.427	35-42%	31-33	136	--	--	--	--
48	SAND	85.9	.1	17	1.46	42%-50%	33-35	188	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-3-
Test Date 02-11-2001 10:18:45

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Elapsed Time (sec.)

Ground Water Depth... 29 Ft.

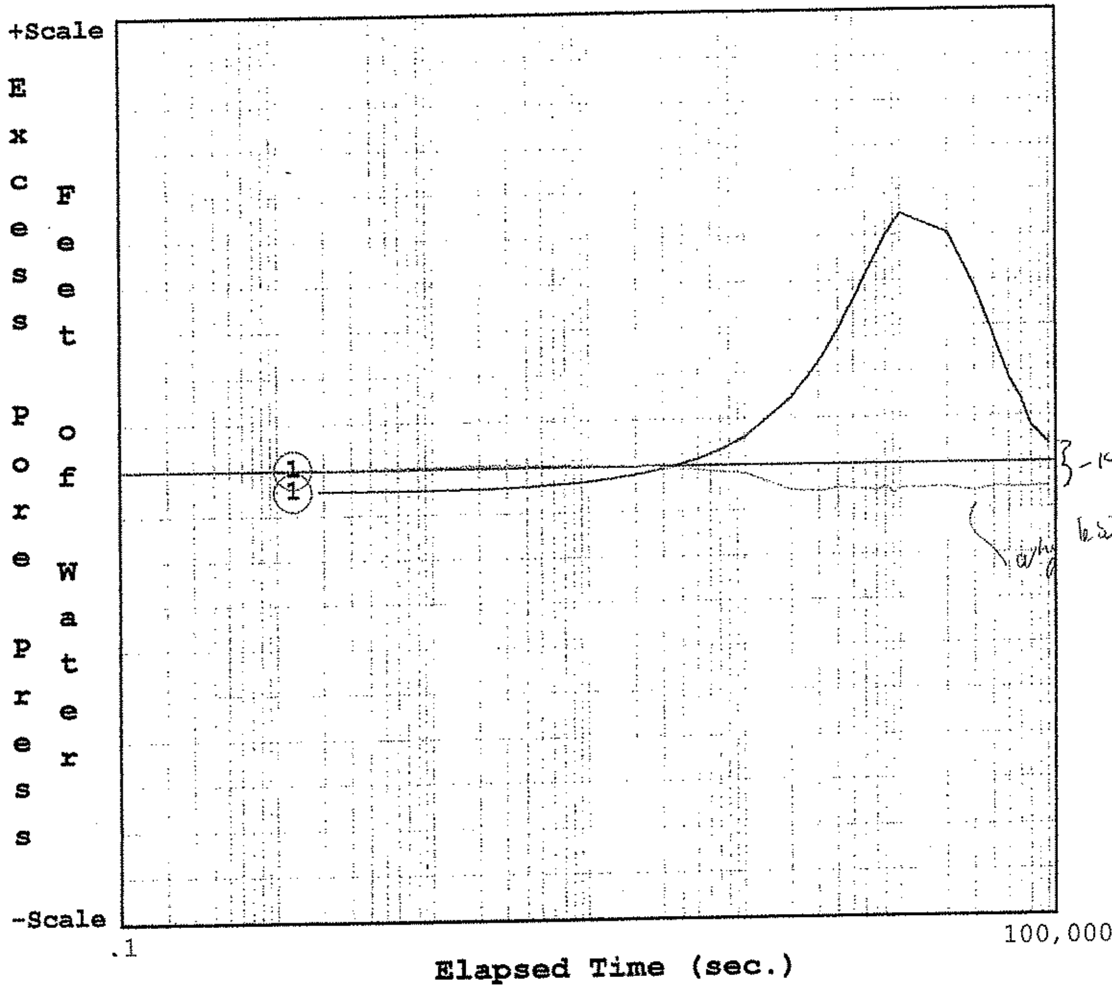
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	35.5	50

In-Situ Group

Madrid Engineering Group
Hillsborough Co. LandfillDam
Sounding Number # PP-3-
Test Date 02-11-2001 10:18:45

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 29 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	40.4	50

Upper Pore Pressure

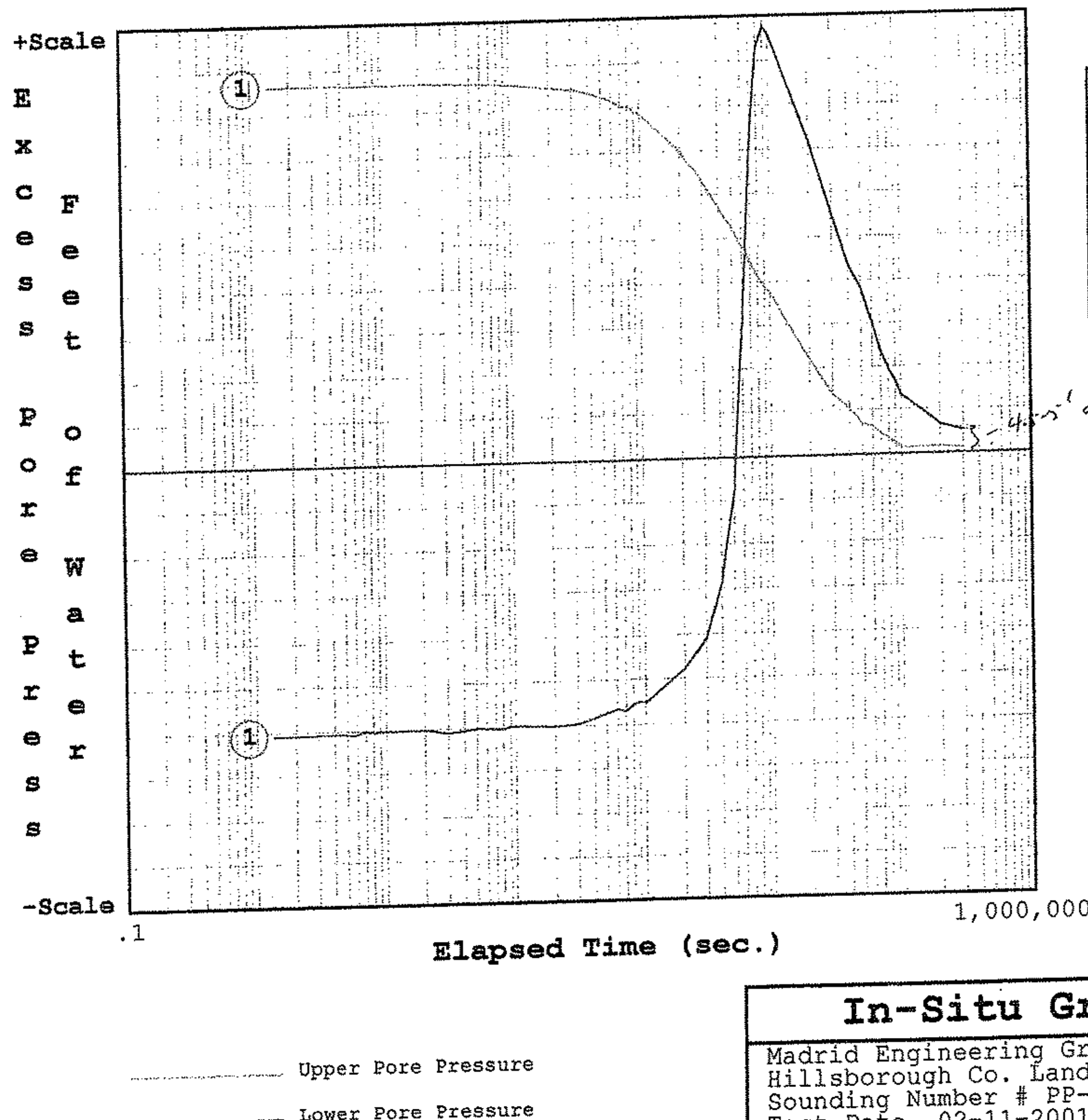
Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Co. LandfillDam
Sounding Number # PP-3-
Test Date 02-11-2001 10:18:45

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 32 Ft.

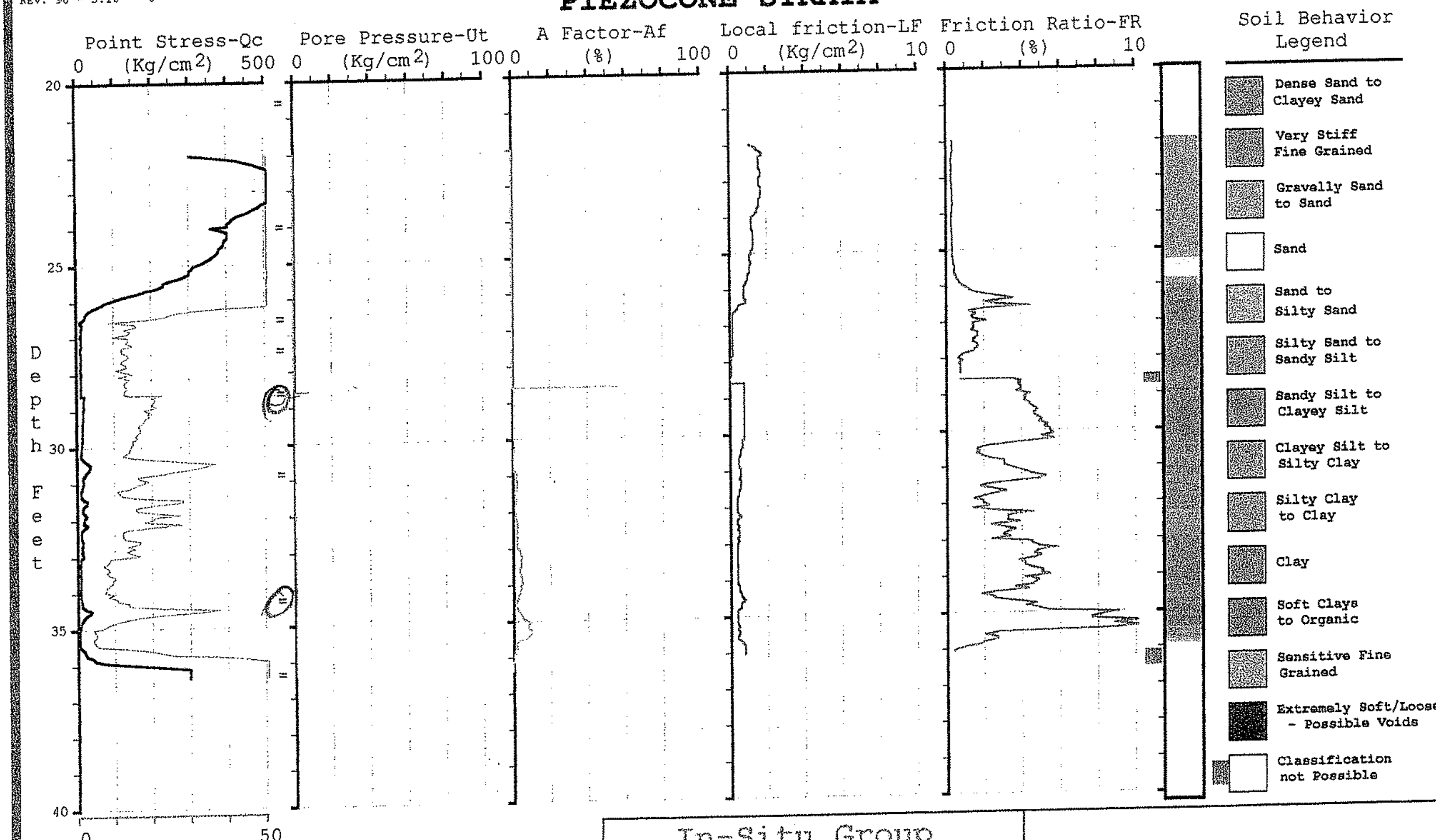
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	44.6	50

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-3-
Test Date 02-11-2001 10:18:45

REV: 98 - 3.10 - 0

PIEZOCONE STRATA



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-4-
Test Date 01-17-2001 12:15:40

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

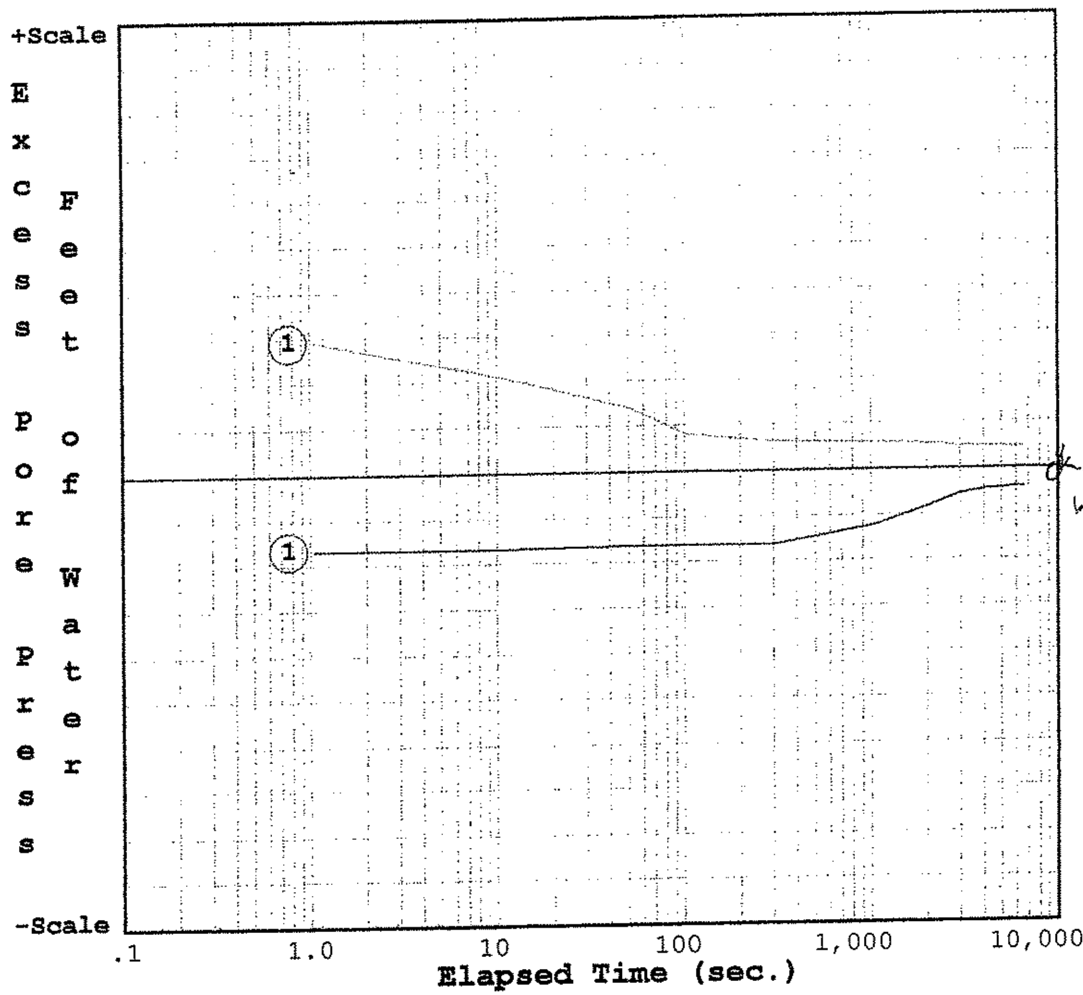
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
24	GRAVELLY SAND TO SAND	404.4	1.3	67	.879	>85%	41-43	889	--	--	--	--
25	GRAVELLY SAND TO SAND	328.6	1.1	55	.915	>85%	41-43	722	--	--	--	--
26	SAND TO SILTY SAND	101.1	.8	25	.946	50%-58%	35-37	222	--	--	--	--
27	CLAYEY SILT TO SILTY CLAY	13.1	.2	7	.973	--	--	--	.72	5.7	.03	3
28	SANDY SILT TO CLAYEY SILT	12.9	.1	5	1.004	35-42%	<25	28	--	--	--	--
29	SILTY CLAY to CLAY	18.2	.8	12	1.031	--	--	--	1.03	2.3	.01	3
30	CLAYS	17.7	.6	18	1.056	--	--	--	1	2.1	.01	3
31	CLAYEY SILT TO SILTY CLAY	19.5	.5	10	1.084	--	--	--	1.11	3	.02	6
32	CLAYEY SILT TO SILTY CLAY	18.9	.4	9	1.111	--	--	--	1.06	3.9	.02	3
33	CLAYS	11.6	.4	12	1.136	--	--	--	.61	2.5	.02	3
34	CLAYS	12.3	.4	12	1.161	--	--	--	.65	2.3	.02	3
35	CLAYS	9.9	.4	10	1.186	--	--	--	.49	1.4	.02	1-1.5
36	SAND TO SILTY SAND	150.7	.5	38	1.216	58%-65%	35-37	331	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-4-
Test Date 01-17-2001 12:15:40

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 25 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	28.5	20

Upper Pore Pressure

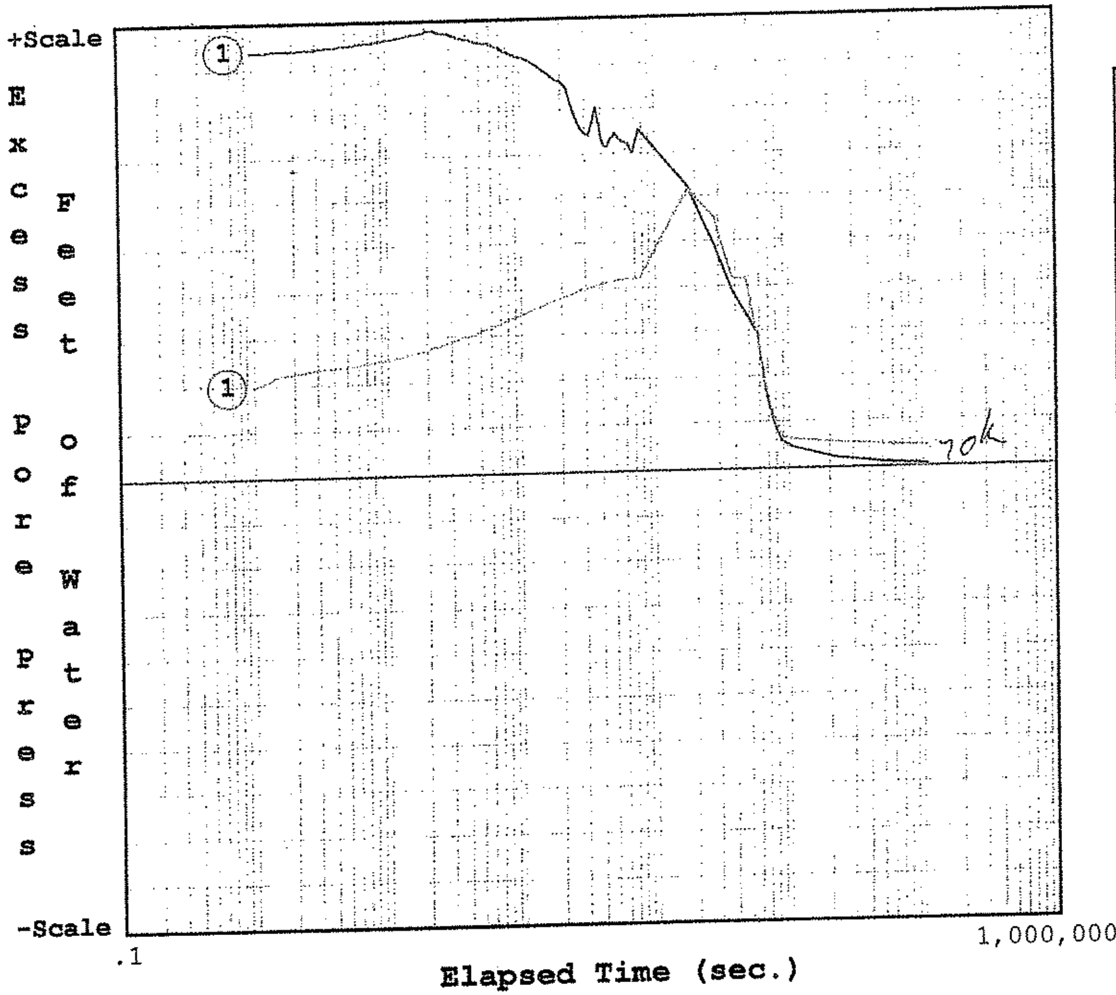
Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-4-
Test Date 01-17-2001 12:15:40

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 30 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	33.6	50

Upper Pore Pressure

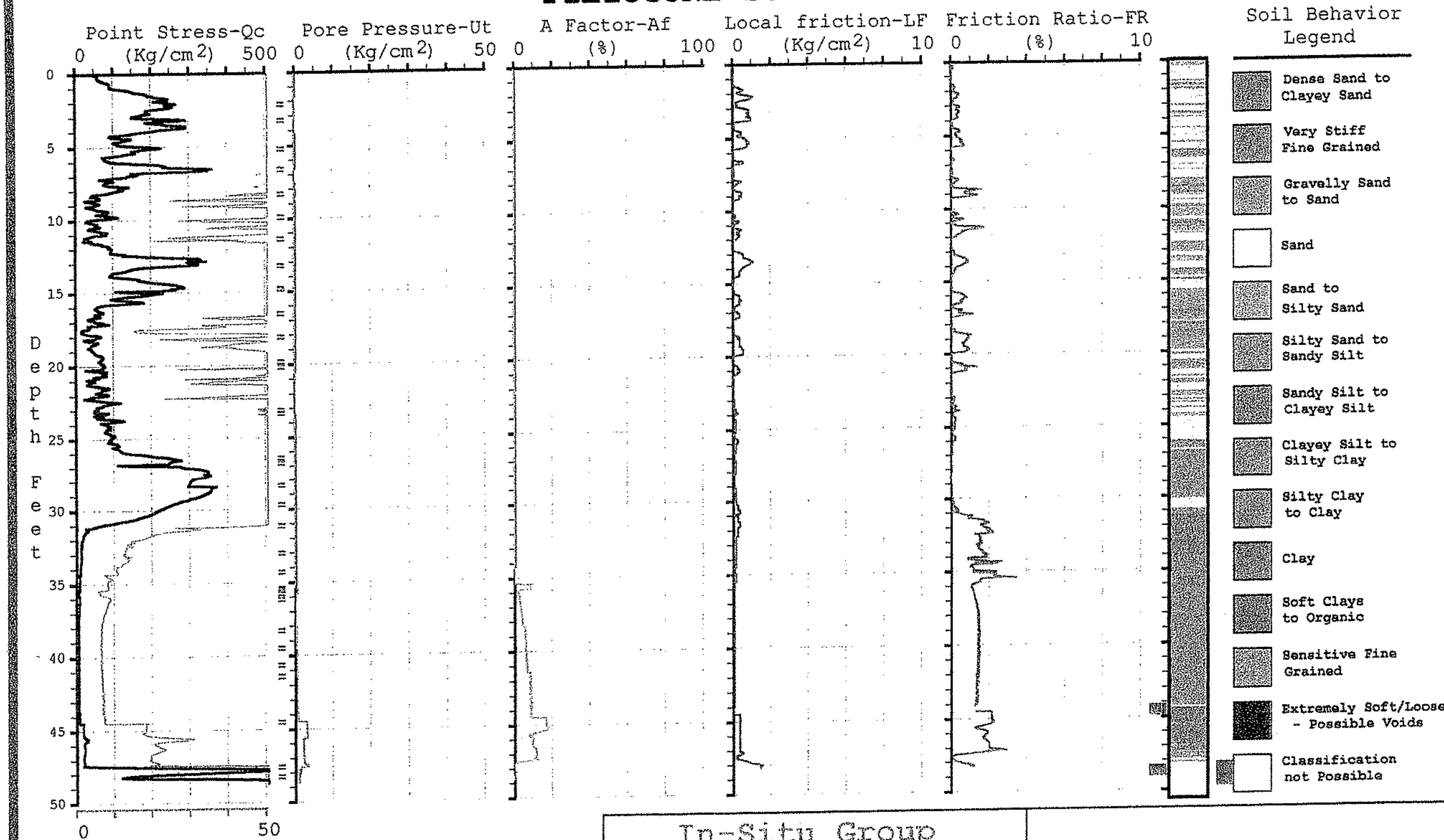
Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding Number # PP-4-
Test Date 01-17-2001 12:15:40

REV: 98 - 3.10 - 0

PIEZOCONE SOUNDING



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding # PP-5-
Test Date 02-26-2001 15:51:48

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

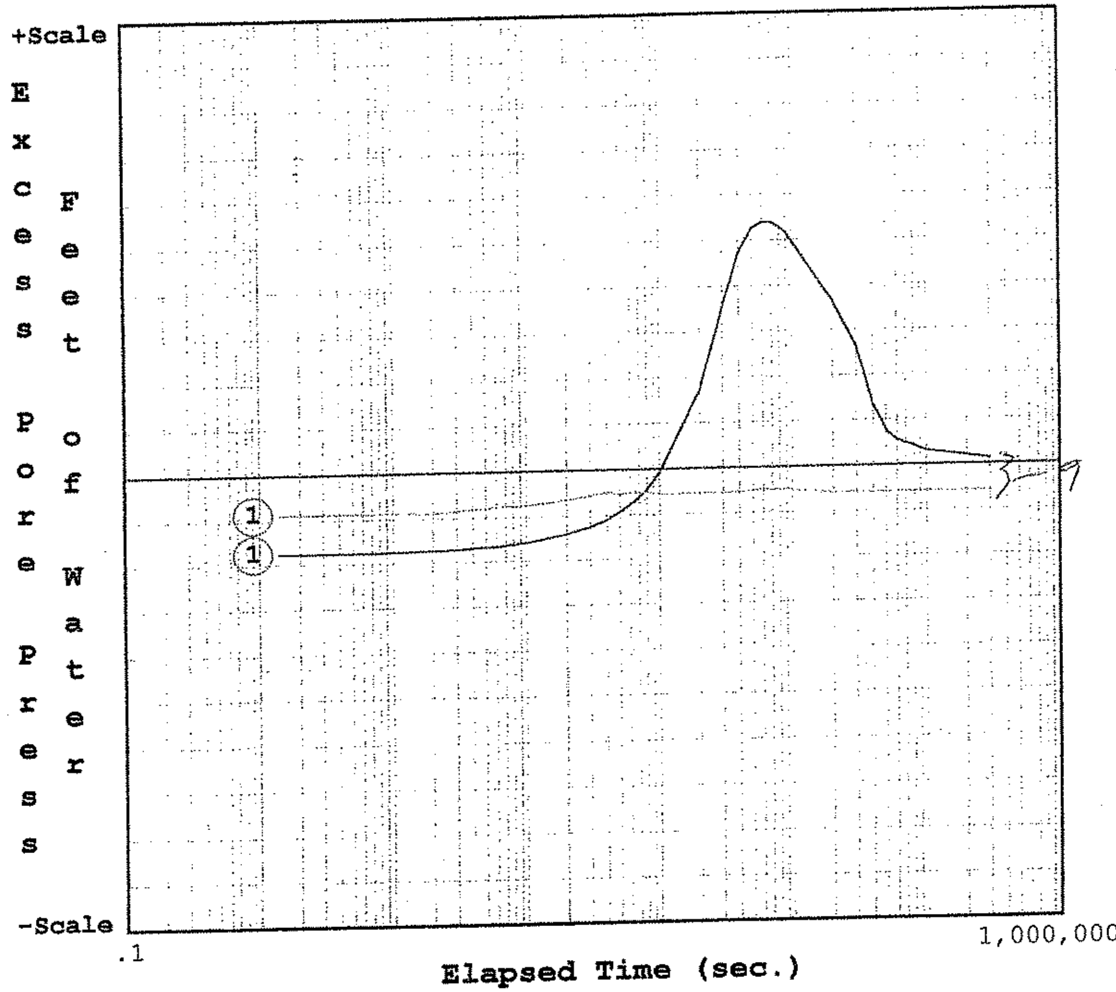
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
29	GRAVELLY SAND TO SAND	327.1	.1	55	1.062	>85%	39-41	719	--	--	--	--
30	GRAVELLY SAND TO SAND	219	.2	36	1.098	65%-85%	37-39	481	--	--	--	--
31	SAND TO SILTY SAND	76.1	.2	19	1.129	42%-50%	33-35	167	--	--	--	--
32	SANDY SILT TO CLAYEY SILT	17.2	.2	7	1.16	35-42%	25-27	37	--	--	--	--
33	SANDY SILT TO CLAYEY SILT	13.3	.2	5	1.191	35-42%	<25	29	.54	5.9	.04	1-1.5
34	CLAYEY SILT TO SILTY CLAY	10.6	.1	5	1.218	--	--	--	.38	5.6	.06	1-1.5
35	CLAYEY SILT TO SILTY CLAY	8.2	.1	4	1.245	--	--	--	.37	7.9	.06	1-1.5
36	CLAYEY SILT TO SILTY CLAY	8	.1	4	1.273	--	--	--	.34	7.6	.06	1-1.5
37	CLAYEY SILT TO SILTY CLAY	7.6	.1	4	1.3	--	--	--	.28	6.8	.07	1
38	CLAYEY SILT TO SILTY CLAY	6.8	.1	3	1.327	--	--	--	.27	6.8	.07	1
39	CLAYEY SILT TO SILTY CLAY	6.7	.1	3	1.355	--	--	--	.27	6.8	.07	1
40	CLAYEY SILT TO SILTY CLAY	6.7	.1	3	1.382	--	--	--	.27	6.8	.07	1
41	CLAYEY SILT TO SILTY CLAY	6.8	.1	3	1.409	--	--	--	.28	7	.07	1
42	CLAYEY SILT TO SILTY CLAY	7	.1	4	1.437	--	--	--	.29	7.2	.06	1
43	CLAYEY SILT TO SILTY CLAY	7.2	.1	4	1.464	--	--	--	.34	7	.06	1
44	CLAYEY SILT TO SILTY CLAY	8.1	.1	4	1.491	--	--	--	--	--	--	--
45	SANDY SILT TO CLAYEY SILT	19.2	.2	8	1.522	35-42%	<25	42	--	--	--	--
46	SANDY SILT TO CLAYEY SILT	23.6	.4	9	1.553	35-42%	25-27	51	--	--	--	--
47	SANDY SILT TO CLAYEY SILT	23.9	.2	10	1.584	35-42%	25-27	52	--	--	--	--
48	GRAVELLY SAND TO SAND	355.2	1.1	59	1.62	>85%	39-41	781	--	--	--	--
49	--	755.5	--	--	--	--	--	--	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-5-
Test Date 02-26-2001 15:51:48

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 26 Ft.

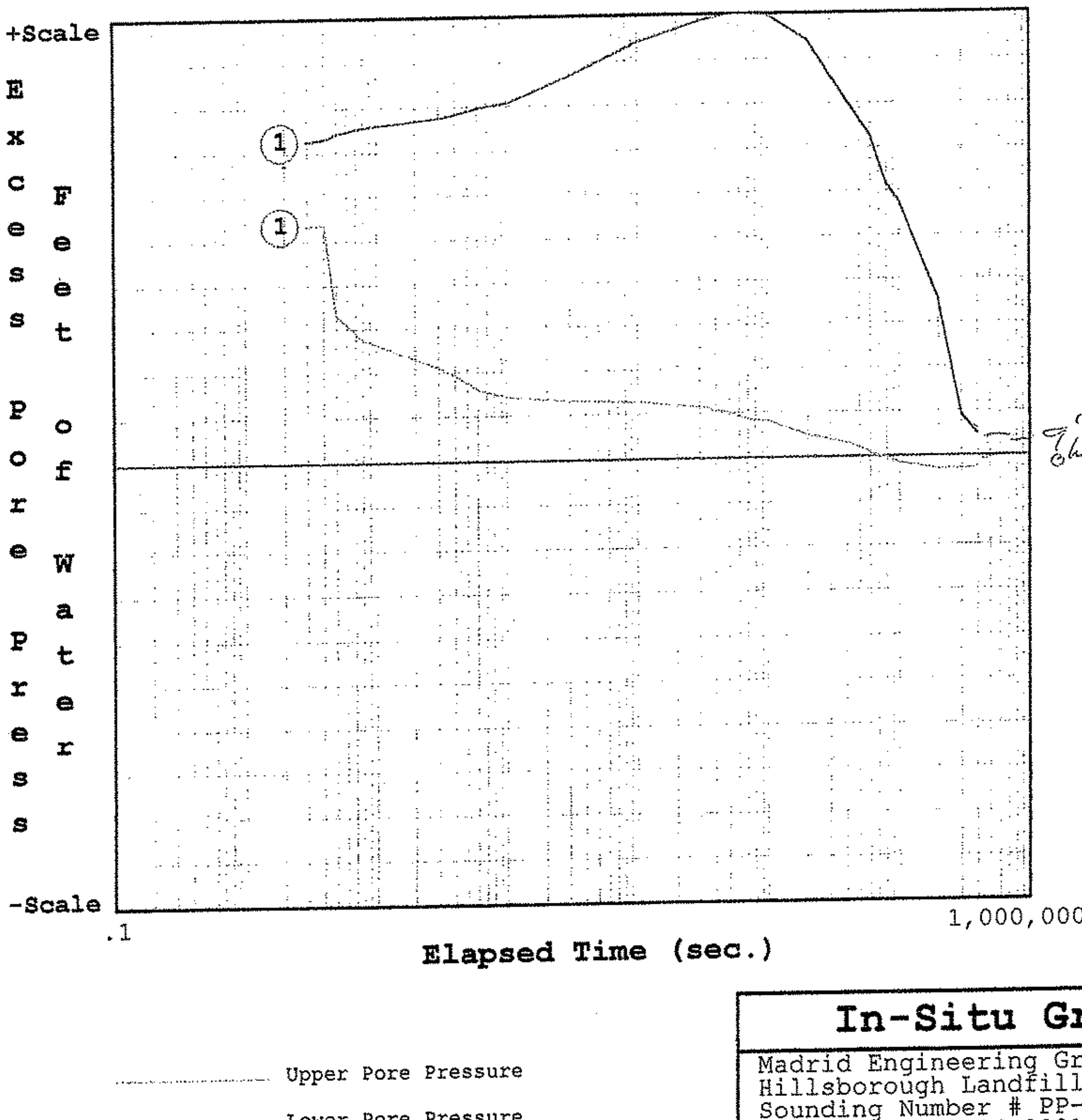
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	35.4	50

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-5-
Test Date 02-26-2001 15:51:48

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 30 Ft.

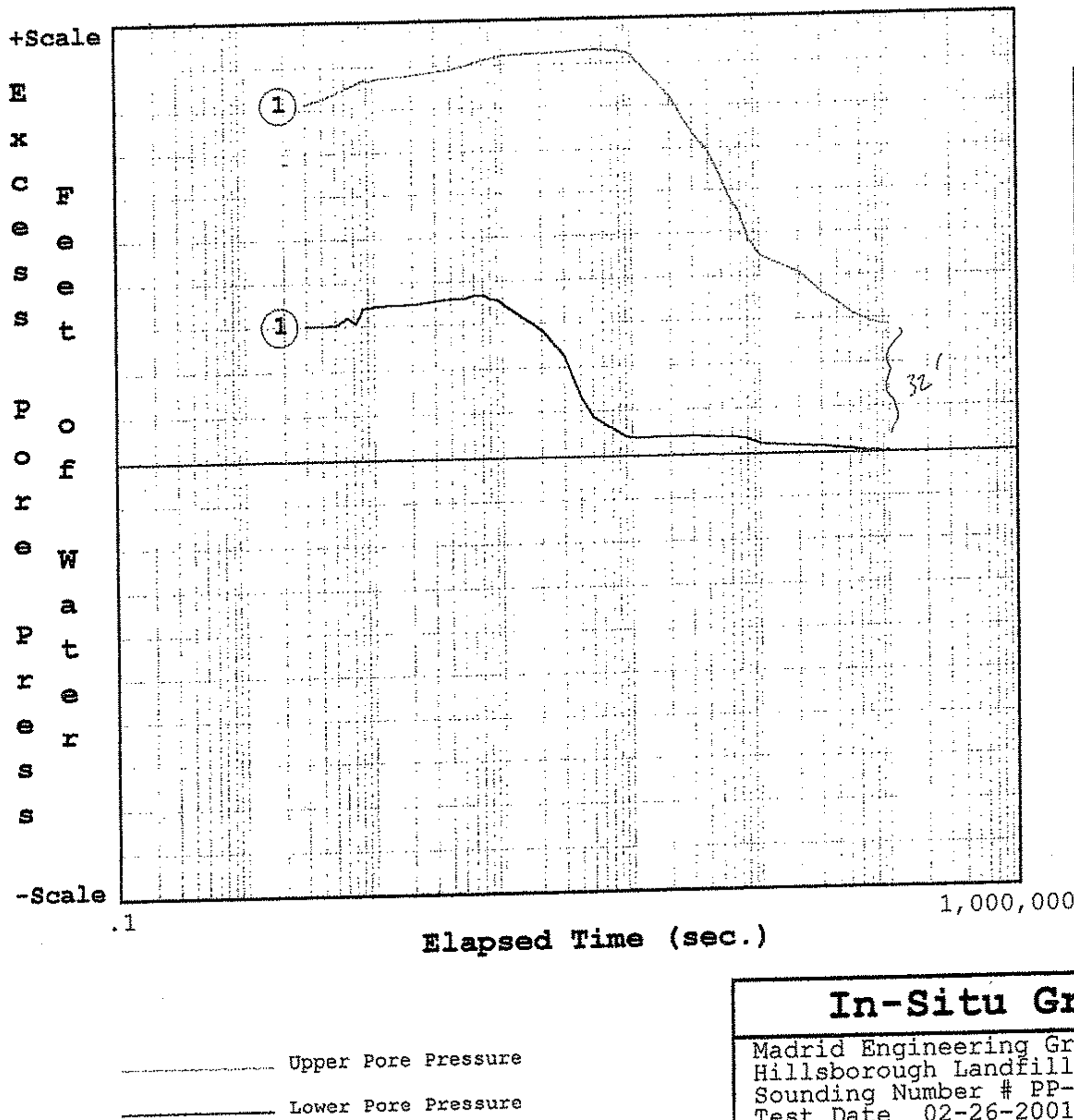
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	44.5	50

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-5-
Test Date 02-26-2001 15:51:48

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 47 Ft.

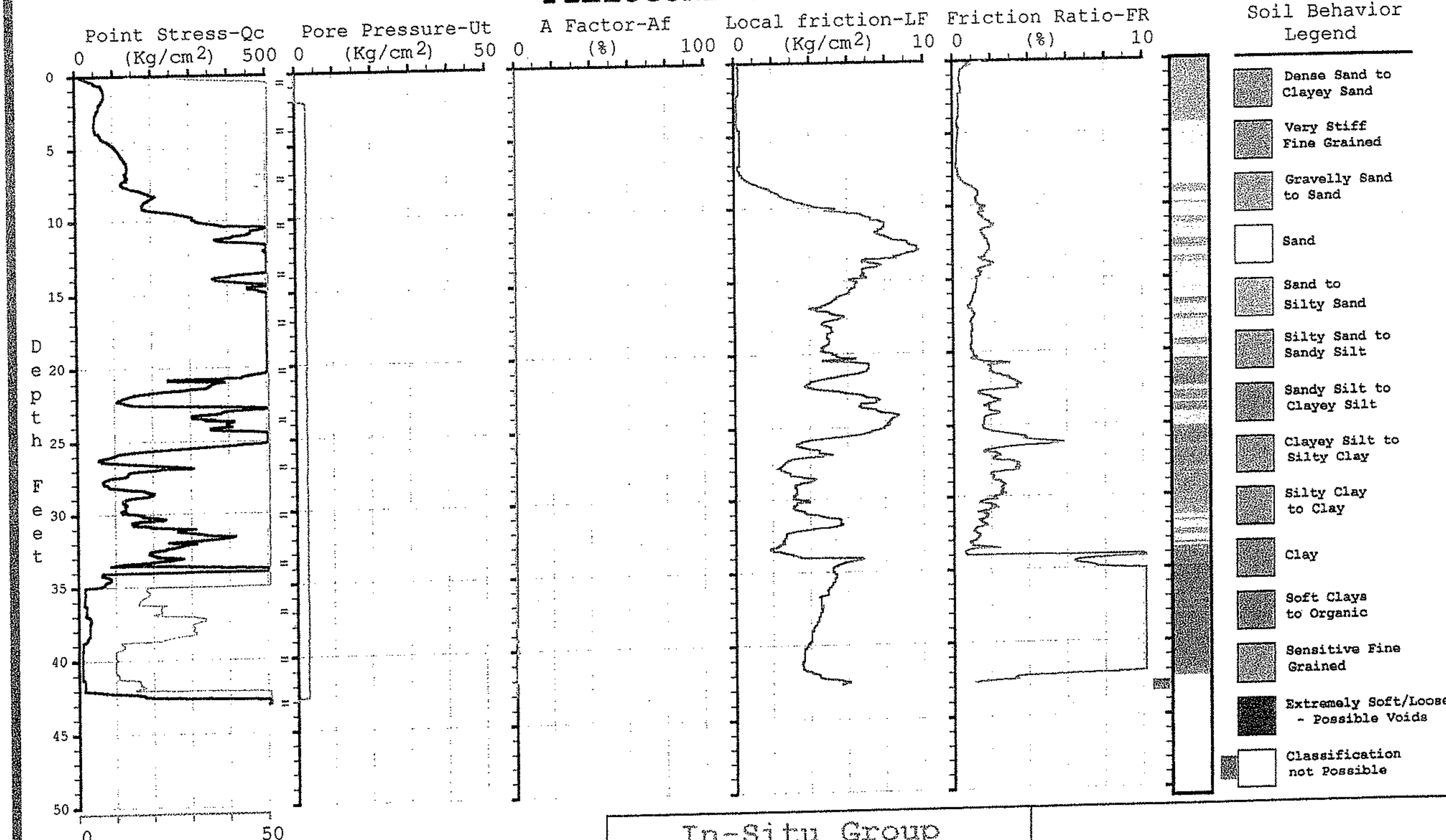
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	47.5	50 ?

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-5-
Test Date 02-26-2001 15:51:48

REV: 98 - 3.10 - 0

PIEZOCONE SOUNDING



'=' Push Interrupted To Add Rod, Pore Pressure Decay Data May Be Available

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding # PP-6
Test Date 03-05-2001 09:22:07

REV: 98 - 3.10 - 0

PIEZOCONE SOIL BEHAVIOR TABLE

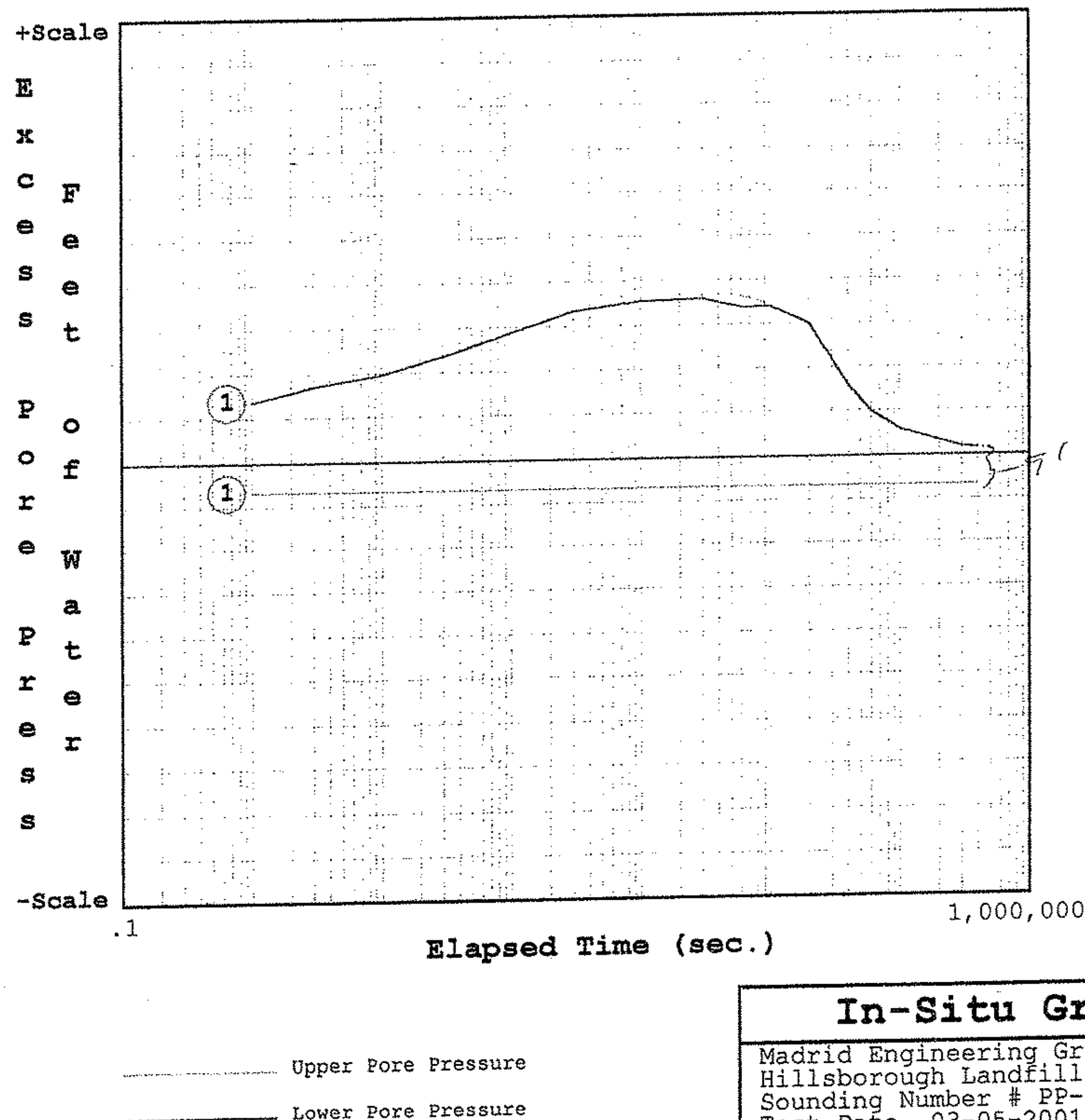
Depth Feet	Soil Behavior Type	Qc (Kg/cm2)	Lf (Kg/cm2)	N #	Vertical Effective Stress	Relative Density (%)	Friction Angle (degrees)	Young Modulus (Kg/cm2)	Undrained Shear Strength	Sens.	Comp.	OCR
34	DENSE SAND TO CLAYEY SAND	221.9	4.2	111	1.129	65%-85%	37-39	488	--	--	--	--
35	SOFT CLAYS TO ORGANIC	45.6	5.2	46	1.145	--	--	--	2.73	.5	0	6
36	SOFT CLAYS TO ORGANIC	18.5	5	18	1.161	--	--	--	1.03	.3	.01	3
37	SOFT CLAYS TO ORGANIC	26.6	4.5	27	1.177	--	--	--	1.53	.5	0	6
38	SOFT CLAYS TO ORGANIC	30	4.4	30	1.193	--	--	--	1.74	.6	0	6
39	SOFT CLAYS TO ORGANIC	14.4	4	14	1.209	--	--	--	.76	.3	.01	3
40	SOFT CLAYS TO ORGANIC	10	3.7	10	1.225	--	--	--	.49	.2	.02	1-1.5
41	SOFT CLAYS TO ORGANIC	11.8	3.6	12	1.241	--	--	--	.6	.3	.02	1-1.5
42	VERY STIFF FINE GRAINED	67	4.5	67	1.272	--	--	--	4.04	.6	0	6
43	SAND	475.4	5.7	95	1.305	>85%	41-43	1045	--	--	--	--

In-Situ Group

Madrid Engineering Group
Hillsborough Co. Landfill
Sounding # PP-6
Test Date 03-05-2001 09:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 29 Ft.

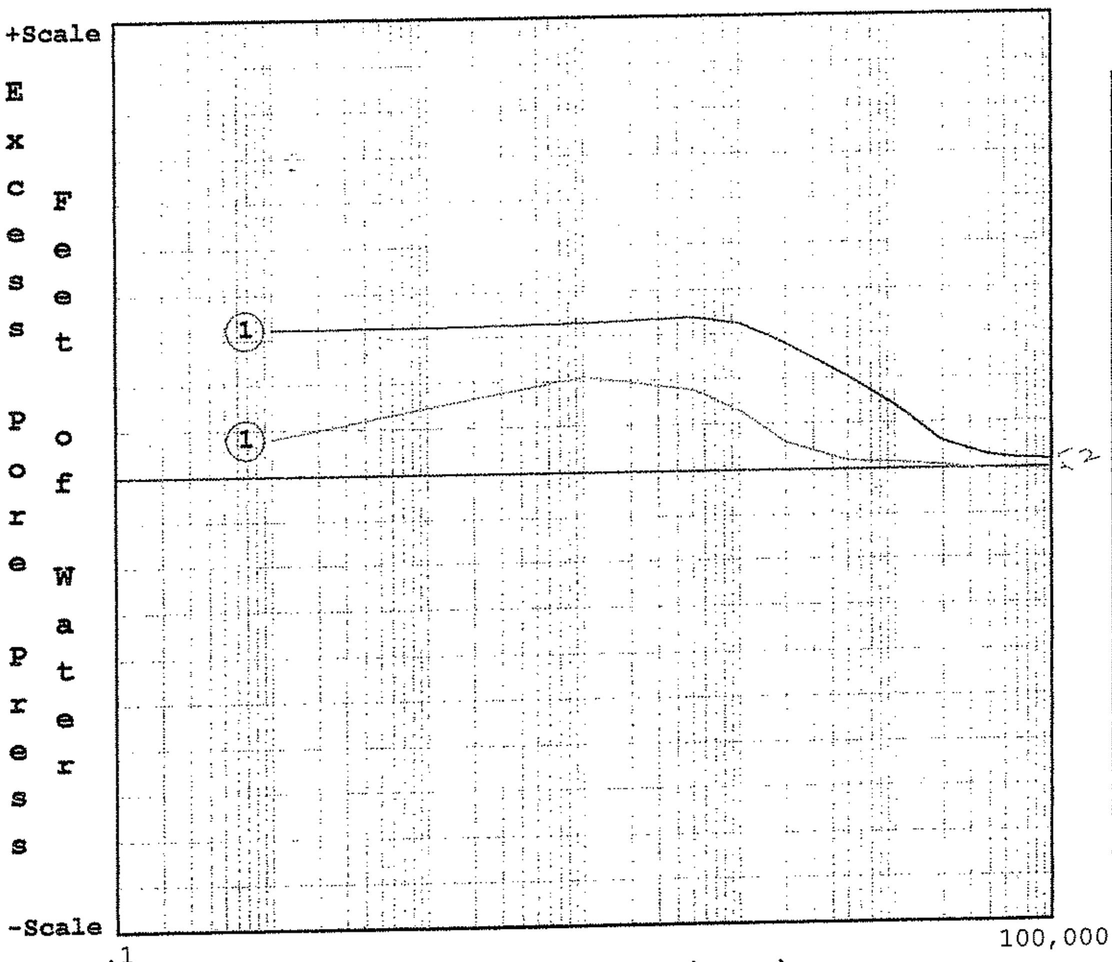
Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	36.8	50

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-6
Test Date 03-05-2001 09:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 33 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	40	50

——— Upper Pore Pressure

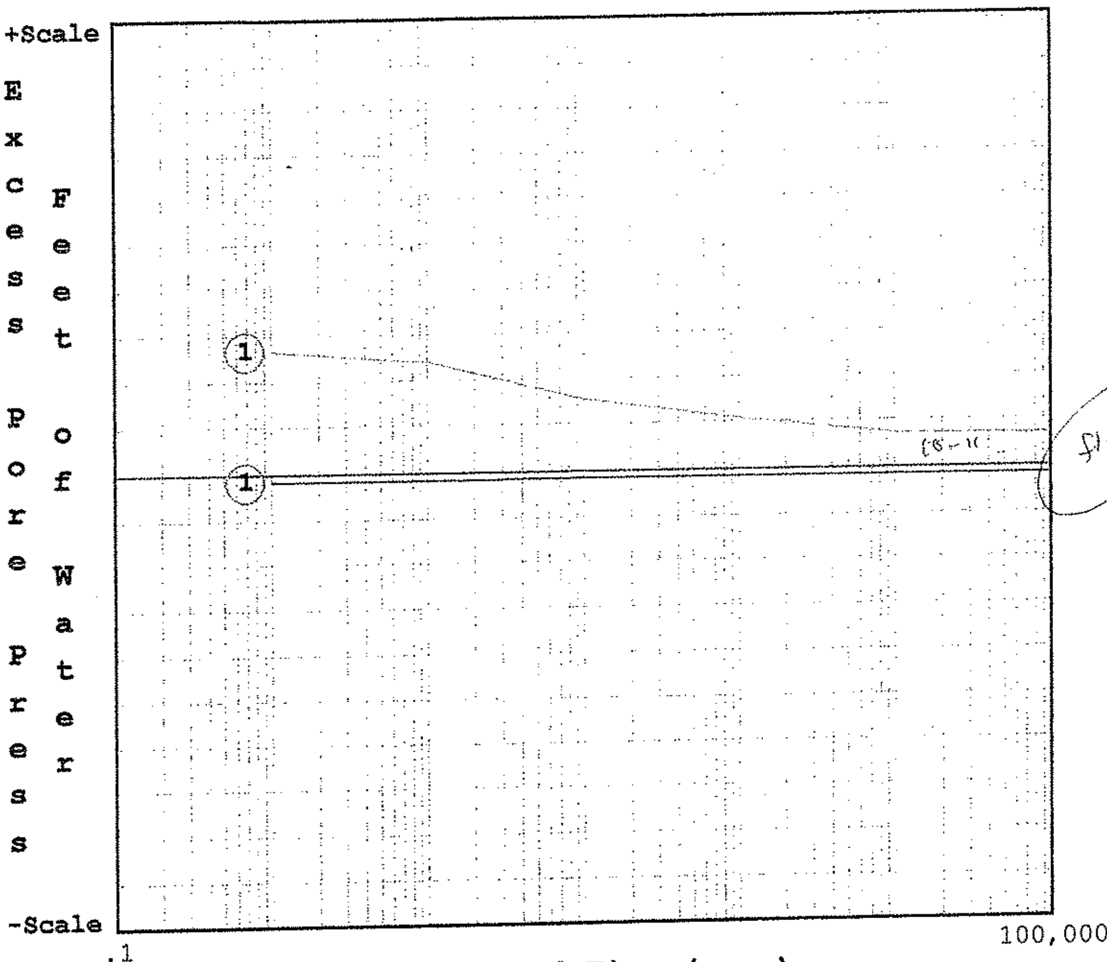
——— Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-6
Test Date 03-05-2001 09:22:07

REV: 98 - 3.10

STATIC PORE PRESSURE DECAY



Ground Water Depth... 42 Ft.

Track Number	Depth (Ft)	+/- Scale (Ft H2O)
1	43	50

Upper Pore Pressure

Lower Pore Pressure

In-Situ Group

Madrid Engineering Group
Hillsborough Landfill
Sounding Number # PP-6
Test Date 03-05-2001 09:22:07

Appendix B

Laboratory Test Results

Oedometer (Consolidation) Test:

Soil Sample # 1: (from PC 5, 30~35 ft.)

Silty Clay

Water Content = 85%

Specific Gravity = 2.63

Specimen diameter = 50 mm and height = 19.0500 mm = 0.75 in.

Wet mass of specimen = 63.2 grams

Initial Void Ratio, $e_o = 2.2355$

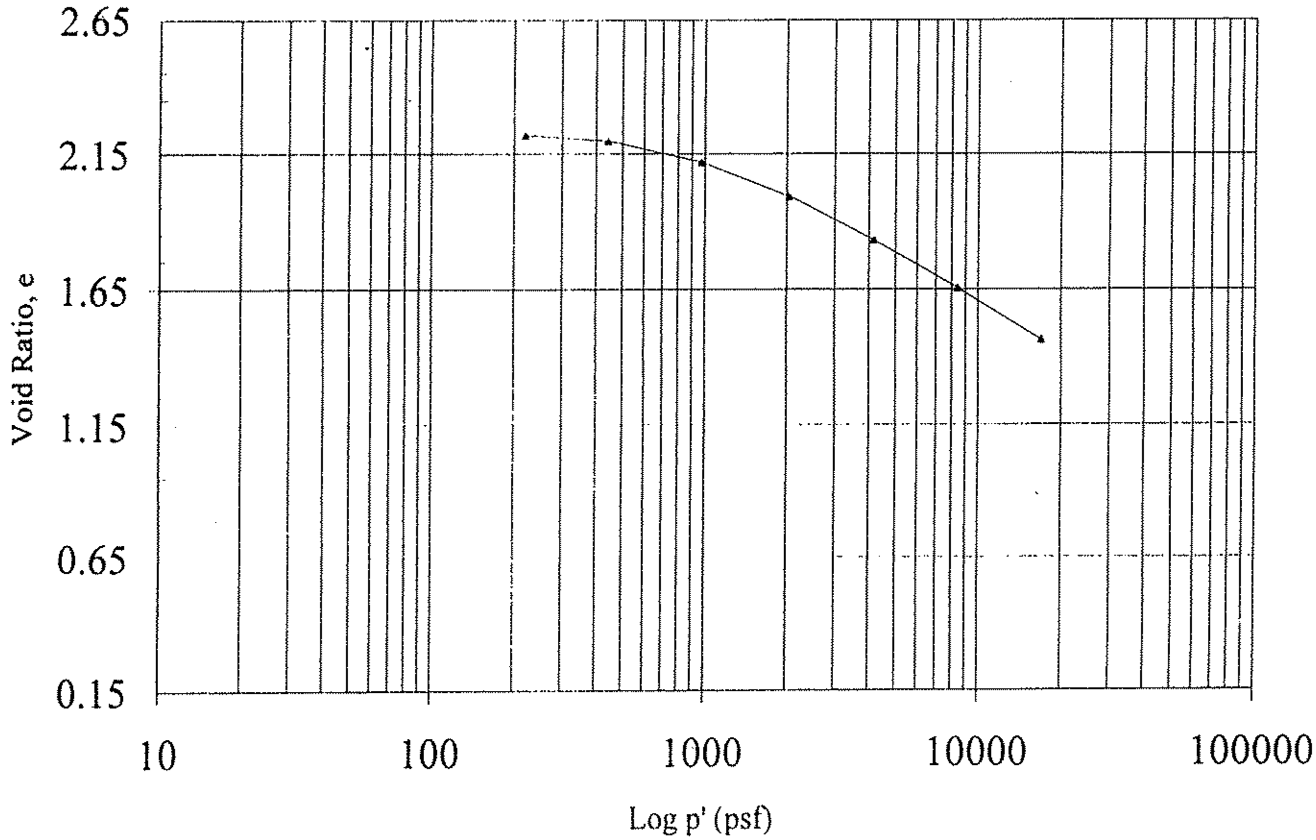
$$Se = wG_s$$

$$e_o = \frac{wG_s}{S} = \frac{0.85 x 2.63}{1} = 2.2355$$

Change in Void Ratio = $\Delta e = \frac{\Delta H (1 + e_o)}{H}$

σ' (psf)	Height (in) H	Change in Height (in) △H	Change in Void Ratio △e	Void Ratio e
220	0.75	0.0038	0.0164	2.2191
440	0.7462	0.0052	0.0224	2.1967
970	0.741	0.0183	0.0789	2.1177
2030	0.7227	0.02905	0.1253	1.9924
4155	0.69365	0.03725	0.1607	1.8317
8400	0.6564	0.0406	0.1751	1.6566
16960	0.6158	0.0444	0.1915	1.4650

e vs. Log p' curve (for sample # 1, PC 5, 30~35 ft.)



Compression Index
$C_c = 0.61$

Average Coeff. of Consolidation
$C_v \simeq .007$ in²/min.

MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	DATE: 3/01 SCALE:	Southeast Hillsborough County Landfill, Florida

Oedometer (Consolidation) Test:

Soil Sample # 2: (from PC 5, 39~41 ft.)

Grayish Clay

Water Content = 160%

Specific Gravity = 2.65

Specimen diameter = 50 mm and height = 19.0500 mm = 0.75 in.

Wet mass of specimen = 49 grams

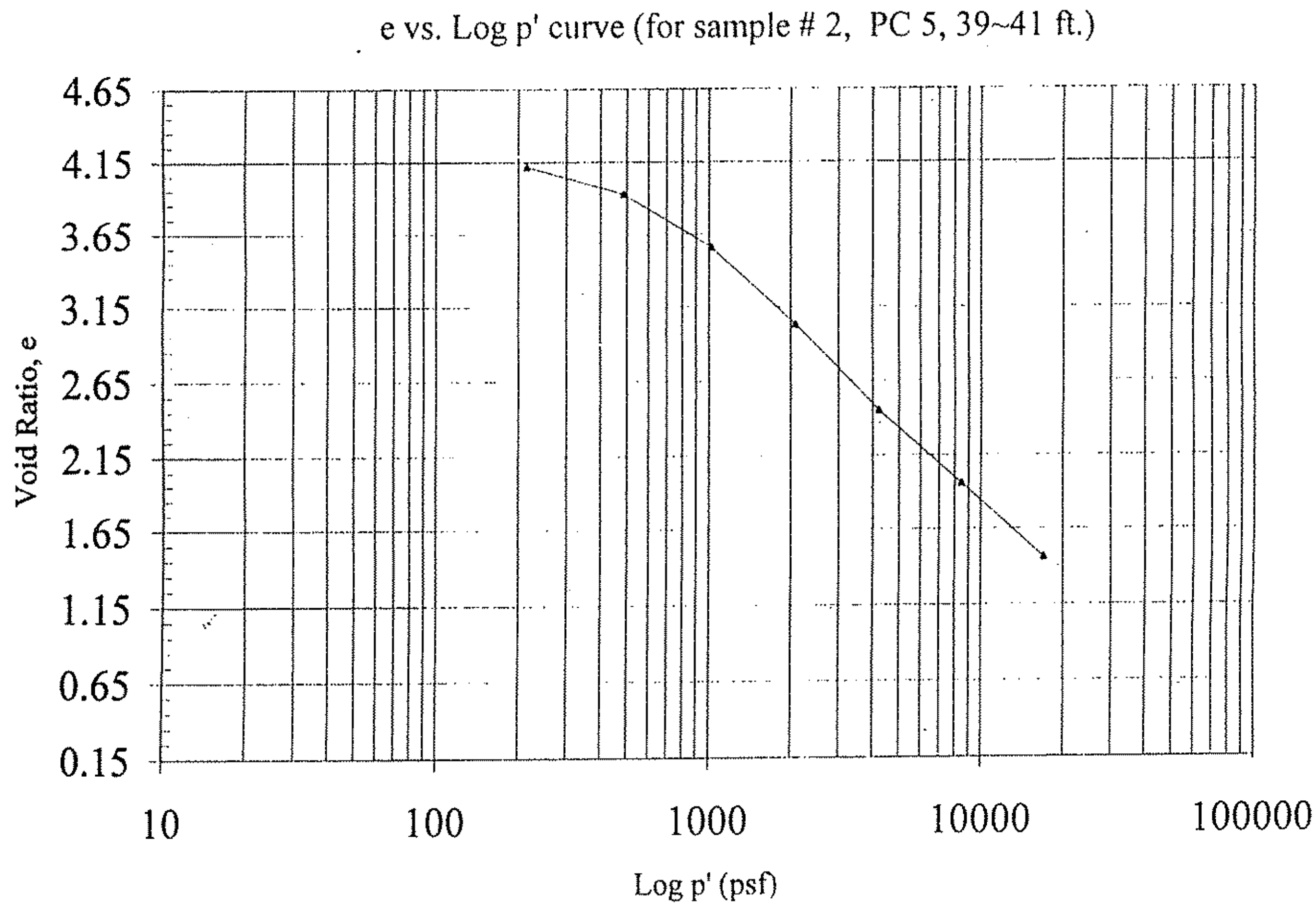
Initial Void Ratio, $e_o = 4.24$

$$Se = wG_s$$

$$e_o = \frac{wG_s}{S} = \frac{1.60 x 2.65}{1} = 4.24$$

Change in Void Ratio = $\Delta e = \frac{\Delta H(1+e_o)}{H}$

σ' (psf)	Height (in) H	Change in Height (in) △H	Change in Void Ratio △e	Void Ratio e
215	0.75	0.0175	0.1223	4.1177
485	0.7325	0.02705	0.1890	3.9287
1020	0.70545	0.05225	0.3651	3.5637
2080	0.6532	0.0762	0.5324	3.0313
4210	0.577	0.0828	0.5785	2.4528
8465	0.4942	0.0702	0.4905	1.9623
17020	0.424	0.0718	0.5016	1.4607

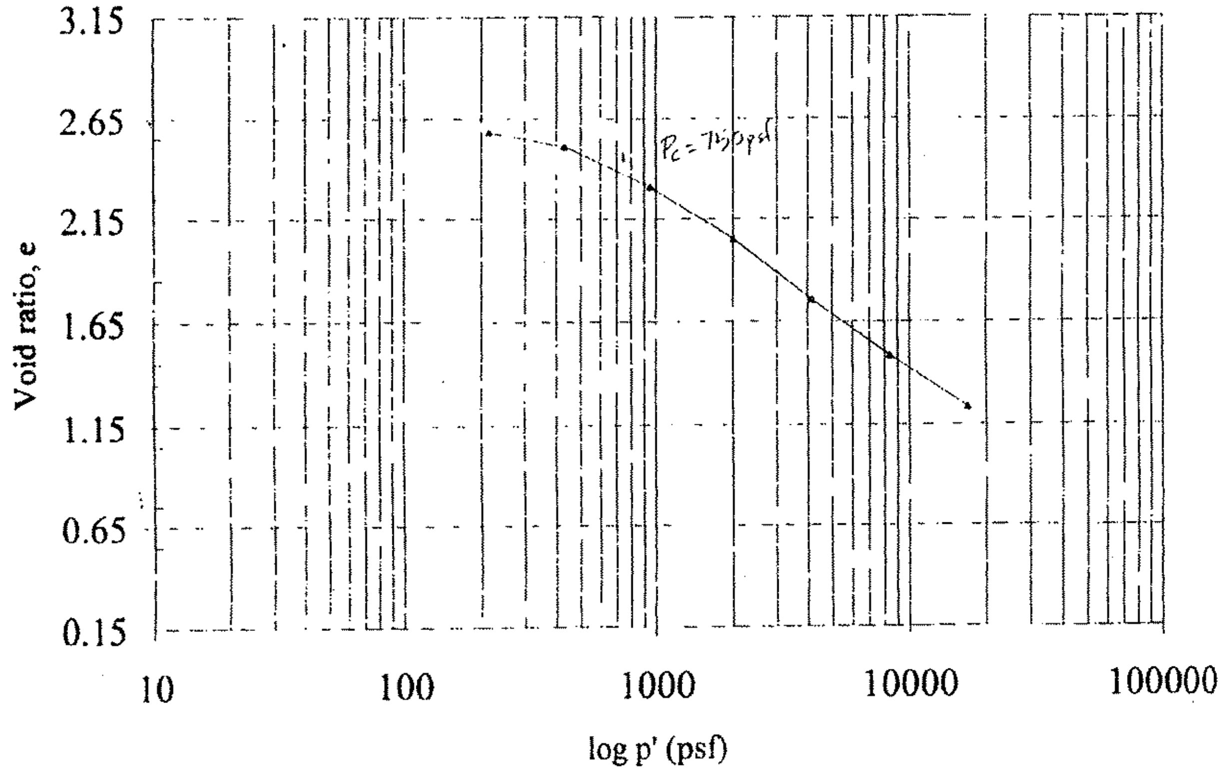


Compression Index
$C_c = 1.36$

Avg. degree of consolidation
$C_v \approx 0.0023$ in^2/min

MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	DATE: 3/01 SCALE:	Southeast Hillsborough County Landfill, Florida

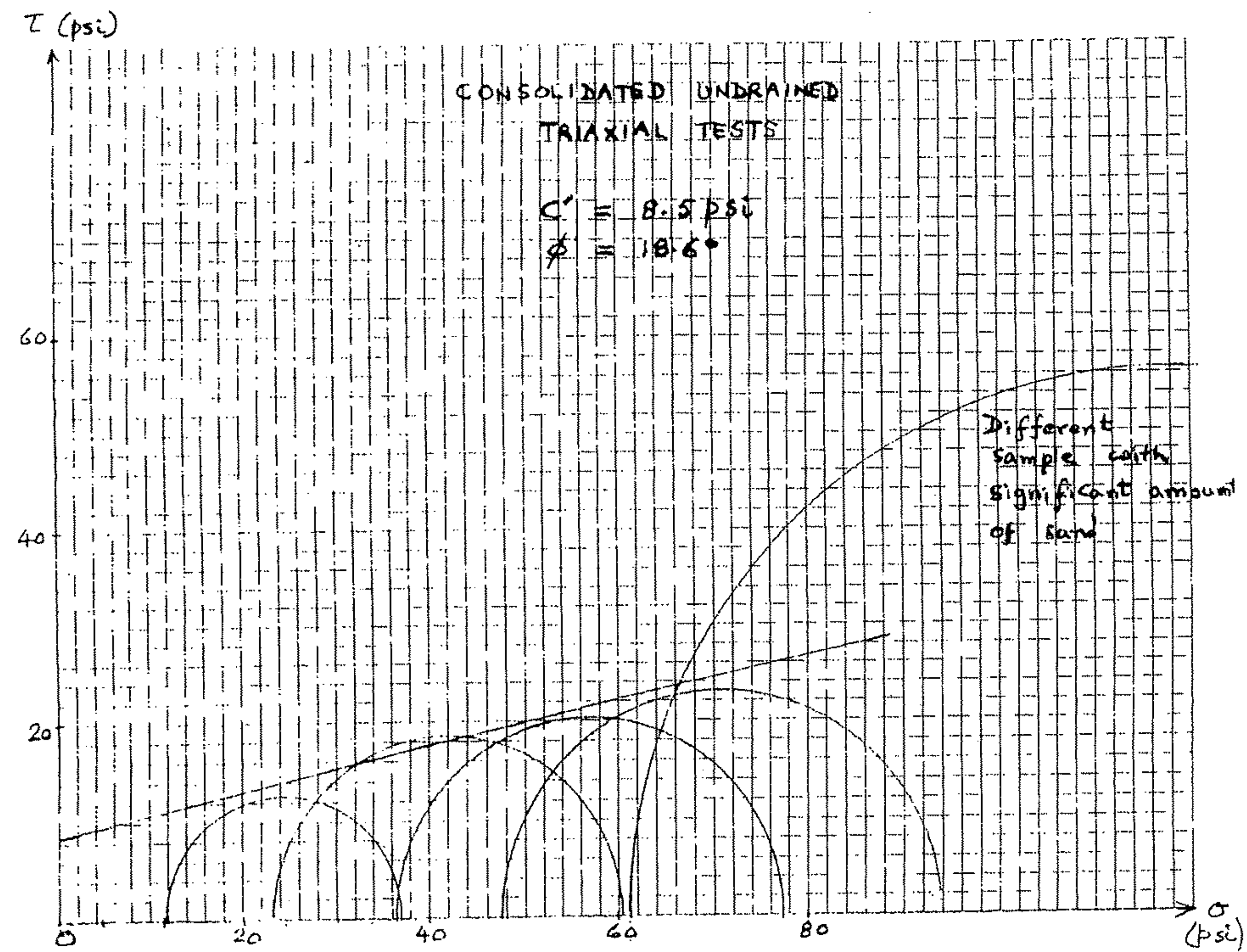
e vs. Log p' curve (for sample # 3, PC 5, 37~39 ft.)



Compression Index
$C_c = 0.91$
e_o 4.18

Average Coeff. of Consol.
C_v = .007 in²/min

MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	DATE: 3/01 SCALE:	Southeast Hillsborough County Landfill, Florida



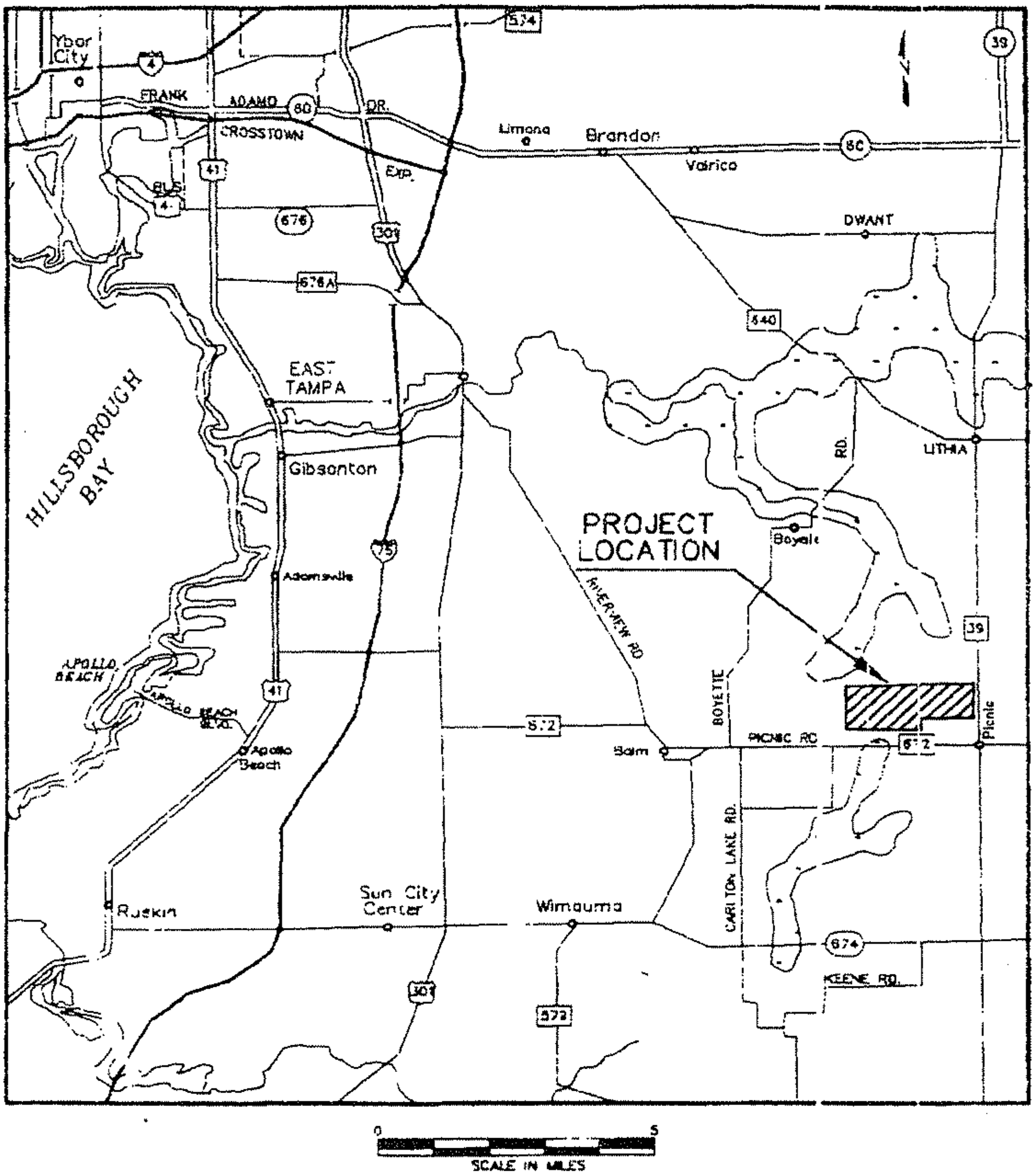
MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	DATE: 3/01 SCALE:	Triaxial Test Results Southeast Hillsborough County Landfil

Appendix C – Summary of CPT Data Sheet Results

- Point Stress – a continuous readout of the end resistance on the cone
- Pore Pressure – the cone is equipped with a porous stone that allows water pressure to be measured at various depths
- A-factor – also known as the pore pressure factor, a measure of the ratio of pore pressure to vertical effective stress
- Local Friction – the friction developed along the side of the cone
- Friction Ratio - f_s / q_c, as discussed above
- Soil Legend – the inferred soil type based on CPT measurements
- N # - the equivalent Standard Penetration Test (SPT) blow count, based on established correlation to the CPT
- Vertical Effective Stress – the stress of the soil minus the effect of the water table, at various depths.
- Relative Density – granular soil density as a percentage of the maximum theoretical density.
- Friction angle – a measure of granular strength
- Young's modulus – the stress-strain characteristics of the soil
- Undrained Shear Strength – the strength of a cohesive soil
- Sensitivity – a measure of the loss of strength due to remolding of a cohesive soil.
- Comp. – Soil compressibility, for cohesive soil
- OCR – the over-consolidation ratio, for clay, the ratio of the maximum past consolidation stress to the current effective stress.

Appendix D

Figures



LOCATION MAP

SCS ENGINEERS

MADRID ENGINEERING GROUP, INC.
GEOENVIRONMENTAL CONSULTANTS
175 E Summerlin St, Bartow, FL
863 533-9007 Fax: 533-8997
The Earth is our Business

Site Location Map

Southeast County Landfill
Hillsborough County, FL

DATE: 3/01	Revised: N/A	Drawn By: N/A	Checked By: LDM	MEG Project No. 2872	SCALE: As Shown

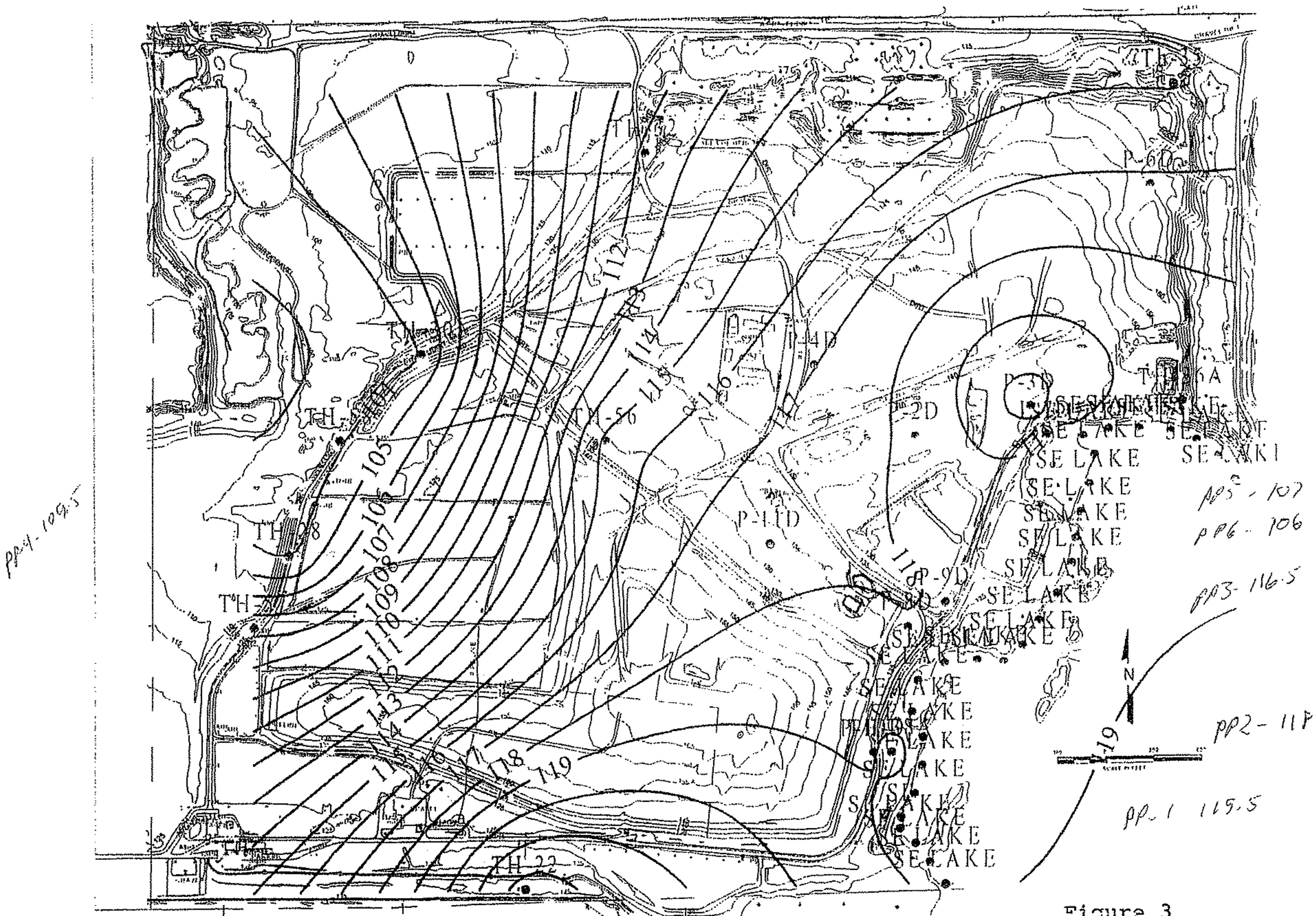


Figure 3

MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	DATE: 3/01 SCALE: As Shown	Southeast County Landfill Hillsborough County, FL

09200020.11
Madrid - wasteclay Report

P.O. Box 2506
175 E Summerlin St
Bartow, FL 33831
Phone: 863-533-9007
Fax: 863-533-8997

Madrid Engineering Group, Inc.

The Earth Is Our Business sm

Fax

To:	Joe O'Neill	**From:**	Larry D. Madrid
Fax:	813-623-6757	**Date:**	July 23, 2001
Phone:	813-621-0080	**Total Pages:**	Over 20
Re:	Consolidation Time results (Cv)	**CC:**	

XX **Urgent** ☐ **For Review** ☐ **Please Comment** ☐ **Please Reply** ☐ **Please Recycle**

•Comments:

I am sending you the test results. These results have been re-calculated, as Dr. Gunaratne from USF College of Engineering, Geotechnical Program is on sabbatical out of the country for the next several weeks. The grad student who ran the tests for us, Mr. Naim Muhammed, has found the raw data and completed the curves on them using the Taylor's method. I checked his results for a few random curves and I agree with them – however I note that a small difference in interpretation of the initial straight line portion can greatly affect the results. This method, being graphical is subject to some interpretation, so it is not surprising that his results (see first page summary) are different from the averages indicated previously by Dr. Gunaratne. I am also not surprised that the field calculated results also vary from the laboratory. Every graphic I have ever seen on the Cv indicates that it varies with the load, and there is no direct correlation. This appears to be the case here as well.

Oedometer (Consolidation) Test:

Specimen diameter = 50 mm and height = 19.0500 mm = 0.75 in.

Soil Sample # 1: (from PC 5, 30~35 ft.)

Consolidation

Non Linear

Linear

σ' (psf)	Height (in) H	Change in Height (in) △H	sqrt (time) $(min)^{0.5}$ $\sqrt{t_{90}}$	Cv (in^2/min)
220	0.75	0.0038	4.20	0.0067
440	0.7462	0.0052	6.05	0.0032
970	0.741	0.0183	4.85	0.0048
2030	0.7227	0.02905	4.75	0.0047
4155	0.69365	0.03725	4.95	0.0039
8400	0.6564	0.0406	4.50	0.0042
16960	0.6158	0.0444	4.00	0.0047
			avg =	0.0046

?

0.004375

Per 1st Report 0.007

(0.00049 cm^2/s)
4.9×10^{-4} cm^2/s

Soil Sample # 2: (from PC 5, 39~41 ft.)

Non Linear

Linear

σ' (psf)	Height (in) H	Change in Height (in) △H	sqrt (time) $(min)^{0.5}$ $\sqrt{t_{90}}$	Cv (in^2/min)
215	0.75	0.0175	9.00	0.0014
485	0.7325	0.02705	9.50	0.0012
1020	0.70545	0.05225	9.50	0.0011
2080	0.6532	0.0762	10.05	0.0008
4210	0.577	0.0828	10.40	0.0006
8465	0.4942	0.0702	10.70	0.0004
17020	0.424	0.0718	10.30	0.0003
			avg =	0.0008

0.000525

0.0023

Soil Sample # 3: (from PC 5, 37~39 ft.)

(0.000086
0.86×10^{-5} cm^2/s)

Non Linear

Linear

σ' (psf)	Height (in) H	Change in Height (in) △H	sqrt (time) $(min)^{0.5}$ $\sqrt{t_{90}}$	Cv (in^2/min)
215	0.75	0.0139	5.50	0.0039
430	0.7361	0.0155	8.70	0.0015
950	0.7206	0.0401	8.30	0.0015
2010	0.6805	0.0539	6.90	0.0019
4140	0.6266	0.0604	7.90	0.0012
8395	0.5662	0.055	6.20	0.0016
16950	0.5112	0.0521	5.70	0.0015
			avg =	0.0019

0.00155

0.007

(0.00020 cm^2/s
2.0×10^{-4} cm^2/s)

from Taylor Method, c_v

$$\boxed{c_v = \frac{0.848Hr^2}{t_{90}} = \frac{0.848x0.374^2}{4.2^2} = 0.0067\,\text{in}^2/\text{min}}$$

Consolidation Test: (Calculation)

Sample # 1, Test data # 1
Load = 220 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	25.200	0	0	0.00	0.000
6	25.150	0.0005	0.1	0.32	0.013
15	25.130	0.0007	0.25	0.50	0.018
30	25.100	0.001	0.5	0.71	0.025
45	25.080	0.0012	0.75	0.87	0.030
60	25.070	0.0013	1	1.00	0.033
90	25.055	0.00145	1.5	1.22	0.037
120	25.040	0.0016	2	1.41	0.041
180	25.020	0.0018	3	1.73	0.046
240	25.010	0.0019	4	2.00	0.048
360	24.990	0.0021	6	2.45	0.053
480	24.975	0.00225	8	2.83	0.057
600	24.968	0.00232	10	3.16	0.059
900	24.947	0.00253	15	3.87	0.064
1200	24.930	0.0027	20	4.47	0.069
1800	24.910	0.0029	30	5.48	0.074
2700	24.895	0.00305	45	6.71	0.077
3600	24.885	0.00315	60	7.75	0.080
5400	24.871	0.00329	90	9.49	0.084
7200	24.865	0.00335	120	10.95	0.085
14400	24.847	0.00353	240	15.49	0.090
28800	24.840	0.0036	480	21.91	0.091
86400	24.820	0.0038	1440	37.95	0.097

Consolidation Test: (Calculation)

Sample # 1, Test data # 2
Load = 440 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	24.820	0	0	0.00	0.000
6	24.750	0.0007	0.1	0.32	0.018
15	24.725	0.00095	0.25	0.50	0.024
30	24.710	0.0011	0.5	0.71	0.028
45	24.700	0.0012	0.75	0.87	0.030
60	24.693	0.00127	1	1.00	0.032
90	24.680	0.0014	1.5	1.22	0.036
120	24.670	0.0015	2	1.41	0.038
240	24.640	0.0018	4	2.00	0.046
360	24.620	0.002	6	2.45	0.051
480	24.601	0.00219	8	2.83	0.056
600	24.589	0.00231	10	3.16	0.059
900	24.560	0.0026	15	3.87	0.066
1200	24.544	0.00276	20	4.47	0.070
1500	24.528	0.00292	25	5.00	0.074
1800	24.515	0.00305	30	5.48	0.077
2700	24.492	0.00328	45	6.71	0.083
3600	24.475	0.00345	60	7.75	0.088
5400	24.450	0.0037	90	9.49	0.094
7200	24.438	0.00382	120	10.95	0.097
14400	24.390	0.0043	240	15.49	0.109
28800	24.360	0.0046	480	21.91	0.12
86400	24.300	0.0052	1440	37.95	0.13

Sample # 1, Test data # 3
Load = 970 psf

Time (sec)	**Reading (dial gauge)**	**Settlement (in)**	**Time (min)**	$\sqrt{Time}$ **(min$^{1/2}$)**	**Settlement (mm)**
0	24.300	**0**	0	0.00	0.000
6	24.050	0.0025	0.1	0.32	0.064
15	23.970	0.0033	0.25	0.50	0.084
30	23.900	0.004	0.5	0.71	0.102
45	23.850	0.0045	0.75	0.87	0.114
60	23.810	0.0049	1	1.00	0.124
90	23.750	0.0055	1.5	1.22	0.140
120	23.690	0.0061	2	1.41	0.155
240	23.550	0.0075	4	2.00	0.191
360	23.450	0.0085	6	2.45	0.216
480	23.365	0.00935	8	2.83	0.237
600	23.302	0.00998	10	3.16	0.253
900	23.185	0.01115	15	3.87	0.283
1200	23.100	0.012	20	4.47	0.305
1500	23.035	0.01265	25	5.00	0.321
1800	22.990	0.0131	30	5.48	0.333
2700	22.885	0.01415	45	6.71	0.359
3600	22.825	0.01475	60	7.75	0.375
5400	22.752	0.01548	90	9.49	0.393
7200	22.710	0.0159	120	10.95	0.404
14400	22.613	0.01687	240	15.49	0.428
28800	22.540	0.0176	480	21.91	0.45
86400	22.470	0.0183	1440	37.95	0.46

Consolidation Test: (Calculation)

Sample # 1, Test data # 4

Load = 2030 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	22.470	0	0	0.00	0.000
6	22.140	0.0033	0.1	0.32	0.084
15	22.000	0.0047	0.25	0.50	0.119
30	21.850	0.0062	0.5	0.71	0.157
45	21.760	0.0071	0.75	0.87	0.180
60	21.685	0.00785	1	1.00	0.199
90	21.580	0.0089	1.5	1.22	0.226
120	21.490	0.0098	2	1.41	0.249
240	21.230	0.0124	4	2.00	0.315
360	21.050	0.0142	6	2.45	0.361
480	20.915	0.01555	8	2.83	0.395
600	20.810	0.0166	10	3.16	0.422
900	20.610	0.0186	15	3.87	0.472
1200	20.470	0.02	20	4.47	0.508
1500	20.365	0.02105	25	5.00	0.535
1800	20.285	0.02185	30	5.48	0.555
2700	20.145	0.02325	45	6.71	0.591
3600	20.070	0.024	60	7.75	0.610
5400	19.985	0.02485	90	9.49	0.631
7200	19.925	0.02545	120	10.95	0.646
14400	19.778	0.02692	240	15.49	0.684
28800	19.680	0.0279	480	21.91	0.709
86400	19.565	0.02905	1440	37.95	0.738

Sample # 1, Test data # 5
Load = 4155 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	19.565	0	0	0.00	0.000
6	19.040	0.00525	0.1	0.32	0.133
15	18.850	0.00715	0.25	0.50	0.182
30	18.750	0.00815	0.5	0.71	0.207
45	18.630	0.00935	0.75	0.87	0.237
60	18.550	0.01015	1	1.00	0.258
90	18.390	0.01175	1.5	1.22	0.298
120	18.280	0.01285	2	1.41	0.326
240	17.940	0.01625	4	2.00	0.413
360	17.710	0.01855	6	2.45	0.471
480	17.535	0.0203	8	2.83	0.516
600	17.390	0.02175	10	3.16	0.552
900	17.130	0.02435	15	3.87	0.618
1200	16.930	0.02635	20	4.47	0.669
1500	16.805	0.0276	25	5.00	0.701
1800	16.695	0.0287	30	5.48	0.729
2700	16.495	0.0307	45	6.71	0.780
3600	16.380	0.03185	60	7.75	0.809
5400	16.253	0.03312	90	9.49	0.841
7200	16.183	0.03382	120	10.95	0.859
14400	16.060	0.03505	240	15.49	0.890
28800	15.952	0.03613	480	21.91	0.92
86400	15.840	0.03725	1440	37.95	0.95

Sample # 1, Test data # 6
Load = 8400 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	15.840	0	0	0.00	0.000
6	15.320	0.0052	0.1	0.32	0.132
15	15.080	0.0076	0.25	0.50	0.193
30	14.880	0.0096	0.5	0.71	0.244
45	14.730	0.0111	0.75	0.87	0.282
60	14.630	0.0121	1	1.00	0.307
90	14.430	0.0141	1.5	1.22	0.358
120	14.290	0.0155	2	1.41	0.394
240	13.860	0.0198	4	2.00	0.503
360	13.580	0.0226	6	2.45	0.574
480	13.350	0.0249	8	2.83	0.632
600	13.165	0.02675	10	3.16	0.679
900	12.860	0.0298	15	3.87	0.757
1200	12.660	0.0318	20	4.47	0.808
1500	12.520	0.0332	25	5.00	0.843
1800	12.413	0.03427	30	5.48	0.870
2700	12.225	0.03615	45	6.71	0.918
3600	12.118	0.03722	60	7.75	0.945
5400	12.005	0.03835	90	9.49	0.974
7200	11.940	0.039	120	10.95	0.991
14400	11.810	0.0403	240	15.49	1.024
28800	11.785	0.04055	480	21.91	1.03
86400	11.780	0.0406	1440	37.95	1.03

Sample # 1, Test data # 7
Load = 16960 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	11.780	0	0	0.00	0.000
6	10.950	0.0083	0.1	0.32	0.211
15	10.780	0.01	0.25	0.50	0.254
30	10.560	0.0122	0.5	0.71	0.310
45	10.380	0.014	0.75	0.87	0.356
60	10.250	0.0153	1	1.00	0.389
90	10.020	0.0176	1.5	1.22	0.447
120	9.850	0.0193	2	1.41	0.490
240	9.420	0.0236	4	2.00	0.599
360	9.140	0.0264	6	2.45	0.671
480	8.930	0.0285	8	2.83	0.724
600	8.650	0.0313	10	3.16	0.795
900	8.375	0.03405	15	3.87	0.865
1200	8.202	0.03578	20	4.47	0.909
1500	8.088	0.03692	25	5.00	0.938
1800	8.010	0.0377	30	5.48	0.958
2700	7.863	0.03917	45	6.71	0.995
3600	7.790	0.0399	60	7.75	1.013
5400	7.705	0.04075	90	9.49	1.035
7200	7.652	0.04128	120	10.95	1.049
14400	7.540	0.0424	240	15.49	1.077
28800	7.445	0.04335	480	21.91	1.10
86400	7.340	0.0444	1440	37.95	1.13

Consolidation Test: (Calculation)

Sample # 2, Test data # 1
Load = 215 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	7.500	0	0	0.00	0.000
6	7.420	0.0008	0.1	0.32	0.020
15	7.350	0.0015	0.25	0.50	0.038
30	7.280	0.0022	0.5	0.71	0.056
45	7.240	0.0026	0.75	0.87	0.066
60	7.210	0.0029	1	1.00	0.074
90	7.160	0.0034	1.5	1.22	0.086
120	7.125	0.00375	2	1.41	0.095
240	6.975	0.00525	4	2.00	0.133
360	6.870	0.0063	6	2.45	0.160
480	6.800	0.007	8	2.83	0.178
600	6.735	0.00765	10	3.16	0.194
900	6.610	0.0089	15	3.87	0.226
1200	6.520	0.0098	20	4.47	0.249
1500	6.420	0.0108	25	5.00	0.274
1800	6.350	0.0115	30	5.48	0.292
2700	6.150	0.0135	45	6.71	0.343
3600	6.030	0.0147	60	7.75	0.373
5400	5.900	0.016	90	9.49	0.406
7200	5.835	0.01665	120	10.95	0.423
14400	5.802	0.01698	240	15.49	0.431
28800	5.772	0.01728	480	21.91	0.439
86400	5.750	0.0175	1440	37.95	0.445

Consolidation Test:

Sample # 2, Test data # 2
Load = 485 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	5.750	0	0	0.00	0.000
6	5.680	0.0007	0.1	0.32	0.018
15	5.630	0.0012	0.25	0.50	0.030
30	5.590	0.0016	0.5	0.71	0.041
45	5.560	0.0019	0.75	0.87	0.048
60	5.530	0.0022	1	1.00	0.056
90	5.475	0.00275	1.5	1.22	0.070
120	5.430	0.0032	2	1.41	0.081
240	5.280	0.0047	4	2.00	0.119
360	5.155	0.00595	6	2.45	0.151
480	5.055	0.00695	8	2.83	0.177
600	4.963	0.00787	10	3.16	0.200
900	4.772	0.00978	15	3.87	0.248
1200	4.622	0.01128	20	4.47	0.287
1500	4.490	0.0126	25	5.00	0.320
1800	4.378	0.01372	30	5.48	0.348
2700	4.125	0.01625	45	6.71	0.413
3600	3.951	0.01799	60	7.75	0.457
5400	3.735	0.02015	90	9.49	0.512
7200	3.620	0.0213	120	10.95	0.541
14400	3.380	0.0237	240	15.49	0.602
28800	3.200	0.0255	480	21.91	0.65
86400	3.045	0.02705	1440	37.95	0.69

Sample # 2, Test data # 3
Load = 1020 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	3.045	0	0	0.00	0.000
6	2.680	0.00365	0.1	0.32	0.093
15	2.520	0.00525	0.25	0.50	0.133
30	2.440	0.00605	0.5	0.71	0.154
45	2.350	0.00695	0.75	0.87	0.177
60	2.290	0.00755	1	1.00	0.192
90	2.180	0.00865	1.5	1.22	0.220
120	2.090	0.00955	2	1.41	0.243
240	1.770	0.01275	4	2.00	0.324
360	1.590	0.01455	6	2.45	0.370
480	1.405	0.0164	8	2.83	0.417
600	1.245	0.018	10	3.16	0.457
900	25.905	0.0214	15	3.87	0.544
1200	25.600	0.02445	20	4.47	0.621
1500	25.380	0.02665	25	5.00	0.677
1800	25.180	0.02865	30	5.48	0.728
2700	24.690	0.03355	45	6.71	0.852
3600	24.415	0.0363	60	7.75	0.922
5400	23.925	0.0412	90	9.49	1.046
7200	23.690	0.04355	120	10.95	1.106
14400	23.315	0.0473	240	15.49	1.201
28800	23.060	0.04985	480	21.91	1.27
86400	22.820	0.05225	1440	37.95	1.33

Consolidation Test: (Calculation)

Sample # 2, Test data # 4
Load = 2080 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	22.820	0	0	0.00	0.000
6	22.290	0.0053	0.1	0.32	0.135
15	22.150	0.0067	0.25	0.50	0.170
30	22.020	0.008	0.5	0.71	0.203
45	21.900	0.0092	0.75	0.87	0.234
60	21.810	0.0101	1	1.00	0.257
90	21.660	0.0116	1.5	1.22	0.295
120	21.520	0.013	2	1.41	0.330
240	21.120	0.017	4	2.00	0.432
360	20.800	0.0202	6	2.45	0.513
480	20.535	0.02285	8	2.83	0.580
600	20.300	0.0252	10	3.16	0.640
900	19.810	0.0301	15	3.87	0.765
1200	19.400	0.0342	20	4.47	0.869
1500	19.060	0.0376	25	5.00	0.955
1800	18.720	0.041	30	5.48	1.041
2700	17.996	0.04824	45	6.71	1.225
3600	17.425	0.05395	60	7.75	1.370
5400	16.720	0.061	90	9.49	1.549
7200	16.415	0.06405	120	10.95	1.627
14400	15.870	0.0695	240	15.49	1.765
28800	15.620	0.072	480	21.91	1.829
86400	15.200	0.0762	1440	37.95	1.935

Sample # 2, Test data # 5
Load = 4210 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	15.200	0	0	0.00	0.000
6	14.620	0.0058	0.1	0.32	0.147
15	14.440	0.0076	0.25	0.50	0.193
30	14.300	0.009	0.5	0.71	0.229
45	14.180	0.0102	0.75	0.87	0.259
60	14.080	0.0112	1	1.00	0.284
90	13.900	0.013	1.5	1.22	0.330
120	13.750	0.0145	2	1.41	0.368
240	13.260	0.0194	4	2.00	0.493
360	12.940	0.0226	6	2.45	0.574
480	12.635	0.02565	8	2.83	0.652
600	12.370	0.0283	10	3.16	0.719
900	11.825	0.03375	15	3.87	0.857
1200	11.365	0.03835	20	4.47	0.974
1500	10.930	0.0427	25	5.00	1.085
1800	10.670	0.0453	30	5.48	1.151
2700	9.805	0.05395	45	6.71	1.370
3600	9.220	0.0598	60	7.75	1.519
5400	8.425	0.06775	90	9.49	1.721
7200	8.000	0.072	120	10.95	1.829
14400	7.400	0.078	240	15.49	1.981
28800	7.080	0.0812	480	21.91	2.06
86400	6.920	0.0828	1440	37.95	2.10

Consolidation Test: (Calculation)

Sample # 2, Test data # 4

Load = 2080 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	22.820	0	0	0.00	0.000
6	22.290	0.0053	0.1	0.32	0.135
15	22.150	0.0067	0.25	0.50	0.170
30	22.020	0.008	0.5	0.71	0.203
45	21.900	0.0092	0.75	0.87	0.234
60	21.810	0.0101	1	1.00	0.257
90	21.660	0.0116	1.5	1.22	0.295
120	21.520	0.013	2	1.41	0.330
240	21.120	0.017	4	2.00	0.432
360	20.800	0.0202	6	2.45	0.513
480	20.535	0.02285	8	2.83	0.580
600	20.300	0.0252	10	3.16	0.640
900	19.810	0.0301	15	3.87	0.765
1200	19.400	0.0342	20	4.47	0.869
1500	19.060	0.0376	25	5.00	0.955
1800	18.720	0.041	30	5.48	1.041
2700	17.996	0.04824	45	6.71	1.225
3600	17.425	0.05395	60	7.75	1.370
5400	16.720	0.061	90	9.49	1.549
7200	16.415	0.06405	120	10.95	1.627
14400	15.870	0.0695	240	15.49	1.765
28800	15.620	0.072	480	21.91	1.829
86400	15.200	0.0762	1440	37.95	1.935

Sample # 2, Test data # 6
Load = 8465 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	6.920	0	0	0.00	0.000
6	6.450	0.0047	0.1	0.32	0.119
15	6.270	0.0065	0.25	0.50	0.165
30	6.120	0.008	0.5	0.71	0.203
45	6.010	0.0091	0.75	0.87	0.231
60	5.920	0.01	1	1.00	0.254
90	5.750	0.0117	1.5	1.22	0.297
120	5.620	0.013	2	1.41	0.330
240	5.205	0.01715	4	2.00	0.436
360	4.875	0.02045	6	2.45	0.519
480	4.605	0.02315	8	2.83	0.588
600	4.390	0.0253	10	3.16	0.643
900	3.860	0.0306	15	3.87	0.777
1200	3.430	0.0349	20	4.47	0.886
1500	3.020	0.039	25	5.00	0.991
1800	2.750	0.0417	30	5.48	1.059
2700	1.965	0.04955	45	6.71	1.259
3600	1.375	0.05545	60	7.75	1.408
5400	25.450	0.0647	90	9.49	1.643
7200	25.050	0.0687	120	10.95	1.745
14400	24.935	0.06985	240	15.49	1.774
28800	24.915	0.07005	480	21.91	1.78
86400	24.900	0.0702	1440	37.95	1.78

Consolidation Test: (Calculation)

Sample # 2, Test data # 4
Load = 2080 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	22.820	0	0	0.00	0.000
6	22.290	0.0053	0.1	0.32	0.135
15	22.150	0.0067	0.25	0.50	0.170
30	22.020	0.008	0.5	0.71	0.203
45	21.900	0.0092	0.75	0.87	0.234
60	21.810	0.0101	1	1.00	0.257
90	21.660	0.0116	1.5	1.22	0.295
120	21.520	0.013	2	1.41	0.330
240	21.120	0.017	4	2.00	0.432
360	20.800	0.0202	6	2.45	0.513
480	20.535	0.02285	8	2.83	0.580
600	20.300	0.0252	10	3.16	0.640
900	19.810	0.0301	15	3.87	0.765
1200	19.400	0.0342	20	4.47	0.869
1500	19.060	0.0376	25	5.00	0.955
1800	18.720	0.041	30	5.48	1.041
2700	17.996	0.04824	45	6.71	1.225
3600	17.425	0.05395	60	7.75	1.370
5400	16.720	0.061	90	9.49	1.549
7200	16.415	0.06405	120	10.95	1.627
14400	15.870	0.0695	240	15.49	1.765
28800	15.620	0.072	480	21.91	1.829
86400	15.200	0.0762	1440	37.95	1.935

Sample # 2, Test data # 7
Load = 17020 psf

Time (sec)	**Reading (dial gauge)**	**Settlement (in)**	**Time (min)**	$\sqrt{Time}$ **(min$^{1/2}$)**	**Settlement (mm)**
0	6.000	0	0	0.00	0.000
6	5.100	0.009	0.1	0.32	0.229
15	4.810	0.0119	0.25	0.50	0.302
30	4.620	0.0138	0.5	0.71	0.351
45	4.560	0.0144	0.75	0.87	0.366
60	4.470	0.0153	1	1.00	0.389
90	4.350	0.0165	1.5	1.22	0.419
120	4.250	0.0175	2	1.41	0.445
240	3.905	0.02095	4	2.00	0.532
360	3.640	0.0236	6	2.45	0.599
480	3.430	0.0257	8	2.83	0.653
600	3.235	0.02765	10	3.16	0.702
900	2.820	0.0318	15	3.87	0.808
1200	2.540	0.0346	20	4.47	0.879
1500	2.250	0.0375	25	5.00	0.953
1800	1.985	0.04015	30	5.48	1.020
2700	1.330	0.0467	45	6.71	1.186
3600	25.890	0.0511	60	7.75	1.298
5400	25.190	0.0581	90	9.49	1.476
7200	24.820	0.0618	120	10.95	1.570
14400	24.240	0.0676	240	15.49	1.717
28800	24.020	0.0698	480	21.91	1.77
86400	23.820	0.0718	1440	37.95	1.82

Consolidation Test:

Sample # 3, Test data # 1
Load = 215 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	10.490	0	0	0.00	0.000
6	10.410	0.0008	0.1	0.32	0.020
15	10.350	0.0014	0.25	0.50	0.036
30	10.295	0.00195	0.5	0.71	0.050
45	10.262	0.00228	0.75	0.87	0.058
60	10.225	0.00265	1	1.00	0.067
90	10.175	0.00315	1.5	1.22	0.080
120	10.135	0.00355	2	1.41	0.090
240	10.010	0.0048	4	2.00	0.122
360	9.920	0.0057	6	2.45	0.145
480	9.853	0.00637	8	2.83	0.162
600	9.797	0.00693	10	3.16	0.176
900	9.700	0.0079	15	3.87	0.201
1200	9.620	0.0087	20	4.47	0.221
1500	9.565	0.00925	25	5.00	0.235
1800	9.507	0.00983	30	5.48	0.250
2700	9.407	0.01083	45	6.71	0.275
3600	9.343	0.01147	60	7.75	0.291
5400	9.270	0.0122	90	9.49	0.310
7200	9.243	0.01247	120	10.95	0.317
14400	9.190	0.013	240	15.49	0.330
28800	9.140	0.0135	480	21.91	0.34
86400	9.100	0.0139	1440	37.95	0.35

Consolidation Test: (Calculation)

Sample # 3, Test data # 2

Load = 430 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ (min$^{1/2}$)	Settlement (mm)
0	9.100	0	0	0.00	0.000
6	9.010	0.0009	0.1	0.32	0.023
15	8.980	0.0012	0.25	0.50	0.030
30	8.950	0.0015	0.5	0.71	0.038
45	8.925	0.00175	0.75	0.87	0.044
60	8.902	0.00198	1	1.00	0.050
90	8.875	0.00225	1.5	1.22	0.057
120	8.845	0.00255	2	1.41	0.065
240	8.755	0.00345	4	2.00	0.088
360	8.665	0.00435	6	2.45	0.110
480	8.610	0.0049	8	2.83	0.124
600	8.575	0.00525	10	3.16	0.133
900	8.462	0.00638	15	3.87	0.162
1200	8.367	0.00733	20	4.47	0.186
1500	8.292	0.00808	25	5.00	0.205
1800	8.225	0.00875	30	5.48	0.222
2700	8.075	0.01025	45	6.71	0.260
3600	7.985	0.01115	60	7.75	0.283
5400	7.860	0.0124	90	9.49	0.315
7200	7.795	0.01305	120	10.95	0.331
14400	7.700	0.014	240	15.49	0.356
28800	7.620	0.0148	480	21.91	0.376
86400	7.550	0.0155	1440	37.95	0.394

Sample # 3, Test data # 3
Load = 950 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	7.550	0	0	0.00	0.000
6	7.310	0.0024	0.1	0.32	0.061
15	7.250	0.003	0.25	0.50	0.076
30	7.150	0.004	0.5	0.71	0.102
45	7.085	0.00465	0.75	0.87	0.118
60	7.010	0.0054	1	1.00	0.137
90	6.880	0.0067	1.5	1.22	0.170
120	6.800	0.0075	2	1.41	0.191
240	6.520	0.0103	4	2.00	0.262
360	6.330	0.0122	6	2.45	0.310
480	6.180	0.0137	8	2.83	0.348
600	6.060	0.0149	10	3.16	0.378
900	5.730	0.0182	15	3.87	0.462
1200	5.500	0.0205	20	4.47	0.521
1500	5.325	0.02225	25	5.00	0.565
1800	5.120	0.0243	30	5.48	0.617
2700	4.635	0.02915	45	6.71	0.740
3600	4.380	0.0317	60	7.75	0.805
5400	4.150	0.034	90	9.49	0.864
7200	4.020	0.0353	120	10.95	0.897
14400	3.800	0.0375	240	15.49	0.953
28800	3.630	0.0392	480	21.91	1.00
86400	3.540	0.0401	1440	37.95	1.02

Consolidation Test: (Calculation)

Sample # 3, Test data # 4
Load = 2010 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	3.540	0	0	0.00	0.000
6	3.200	0.0034	0.1	0.32	0.086
15	3.100	0.0044	0.25	0.50	0.112
30	2.970	0.0057	0.5	0.71	0.145
45	2.880	0.0066	0.75	0.87	0.168
60	2.770	0.0077	1	1.00	0.196
90	2.615	0.00925	1.5	1.22	0.235
120	2.490	0.0105	2	1.41	0.267
240	2.100	0.0144	4	2.00	0.366
360	1.825	0.01715	6	2.45	0.436
480	1.560	0.0198	8	2.83	0.503
600	1.335	0.02205	10	3.16	0.560
900	25.935	0.02605	15	3.87	0.662
1200	25.615	0.02925	20	4.47	0.743
1500	25.360	0.0318	25	5.00	0.808
1800	25.150	0.0339	30	5.48	0.861
2700	24.630	0.0391	45	6.71	0.993
3600	24.250	0.0429	60	7.75	1.090
5400	23.880	0.0466	90	9.49	1.184
7200	23.690	0.0485	120	10.95	1.232
14400	23.445	0.05095	240	15.49	1.294
28800	23.300	0.0524	480	21.91	1.331
86400	23.150	0.0539	1440	37.95	1.369

Sample # 3, Test data # 5
Load = 4140 psf

Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	23.150	0	0	0.00	0.000
6	22.780	0.0037	0.1	0.32	0.094
15	22.640	0.0051	0.25	0.50	0.130
30	22.500	0.0065	0.5	0.71	0.165
45	22.400	0.0075	0.75	0.87	0.191
60	22.305	0.00845	1	1.00	0.215
90	22.140	0.0101	1.5	1.22	0.257
120	22.000	0.0115	2	1.41	0.292
240	21.580	0.0157	4	2.00	0.399
360	21.220	0.0193	6	2.45	0.490
480	20.970	0.0218	8	2.83	0.554
600	20.650	0.025	10	3.16	0.635
900	20.100	0.0305	15	3.87	0.775
1200	19.690	0.0346	20	4.47	0.879
1500	19.345	0.03805	25	5.00	0.966
1800	19.060	0.0409	30	5.48	1.039
2700	18.435	0.04715	45	6.71	1.198
3600	18.062	0.05088	60	7.75	1.292
5400	17.690	0.0546	90	9.49	1.387
7200	17.535	0.05615	120	10.95	1.426
14400	17.380	0.0577	240	15.49	1.466
28800	17.230	0.0592	480	21.91	1.50
86400	17.110	0.0604	1440	37.95	1.53

Sample # 3, Test data # 6
Load = 8395 psf

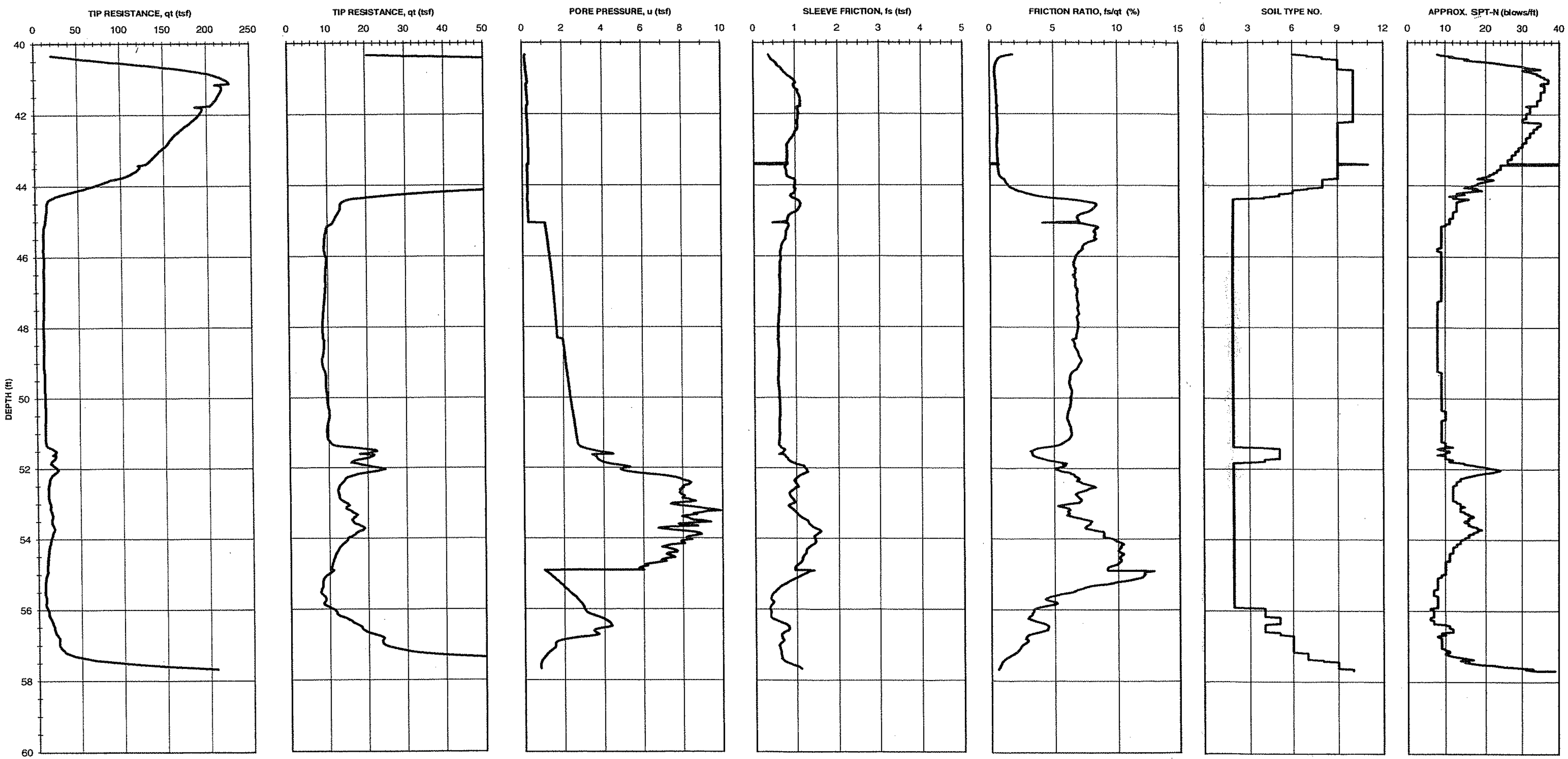
Time (sec)	Reading (dial gauge)	Settlement (in)	Time (min)	$\sqrt{Time}$ ($min^{1/2}$)	Settlement (mm)
0	17.110	0	0	0.00	0.000
6	16.700	0.0041	0.1	0.32	0.104
15	16.570	0.0054	0.25	0.50	0.137
30	16.400	0.0071	0.5	0.71	0.180
45	16.285	0.00825	0.75	0.87	0.210
60	16.180	0.0093	1	1.00	0.236
90	15.990	0.0112	1.5	1.22	0.284
120	15.835	0.01275	2	1.41	0.324
240	15.355	0.01755	4	2.00	0.446
360	15.000	0.0211	6	2.45	0.536
480	14.690	0.0242	8	2.83	0.615
600	14.430	0.0268	10	3.16	0.681
900	13.910	0.032	15	3.87	0.813
1200	13.480	0.0363	20	4.47	0.922
1500	13.200	0.0391	25	5.00	0.993
1800	12.980	0.0413	30	5.48	1.049
2700	12.450	0.0466	45	6.71	1.184
3600	12.205	0.04905	60	7.75	1.246
5400	11.980	0.0513	90	9.49	1.303
7200	11.880	0.0523	120	10.95	1.328
14400	11.750	0.0536	240	15.49	1.361
28800	11.690	0.0542	480	21.91	1.38
86400	11.610	0.055	1440	37.95	1.40

Sample # 3, Test data # 7
Load = 16950 psf

Time (sec)	**Reading (dial gauge)**	**Settlement (in)**	**Time (min)**	**$\sqrt{Time}$ ($min^{1/2}$)**	**Settlement (mm)**
0	11.610	0	0	0.00	0.000
6	11.200	0.0041	0.1	0.32	0.104
15	11.070	0.0054	0.25	0.50	0.137
30	10.900	0.0071	0.5	0.71	0.180
45	10.780	0.0083	0.75	0.87	0.211
60	10.680	0.0093	1	1.00	0.236
90	10.490	0.0112	1.5	1.22	0.284
120	10.330	0.0128	2	1.41	0.325
240	9.840	0.0177	4	2.00	0.450
360	9.450	0.0216	6	2.45	0.549
480	9.175	0.02435	8	2.83	0.618
600	8.910	0.027	10	3.16	0.686
900	8.403	0.03207	15	3.87	0.815
1200	8.025	0.03585	20	4.47	0.911
1500	7.740	0.0387	25	5.00	0.983
1800	7.570	0.0404	30	5.48	1.026
2700	7.120	0.0449	45	6.71	1.140
3600	6.940	0.0467	60	7.75	1.186
5400	6.750	0.0486	90	9.49	1.234
7200	6.690	0.0492	120	10.95	1.250
14400	6.580	0.0503	240	15.49	1.278
28800	6.510	0.051	480	21.91	1.30
86400	6.400	0.0521	1440	37.95	1.32

APPENDIX B

ARDAMAN AND ASSOCIATES, INC. REPORT



Soil I.D. #	Soil Description	UCS
1	Sensitive Fine Grained	OH/CH
2	Organic Material	OH
3	Clay	CH
4	Silty Clay to Clay	CL/MH
5	Clayey Silt to Silty Clay	MH/CL
6	Silty Sand to Sandy Silt	SC

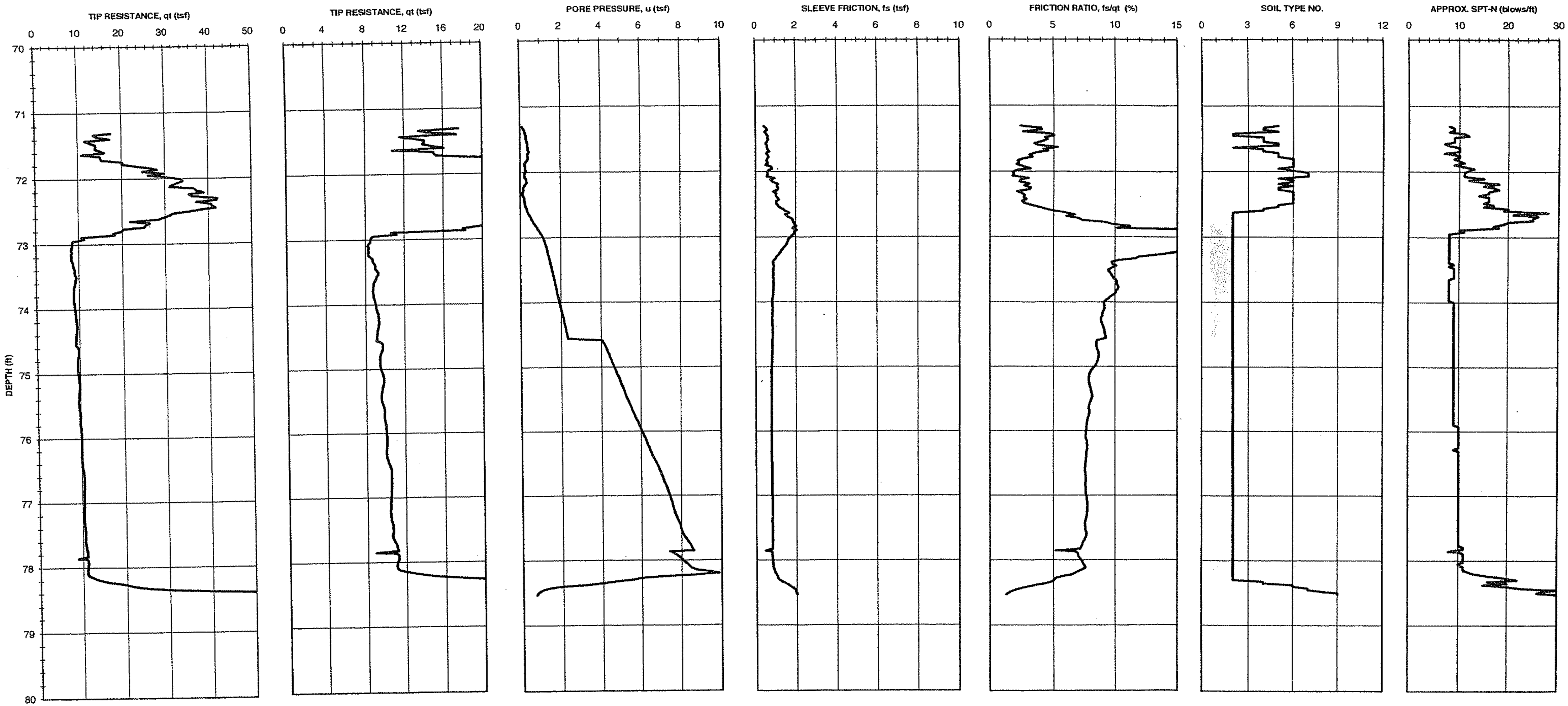
Soil I.D. #	Soil Description	UCS
7	Sand to Sandy Silt	SP/SC
8	Sand to Silty Sand	SP
9	Sand	SP/SW
10	Gravelly Sand to Sand	SP/GW
11	Very Stiff Fine Grained	OC Clay
12	Sand to Clayey Sand	Cemented

Soil Classification by Robertson etal., 1986

Date Tested= 10/02/01

Ardaman & Associates, Inc.
Piezocone Test Results

Project No.: SCS SE LANDFILL
File No.: 01-122
Sounding No.: PC-1B



Soil I.D. #	Soil Description	UCS
1	Sensitive Fine Grained	OH/CH
2	Organic Material	OH
3	Clay	CH
4	Silty Clay to Clay	CL/MH
5	Clayey Silt to Silty Clay	MH/CL
6	Silty Sand to Sandy Silt	SC

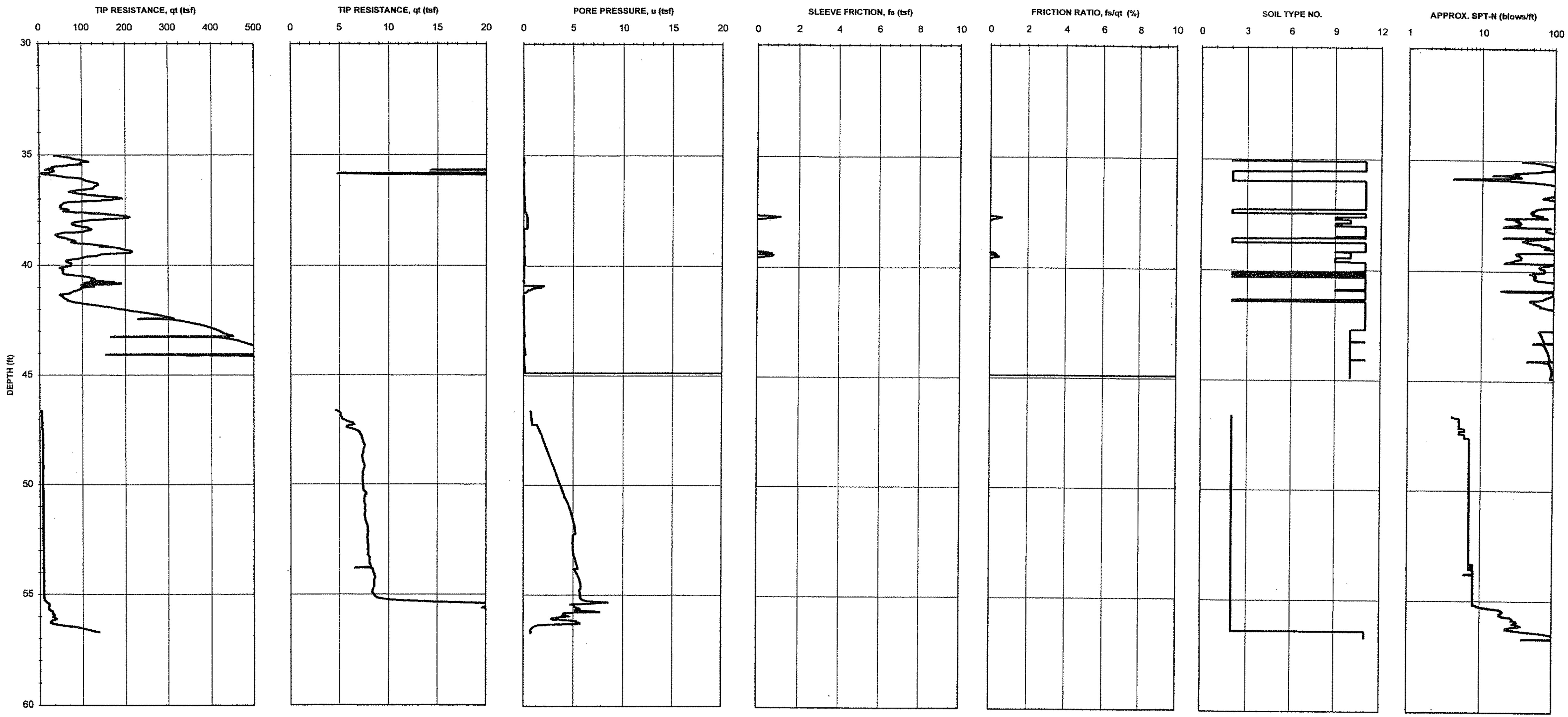
Soil I.D. #	Soil Description	UCS
7	Sand to Sandy Silt	SP/SC
8	Sand to Silty Sand	SP
9	Sand	SP/SW
10	Gravelly Sand to Sand	SP/GW
11	Very Stiff Fine Grained	OC Clay
12	Sand to Clayey Sand	Cemented

Soil Classification by Robertson etal., 1986

Date Tested= 09/25/01

Ardaman & Associates, Inc.
Piezocone Test Results

Project No. : SCS SE LANDFILL
File No. : 01-122
Sounding No.: PC-2A



Soil I.D. #	Soil Description	UCS
1	Sensitive Fine Grained	OH/CH
2	Organic Material	OH
3	Clay	CH
4	Silty Clay to Clay	CL/MH
5	Clayey Silt to Silty Clay	MH/CL
6	Silty Sand to Sandy Silt	SC

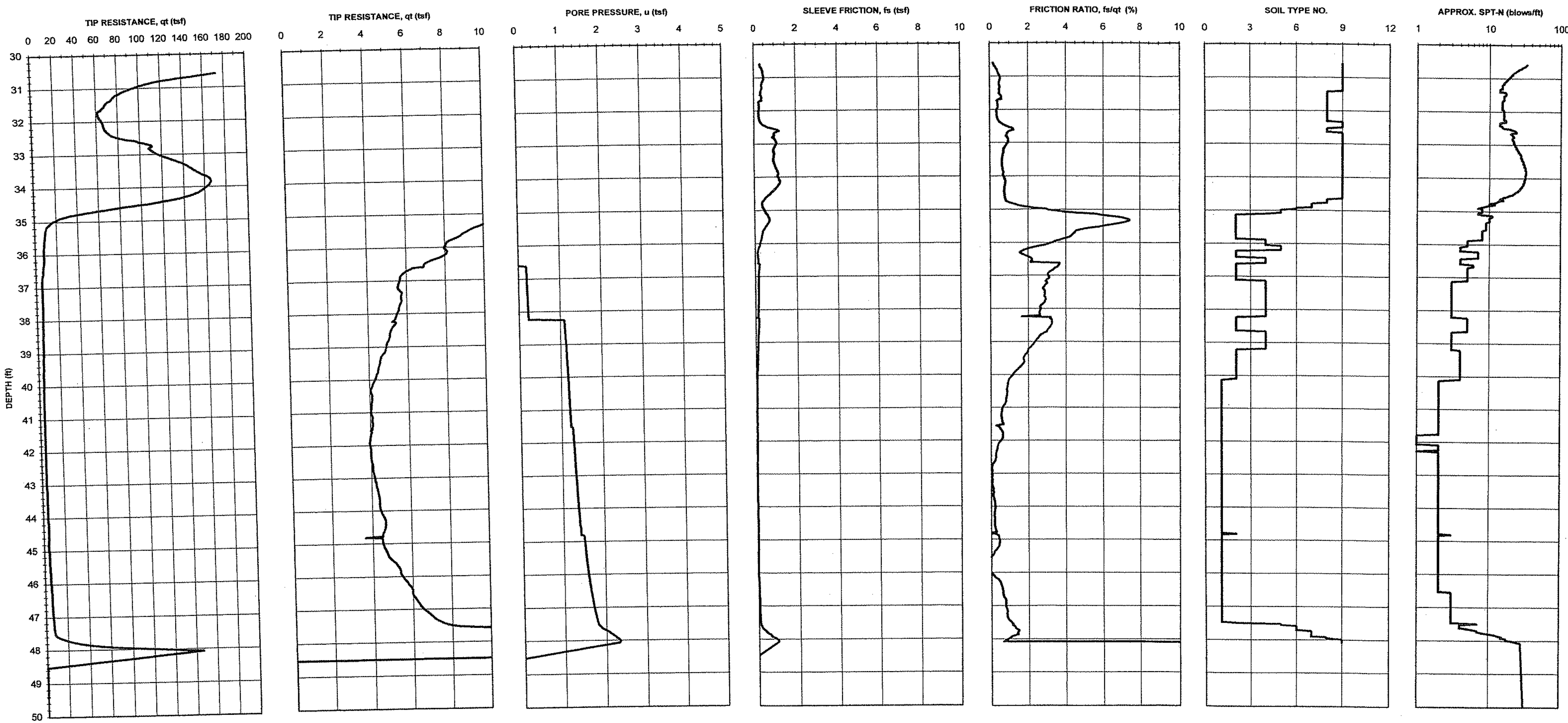
Soil I.D. #	Soil Description	UCS
7	Sand to Sandy Silt	SP/SC
8	Sand to Silty Sand	SP
9	Sand	SP/SW
10	Gravelly Sand to Sand	SP/GW
11	Very Stiff Fine Grained	OC Clay
12	Sand to Clayey Sand	Cemented

Soil Classification by Robertson etal., 1986

Date Tested= 09/13/01

Ardaman & Associates, Inc.
Piezocone Test Results

Project No. : SCS SE LANDFILL
File No. : 01-122
Sounding No.: PC-3A



Soil I.D. #	Soil Description	UCS
1	Sensitive Fine Grained	OH/CH
2	Organic Material	OH
3	Clay	CH
4	Silty Clay to Clay	CL/MH
5	Clayey Silt to Silty Clay	MH/CL
6	Silty Sand to Sandy Silt	SC

Soil I.D. #	Soil Description	UCS
7	Sand to Sandy Silt	SP/SC
8	Sand to Silty Sand	SP
9	Sand	SP/SW
10	Gravelly Sand to Sand	SP/GW
11	Very Stiff Fine Grained	OC Clay
12	Sand to Clayey Sand	Cemented

Soil Classification by Robertson etal., 1986

Date Tested= 09/12/01

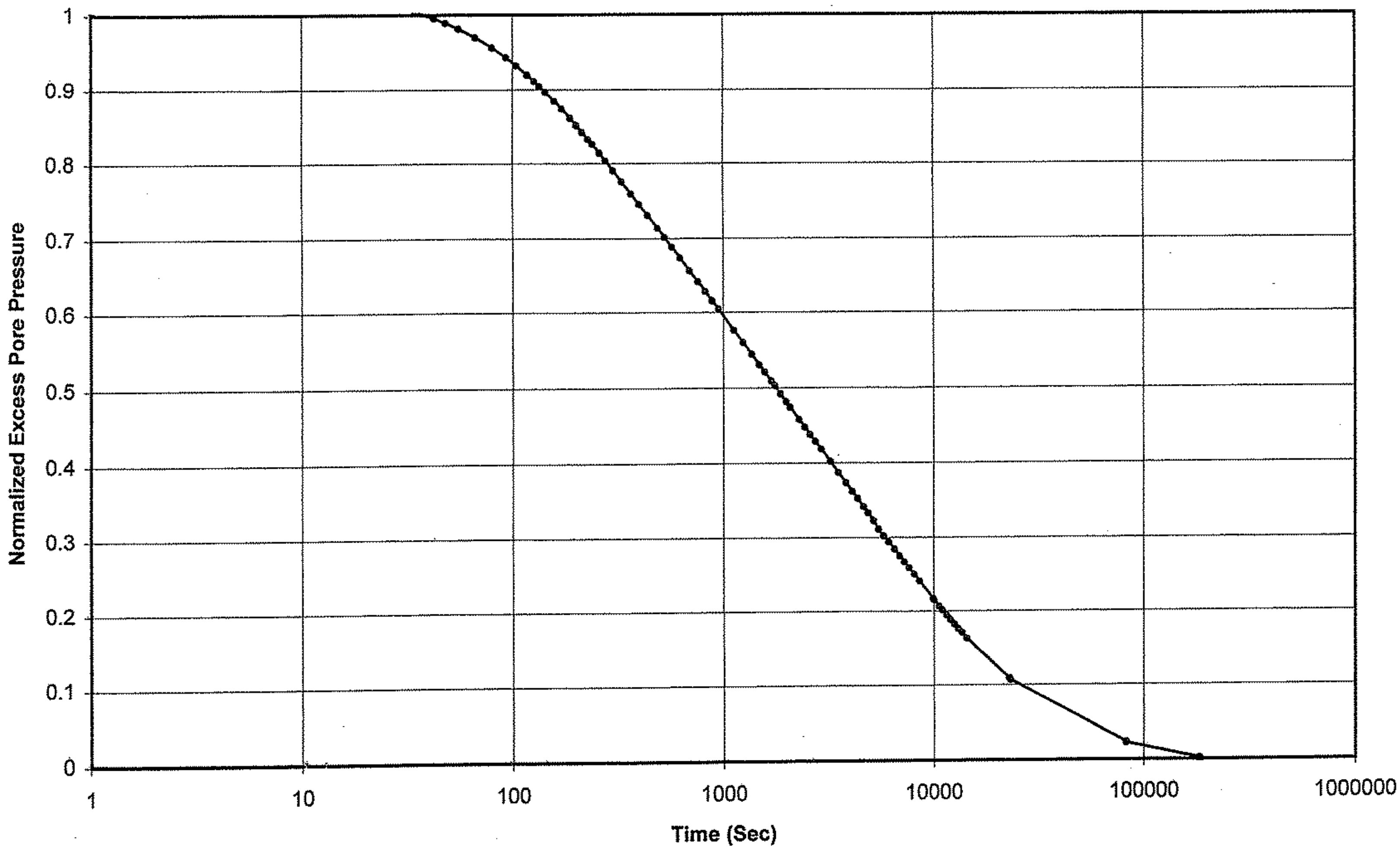
Ardaman & Associates, Inc.

Piezocone Test Results

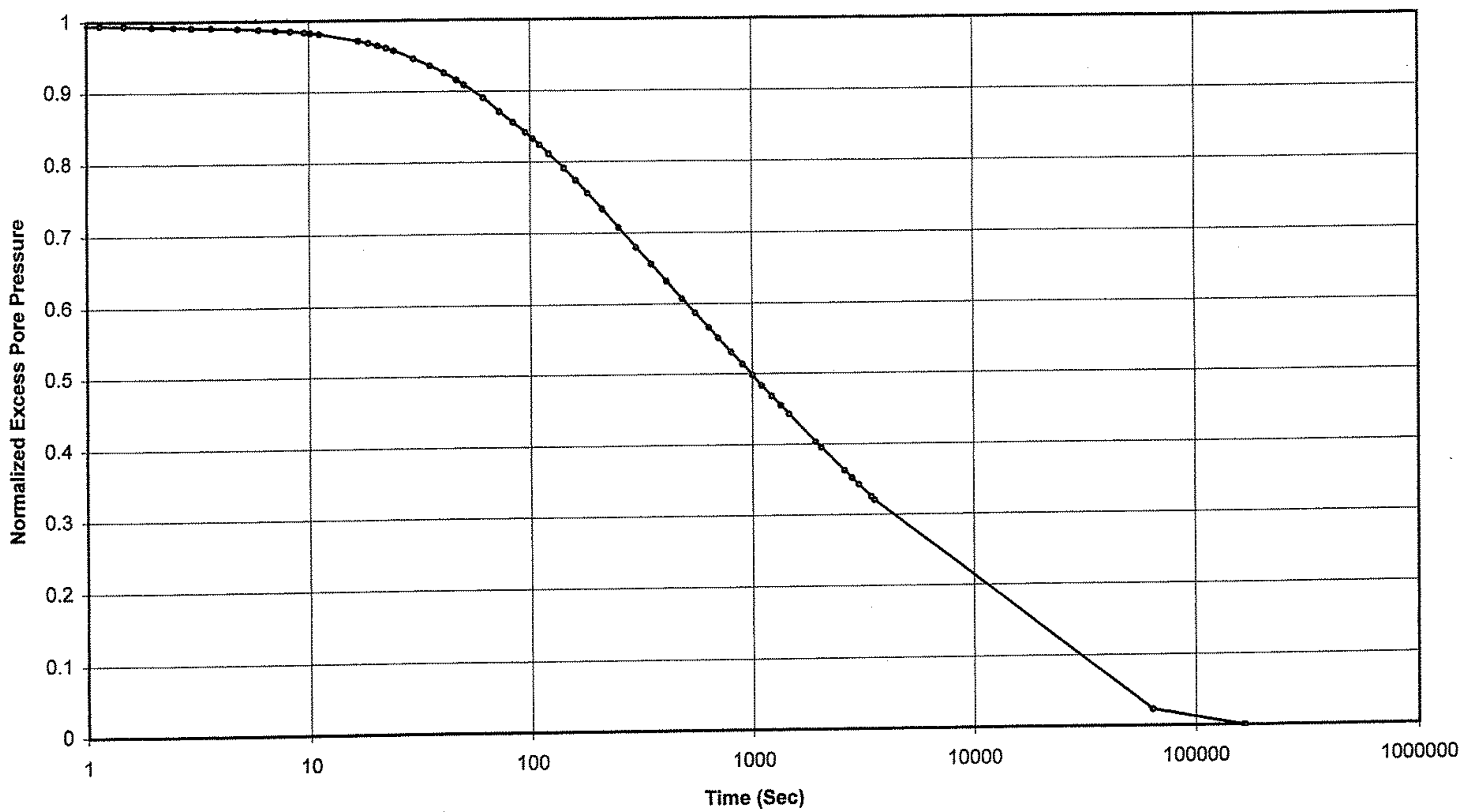
Project No. : SCS SE LANDFILL

File No. : 01-122

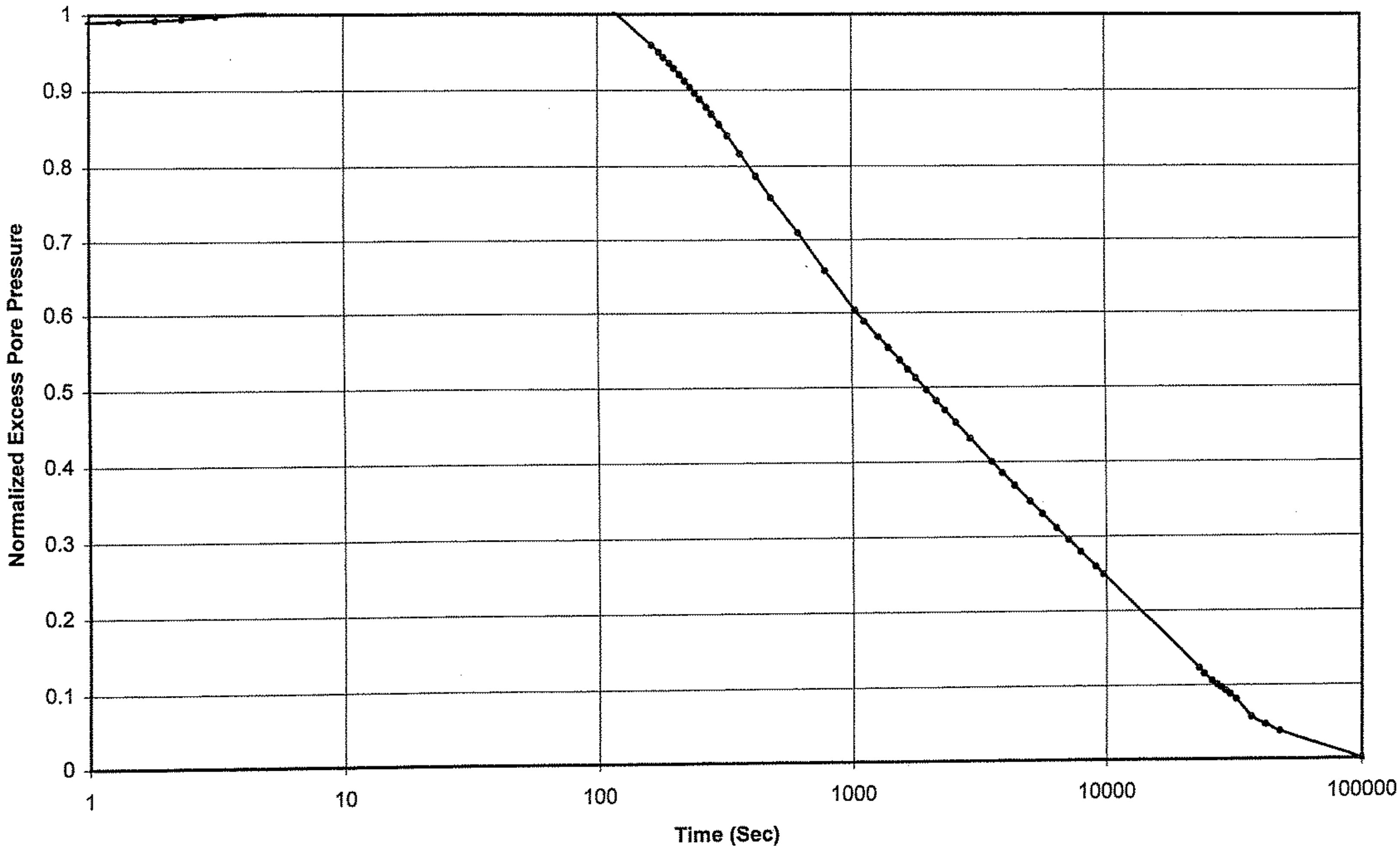
Sounding No.: PC-6A



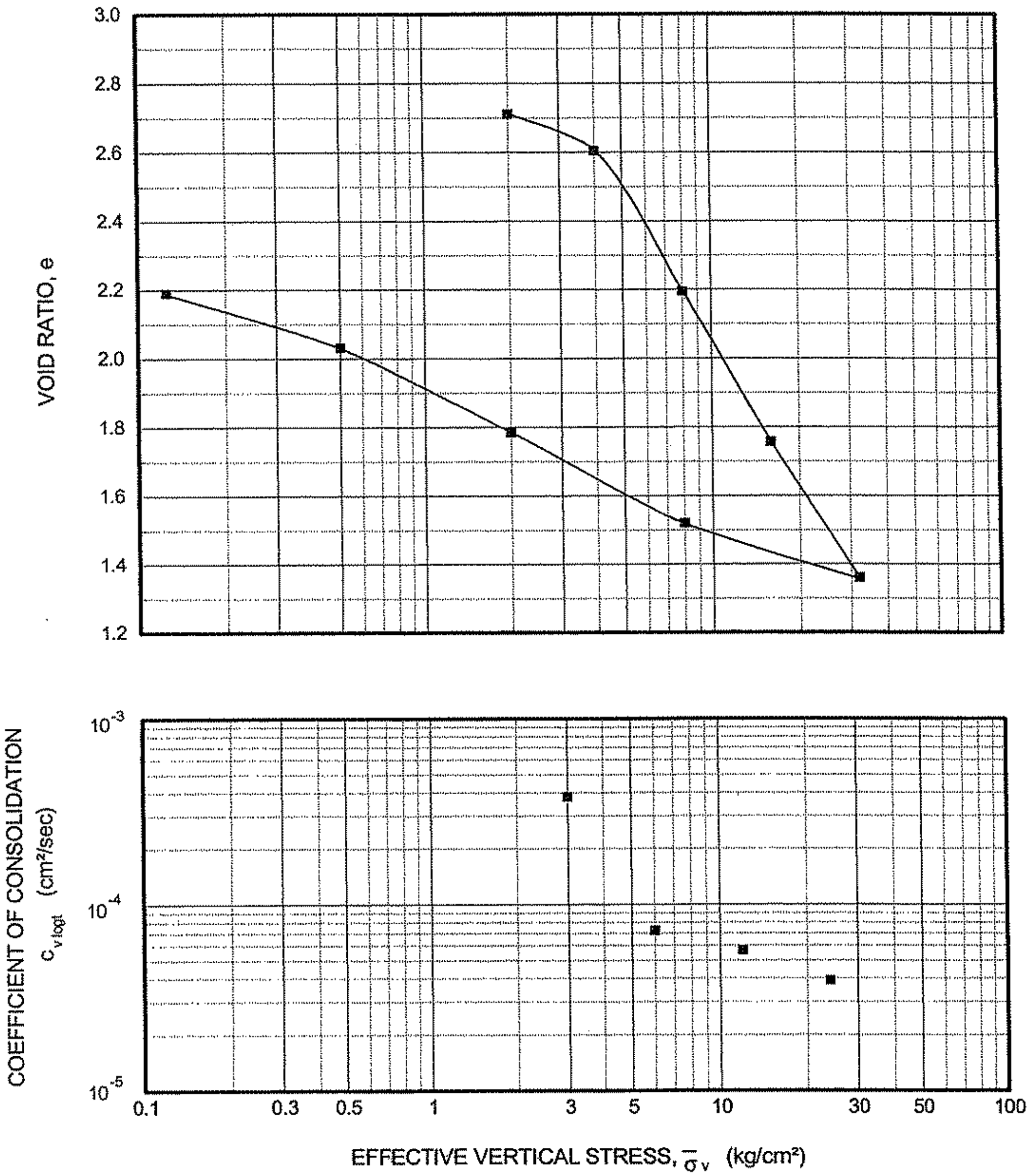
DECAY OF EXCESS PORE PRESSURE AT 73 FEET BELOW GROUND IN PC-2A



DECAY OF EXCESS PORE PRESSURE AT 75 FEET BELOW GROUND IN PC-2A



DECAY OF EXCESS PORE PRESSURE AT 77 FEET BELOW GROUND IN PC-2A



SAMPLE DATA	
BORING NUMBER:	PC2B
SAMPLE NUMBER:	US-2
DEPTH (FEET):	75.5 - 77.5
DESCRIPTION:	GRAY STIFF CLAY

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	98.8	85.3
DRY DENSITY (lb/ft³):	46.0	51.3
VOID RATIO:	2.73	2.35
SATURATION (%):	100	100

INDEX PROPERTIES	
LIQUID LIMIT (%):	179
PLASTIC LIMIT (%):	44
PLASTICITY INDEX (%):	135
% PASSING NO. 200:	100
SPECIFIC GRAVITY:	2.75 (Assumed)

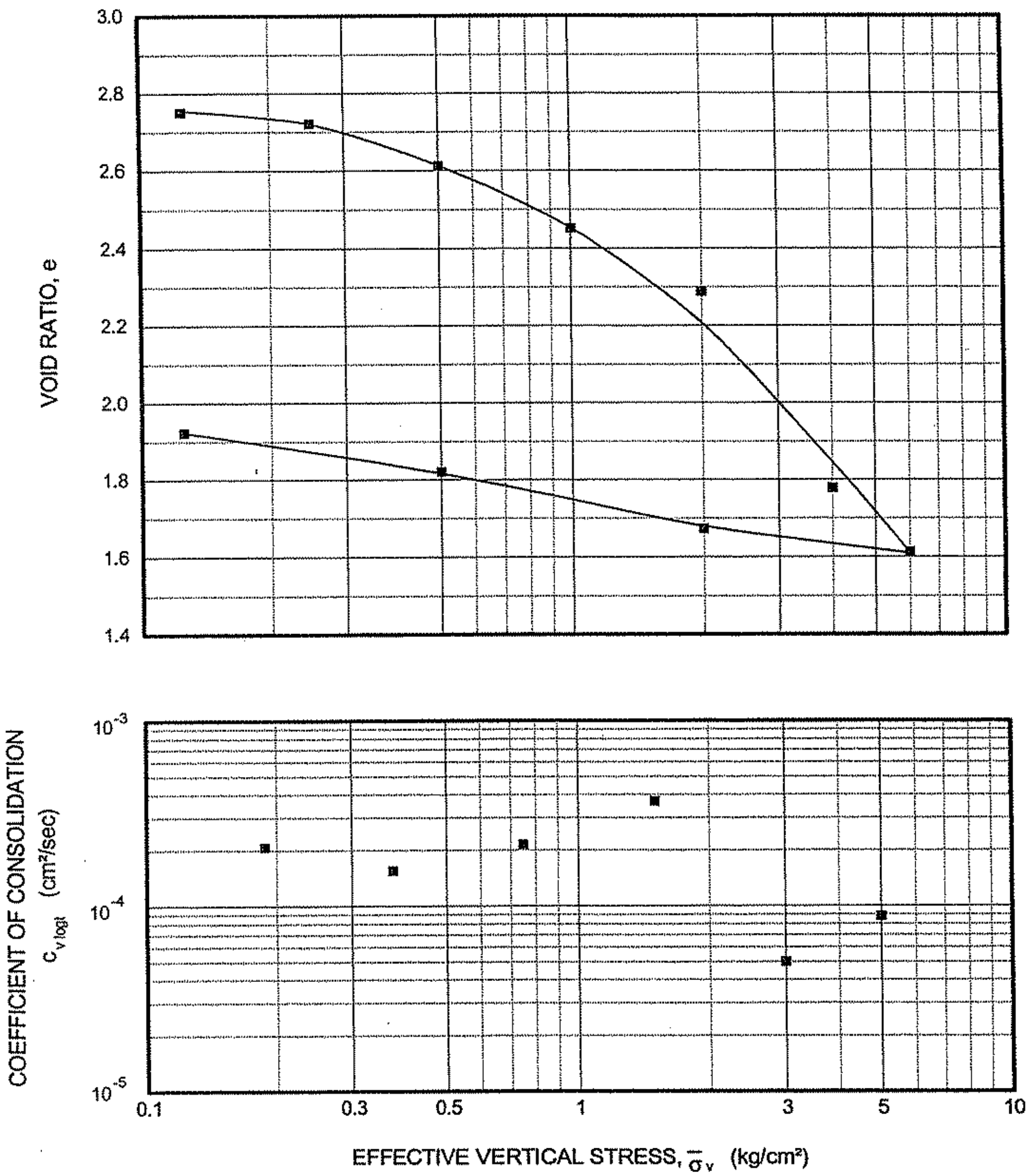
CONSOLIDATION TEST RESULTS FOR CLAY SAMPLE FROM PC-2A

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

SOUTHEAST LANDFILL
OPERATION PERMIT RENEWAL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: HHC	CHECKED BY: HHC	DATE: 11/14/01
FILE NO. 01-122	APPROVED BY:	FIGURE :

C:\PROJECT\Landfill\01-122\FL\Consol.pre



SAMPLE DATA

BORING NUMBER:	PLGE
SAMPLE NUMBER:	US-1
DEPTH (FEET):	38.0 - 40.0
DESCRIPTION:	LIGHT-GRAY SOFT CLAY

SPECIMEN CONDITIONS

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	101.7	75.3
DRY DENSITY (lb/ft³):	45.3	55.9
VOID RATIO:	2.79	2.07
SATURATION (%):	100	100

INDEX PROPERTIES

LIQUID LIMIT (%):	149
PLASTIC LIMIT (%):	41
PLASTICITY INDEX (%):	108
% PASSING NO. 200:	100
SPECIFIC GRAVITY:	2.75

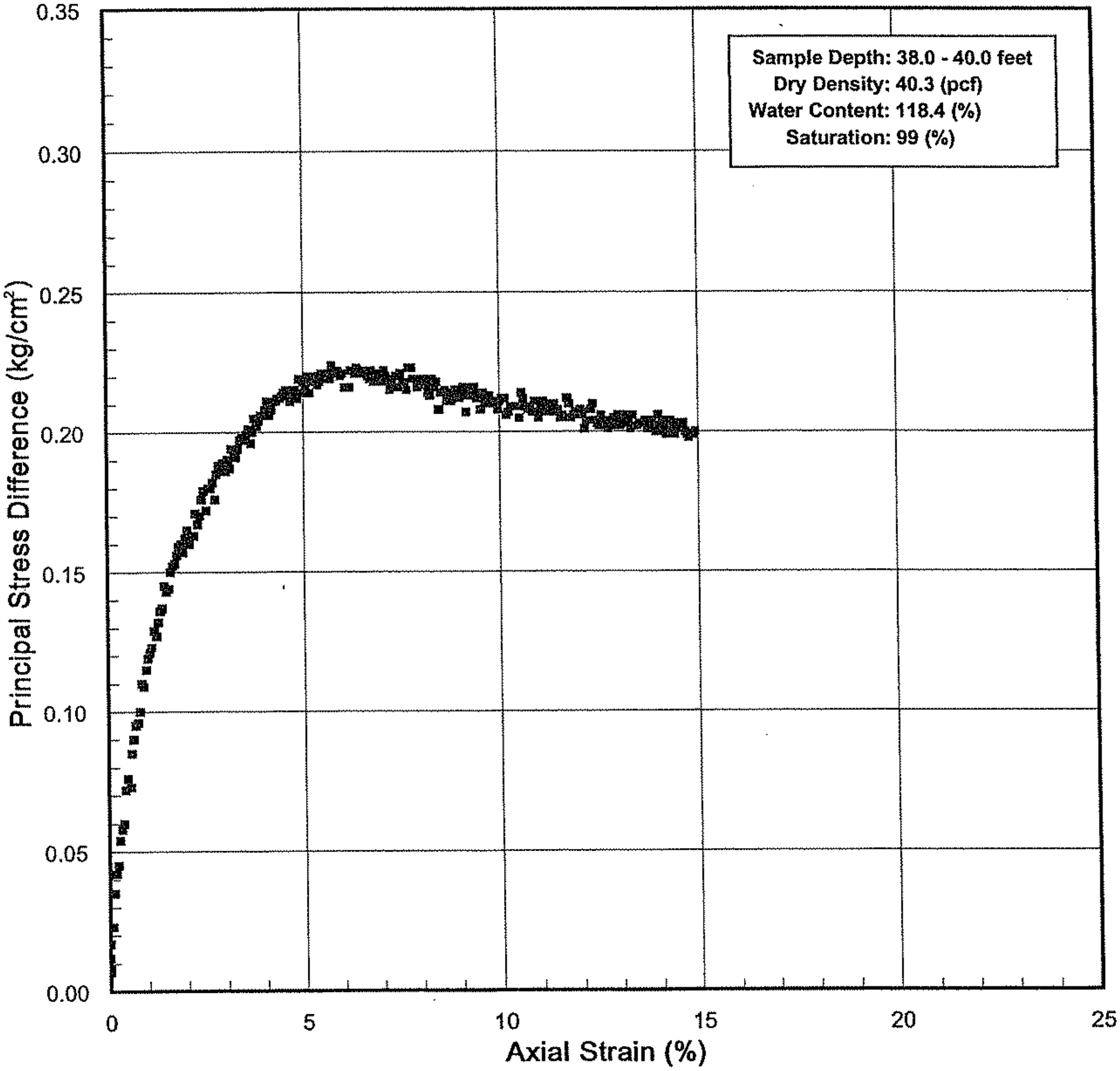
CONSOLIDATION TEST RESULTS FOR CLAY SAMPLE FROM PC-6A

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

**SOUTHEAST LANDFILL
OPERATION PERMIT RENEWAL
HILLSBOROUGH COUNTY, FLORIDA**

DRAWN BY: HHC	CHECKED BY: HHC	DATE: 11/14/01
FILE NO. 01-122	APPROVED BY:	FIGURE :

C:\PROJECT\Landfill\01-122\FL\Consol.pre



UNCONFINED COMPRESSION TEST FOR CLAY SAMPLE FROM PC-6A

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

SOUTHEAST LANDFILL
OPERATION PERMIT RENEWAL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: HHC	CHECKED BY: HHC	DATE: 11/14/01
FILE NO. 01-122	APPROVED BY:	FIGURE :

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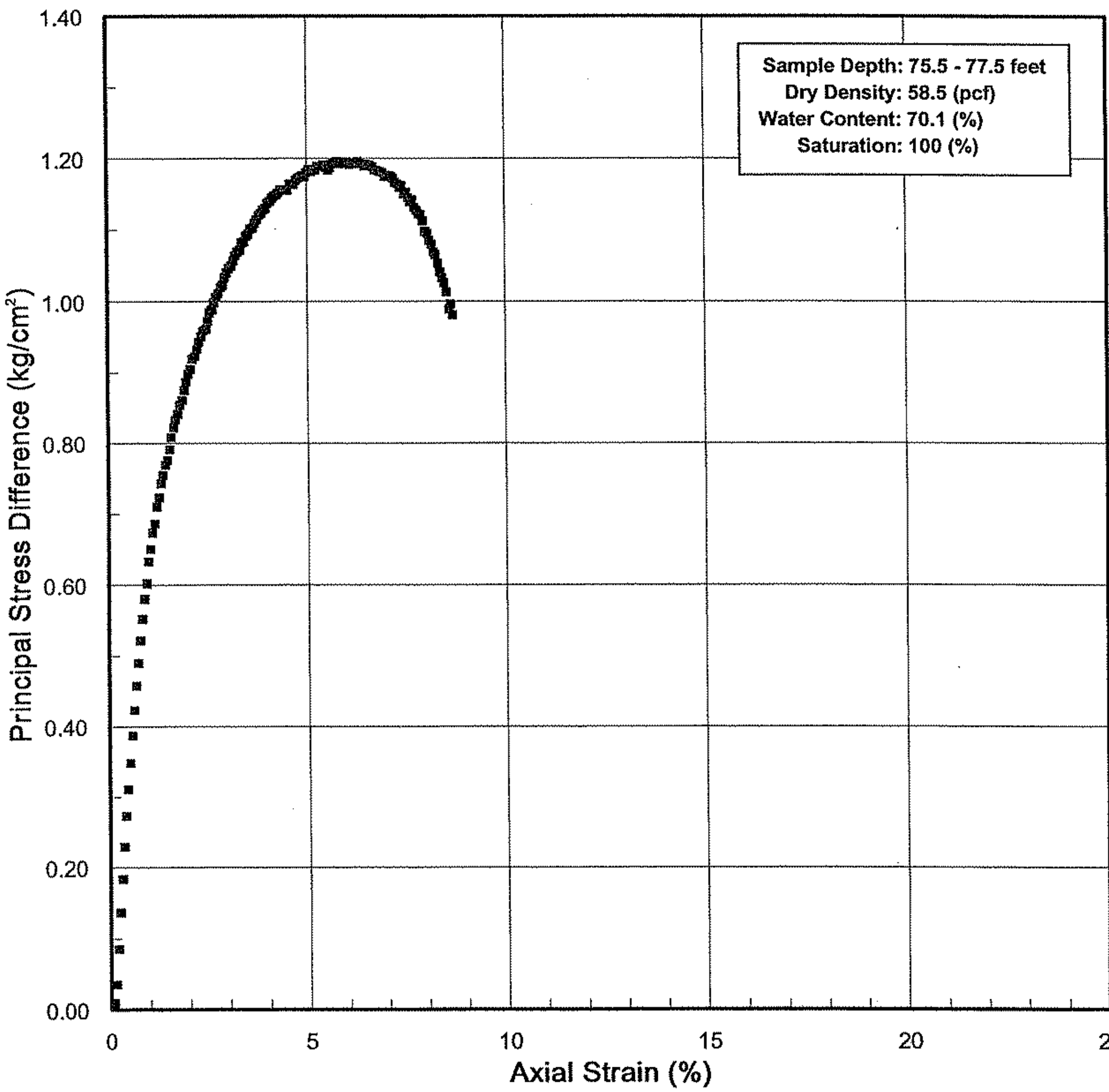

UNCONFINED COMPRESSION TEST FOR CLAY SAMPLE FROM PC-2A

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

SOUTHEAST LANDFILL
OPERATION PERMIT RENEWAL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: HHC	CHECKED BY: HHC	DATE: 11/14/01
FILE NO. 01-122	APPROVED BY:	FIGURE :

C:\PROJECT\Landfill\01-122\FL1\UComp.pre

TABLE B-1. BORING LOG

Location	Existing Ground Surface Elevation (ft)	Depth to Top of Clay (ft)	Depth to Bottom of Clay (ft)	Top Elevation of Clay (ft)	Bottom Elevation of Clay (ft)	Thickness of Clay (ft)	Approximate Piezometric Level Over Top of Clay
PC-1B	155.0	45.0	52.0	110.0	103.0	6.5	5.0
PC-1C	157.1	43.5	--	113.6	--	--	--
PC-1D	159.0	45.5	--	113.5	--	--	--
PC-1E	144.6	32.5	--	112.1	--	--	--
PC-2A	187.8	72.8	78.3	115.0	109.5	5.5	1.4
PC-3A	160.4	46.0	55.3	114.4	105.1	9.3	1.7
PC-6A	152.0	35.2	47.6	116.8	104.4	12.4	1.4

-- Not Measured