2008

Geotechnical Study Associated with
2008 Annual Monitoring of the
Phosphatic Clay Liner
Beneath the Southeast County Landfill
in Hillsborough County, Florida

December 19, 2008

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File Number 08-032

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Attention: Mr. Jason Timmons

Subject: Geotechnical Study Associated with 2008 Annual Monitoring of the Phosphatic Clay Liner Beneath the Southeast County Landfill in Hillsborough County, Florida

Gentlemen/Ladies:

As requested by Jones Edmunds & Associates, Inc. (Jones Edmunds), Ardaman & Associates, Inc. (Ardaman) has completed a geotechnical study associated with annual monitoring of the waste phosphatic clay liner beneath the Southeast County Landfill (SCLF) in Hillsborough County. The annual monitoring program was mandated by the Florida Department of Environmental Protection (FDEP) under Specific Condition No. 16f of the Landfill Operation Permit No. 35435-006-SO issued on June 25, 2002. The monitoring program involves performance of piezocone soundings and measurements of pore water pressures in the vicinity of the following four test sites where Ardaman and Madrid Engineering Group, Inc. (Madrid) had previously explored in 2001 and 2002:

- PC-1B in the Phase I area
- PC-4B/PC-4C in the Phase IV area
- PC-3/PC-3B in the Phase III area
- PC-1F in the Phase I area

Specifically, the permit condition requires documentation and interpretation of the following data:

- Piezometric elevations in the drainage sand layer above the phosphatic clay.
- Top and bottom elevations of the phosphatic clay layer.
- Pore water pressures near the top, middle, and bottom of the phosphatic clay layer.
- Piezometric elevations in the natural soils below the phosphatic clay.

Evaluations of the phosphatic clay liner in 2003, 2004, 2005, 2006, and 2007 were documented in Ardaman reports dated April 12, 2003, April 15, 2004, May 27, 2005, April 13, 2006, and May

16, 2007, respectively. This report contains the results of the piezocone soundings and pore water pressure measurements obtained by Ardaman in July and August 2008, and presents our interpretation of the test data.

Site Location

The SCLF is located within Sections 14, 15, 22, and 23 of Township 31 South, Range 21 East, in unincorporated Hillsborough County, Florida. More specifically, the landfill site is located between Picnic and Pinecrest, about 2 miles west of County Road 39 and about 0.5 miles north of County Road 672. The approximate site location, as superimposed on a reproduction of the United States Geological Survey (USGS) quadrangle map of Lithia, Florida (1955, photorevised 1987), is shown in Figure 1.

Project History

Phases I through VI of the SCLF are constructed directly above a waste clay settling area at a former phosphate mine known as Lonesome Phosphate Mine or Boyette Mine. The settling area, also known as Settling Area No. 1, was built on natural ground within a perimeter dike constructed of sand borrowed from surrounding areas. Waste phosphatic clay slurry was discharged into the settling area during the mining operation.

Between 1981 and 1983, Ardaman conducted a comprehensive geotechnical study to characterize the phosphatic clay deposit and to evaluate the feasibility of constructing a landfill within the waste clay settling area. Results of that study were documented in an Ardaman report titled "Hydrogeological Investigation, Southeast County Landfill, Hillsborough County, Florida," dated February 22, 1983. Based on the data and analyses documented in that report, Ardaman concluded that a landfill could be constructed directly on top of the phosphatic clay. However, to maintain an adequate factor of safety against slope failure, the waste disposal area was divided into different phases, and each phase had to be filled in lifts so that filling above a previous lift would occur only when the underlying phosphatic clay had consolidated under the weight of the previous refuse lift and had experienced sufficient increase in shear strength to support the additional load. In areas where the clay thickness was greater than 14 feet, it was recommended that the waste phosphatic clay should be pre-loaded before the first lift of refuse was placed. Figure 2 shows the original thickness of the waste phosphatic clay within the former settling area. As shown, the waste phosphatic clay had an original thickness that varied between 4 and 18 feet.

Ardaman completed another comprehensive geotechnical study in 1994 in association with the operation permit renewal for the SCLF. Results of that study were documented in an Ardaman report titled "Geotechnical Investigation at Southeast Landfill, Hillsborough County, Florida," dated March 7, 1994. The strength and consolidation properties of the phosphatic clay obtained from that study were in good agreement with those used in the original stability analyses and affirmed the recommended filling schedule.

In support of the operation permit renewal application in 2002, SCS Engineers retained Madrid Engineering Group and Ardaman to perform supplemental studies to confirm the engineering properties of the phosphatic clay, and to determine whether the material had been consolidating and gaining strength as predicted. Results of that study were presented in an Ardaman report titled "Geotechnical Study Associated with Operation Permit Renewal for Hillsborough County Southeast Landfill," dated March 4, 2002. Key findings of our 2002 study were as follows:

- There was a consistent trend of increased tip resistance from the cone soundings within the phosphatic clay deposit, which was expected as a result of landfill loading.

- The measured undrained shear strengths of the phosphatic clay under existing conditions were generally within the expected range.

- The coefficients of consolidation of the waste phosphatic clay were generally consistent with those documented from previous studies.

The original geotechnical investigation completed in 1983 and the follow-up studies completed in 1994 and 2002 recommended that each lift of refuse should have a thickness no greater than 20 feet and that a waiting period should be provided between placements of successive refuse lifts. These requirements were derived based on stability analyses using an undrained shear strength to effective vertical stress ratio of 0.21 and a coefficient of consolidation of $1.5x10^{-4}$ cm^2/sec for the waste phosphatic clay. The undrained shear strength to effective vertical stress ratio determines the magnitude of strength increase in the phosphatic clay, whereas the coefficient of consolidation governs the rate of strength increase.

Field Testing Program

Current operation at the SCLF divides the waste disposal area into six phases designated Phases I through VI, as shown on a topographic site plan in Figure 3. The topographic site plan was generated from photogrammetric data obtained by Pickett & Associates from aerial photography taken in July 2008, at which time refuse was being placed in the Phase V and VI areas. Placement of the current refuse lift in the Phase V and VI areas has occurred since the early part of 2007 and is scheduled to be completed by the end of 2008.

For the 2008 annual monitoring program, Ardaman performed piezocone soundings and pore water pressure measurements at the following four test sites selected by Jones Edmunds:

- PC-1Q in the western portion of the Phase I area

- PC-1R in the eastern portion of the Phase I area

- PC-3H in the Phase III area

- PC-4I in the Phase IV area

The approximate locations of these test sites are shown in Figure 3 along with the test site locations for the 2001/2002 studies performed by Madrid and Ardaman, and for the 2003, 2004, 2005, 2006, and 2007 annual monitoring programs. As shown in Figure 3, PC-1Q was located between the previous PC-1B, PC-1H and PC-1J test site locations, and PC-1R was selected between the previous PC-1, PC-1F, PC-1G, and PC-1I test site locations. PC-3H was located between the previous PC-3, PC-3C, PC-3D, and PC-3B test site locations. PC-4I was selected approximately 70 to 80 feet east of the PC-4D test site location. Ardaman performed the current field work and testing in July and August 2008.

The surveyed coordinates and ground surface elevations at the current test site locations, as provided by Jones Edmunds, are summarized in Table 1. The coordinates were referenced to the Florida State Plane Coordinate System (NAD83). The elevations were surveyed using the National Geodetic Vertical Datum of 1929 (NGVD). The elevation data used in the previous geotechnical studies were also based on NGVD.

Ardaman performed the field work in two stages. The first stage of our work involved drilling through the refuse using an auger rig at the four test sites followed by performing piezocone soundings from the drainage sand layer, through the phosphatic clay layer, and into the underlying natural soils. The objectives of the first stage of work were to determine the depths to the top and bottom of the waste phosphatic clay layer and, where possible, the depths to the drainage sand layer, and to establish the depths where the piezoprobes for measurements of pore water pressures should be installed. We also attempted to document the pore water pressures within the sandy soils above and below the phosphatic clay layer using the piezocone. The second stage of our work involved drilling additional test holes through the refuse adjacent to the piezocone sounding locations, installing piezoprobes at different depths within the phosphatic clay layer at each of the four test sites, and monitoring of pore water pressure dissipation in the piezoprobes.

The test holes were grouted upon completion of the piezocone soundings and piezoprobe measurements and the test site locations were staked for surveying by Jones Edmunds.

The drilling program was performed in accordance with a health and safety plan that Ardaman developed for this project. An Ardaman technician performed continuous air monitoring using a gas/methane detection meter and an organic vapor analyzer during the drilling operations.

Piezocone Penetration Tests

The piezocone consists of a conical point attached to a steel rod and a friction sleeve. The test is performed by pushing the assembly into the soil at a constant rate of penetration. Resistance to penetration at the cone tip and on the friction sleeve is measured by load cells placed within the assembly and the pore water pressure in the soil is measured using a pressure transducer connected to the porous element placed near the cone tip. Before the piezocone was pushed through the waste phosphatic clay, an auger was used to create a borehole through the refuse.

Results of the four piezocone penetration tests (i.e., PC-1Q, PC-1R, PC-3H, and PC-4I) performed at the four test site locations are presented in Figures 4 through 7, respectively. As shown in the figures, results are presented in the form of tip resistance (i.e., the resistance to penetration at the cone tip), sleeve resistance (i.e., the resistance to penetration of the friction sleeve), pore water pressure (i.e., the total pore water pressure including the pore pressure generated due to penetration of the cone), and friction ratio (i.e., the ratio of sleeve resistance to tip resistance).

Since sandy soils typically exist above and below the phosphatic clay, the depth and thickness of the phosphatic clay layer could be inferred by examining the variations of tip resistance and pore water pressure with depth. The tip resistance and the pore water pressure in a clayey soil are expected to be lower and higher, respectively, than those in a sandy soil. Higher friction ratios generally indicate clayey soils, whereas lower ratios generally indicate the presence of silty and sandy soils. Sudden changes in tip resistance, pore water pressure, and friction ratio are expected to occur at the interface between the drainage sand layer and the underlying

phosphatic clay as well as the interface between the phosphatic clay and the underlying natural sandy soils.

Thickness of Refuse/Ash

The refuse/ash thicknesses at the four piezocone test sites (i.e., PC-1Q, PC-1R, PC-3H, and PC-4I) varied from approximately 59 feet at PC-3H to 72 feet at PC-1Q in July and August 2008, assuming that the drainage sand layer has a thickness of 2 feet.

PC-1Q was located in the western portion of the Phase I area. The refuse/ash thickness at PC-1Q was estimated to be 72 feet in July 2008, compared to a refuse/ash thickness of 69 feet at PC-1O in February 2007, 53 feet at PC-1M in February 2006, a refuse/ash thickness of 51 feet at PC-1K in March 2005, and a refuse/ash thickness of 65 feet at PC-1J in March 2004. The main reasons for the variations in refuse/ash thickness were because these test sites were not at identical locations and the landfill surface elevations in the test sites area varied considerably. We understand that no refuse/ash was placed in the Phase I area since August 2004.

PC-1R was located in the eastern portion of the Phase I area. The refuse/ash thickness at PC-1R was estimated to be 67 feet in July 2008, compared to a refuse/ash thickness of 67 feet at PC-1P in February 2007, 70 feet at PC-1N in February 2006, and 66 feet at PC-1L in March 2005. We understand that no refuse/ash was placed in the Phase I area since August 2004.

The refuse/ash thickness at PC-3H was documented to be approximately 59 feet in July 2008, compared to a thickness of approximately 63 feet at PC-3G in February 2007, and 53 feet at PC-3F in February 2006. We understand that no refuse/ash was placed in the Phase III area since August 2006.

The refuse/ash thickness at PC-4I during the current field program was estimated at 68 feet, compared to a refuse/ash thickness of 61 feet at PC-4H in February 2007, 55 feet at PC-4G in February 2006, slightly greater than 60 feet at PC-4F in March 2005, and slightly greater than 50 feet at PC-4E in March 2004. The landfill surface elevation at PC-4I was approximately 7 feet higher than the landfill surface elevation at PC-4H. We understand that some filling, short of a full 20-foot lift, occurred near these test site locations.

Filling Schedule in Phase I through VI Areas

According to the landfilling schedule provided by Hillsborough County, the latest lift of refuse/ash in the Phase I area was placed between March 2003 and August 2004 (i.e., around 4 years ago), and that in the Phase II area was placed between August 2004 and August 2005 (i.e., around 3 years ago), corresponding to 7 to 8 years after the prior design lifts of refuse/ash were placed in these two areas.

The landfilling schedule also indicates that the latest lift of refuse/ash in the Phase III area was placed between August 2005 and August 2006 (i.e., around 3 years ago), corresponding to more than 14 years after the prior 20-foot thick refuse/ash lift was placed. Beginning in August 2006, the landfilling operation moved to the Phase IV area and continued through the time of our field exploration in February/March of 2007, corresponding to approximately 12 years after the prior refuse/ash lift was placed.

After completion of filling of the Phase IV in the early part of 2007, the landfill operation has moved into the Phase V and VI areas and continued through the time of our field exploration in July/August of 2008. Placement of the current refuse lift in the Phase V and VI areas is scheduled to be completed by the end of 2008, at which time Hillsborough County will either move the landfill operation to the top of the Phase I area or to Section 9, which has recently been constructed.

Elevations and Thicknesses of Phosphatic Clay

Before landfill construction, the top surface of the waste phosphatic clay within the Phases I through VI areas was documented in the 1983 Ardaman report to have typical elevations in the range of +121 to +123 feet (NGVD) and the bottom of the waste phosphatic clay reportedly occurred at typical elevations ranging from +103 to +117 feet (NGVD). As indicated previously, the original thickness of the waste phosphatic clay ranged from 4 to 18 feet.

Based on results of the 2008 piezocone soundings, the top and bottom elevations of the phosphatic clay and the phosphatic clay thicknesses encountered at the four test sites in 2008 are summarized in Table 2. As shown, the top elevations of the phosphatic clay ranged from a low of +109.4 feet (NGVD) at PC-4Q to a high of +119.0 feet (NGVD) at PC-3H, whereas the bottom elevations of the phosphatic clay layer ranged from a low of +102.3 feet (NGVD) at PC-4I to a high of +112.0 feet (NGVD) at PC-3H.

A comparison of the original and latest phosphatic clay elevations indicated that the top elevations of the phosphatic clay have become lower, which was likely a result of compression of the phosphatic clay layer from landfill loading. The bottom elevations appeared to have remained within the same range as the original elevations before landfill construction.

At the location of PC-1Q, the top and bottom of the phosphatic clay layer were encountered at approximately +109.4 and +102.9 feet (NGVD), respectively, for a clay thickness of approximately 6.5 feet, which is 1 foot less than the clay thickness (7.5 feet) documented at PC-1O in February 2007.

Based on the piezocone sounding performed at the location of PC-1R, the phosphatic clay had top and bottom elevations of +117.1 and +108.6 feet (NGVD), respectively, for a phosphatic clay thickness of 8.5 feet, which is much greater than the thickness of 3.6 feet documented at the nearby PC-1P in February 2007 and 3.9 feet documented at the nearby PC-1N in February 2006. A review of the elevation data indicated that the bottom elevation of the waste phosphatic clay at PC-1R was 6.5 feet lower than that documented at PC-1P and 3.9 feet lower than that documented at PC-1N. The greater waste phosphatic clay thickness could be explained by a localized depression on natural ground before discharge of the waste phosphatic clay slurry into the former settling area.

The piezocone sounding performed at the location of PC-3H revealed the top elevation of the phosphatic clay at +119.0 feet (NGVD) and the bottom elevation at +112.0 feet (NGVD), for a clay thickness of 7.0 feet. This thickness is relatively close to the clay thickness of 7.2 feet documented at PC-3G in February 2007.

Based on the piezocone sounding performed at the location of PC-4I, the top and bottom elevations of the phosphatic clay were documented at +110.3 and +102.3 feet (NGVD), for a

clay thickness of 8.0 feet, which is fairly close to the thickness of 7.6 feet documented at the nearby PC-4H test site in February 2007.

Compared to the latest known thicknesses at corresponding nearby test sites, the phosphatic clay thicknesses documented at the four current test sites in July/August 2008 indicated that the clay thicknesses near PC-1Q and PC-3H have decreased by 0.2 to 1.0 foot, whereas the phosphatic clay thickness near PC-4I has increased by 0.4 feet. These differences are considered minimal. Differences in phosphatic clay thickness could be attributed to settlement of the phosphatic clay layer or the test sites not being at identical locations from year to year.

At PC-1R, the thickness of the clay layer appeared to have increased by 4.9 feet compared with the documented values of 3.6 feet in February 2007 and 3.9 feet in February 2006. A review of the elevation data indicated that the bottom elevation of the waste phosphatic clay at PC-1R was lower than that documented at PC-1P by approximately 6.5 feet. Therefore, the increase in phosphatic clay thickness could be explained by a localized depression on natural ground before discharge of the waste phosphatic clay slurry into the former settling area.

Piezometric Elevations on Top of Phosphatic Clay

The piezometric heads in the drainage sand layer on top of the phosphatic clay could theoretically be inferred from the piezocone penetration test results. As the piezocone was pushed through the drainage sand layer on top of the phosphatic clay, it could be held stationary at a selected depth to allow the excess pore water pressure generated as a result of pushing of the piezocone to stabilize. Because of the relatively high permeability of the sand, any excess pore water pressure should dissipate in a relatively short time. The pore pressures could be monitored for several minutes to obtain the final readings that represent the piezometric elevation at the selected depth. Piezoprobes were also used to determine the piezometric elevations within the phosphatic clay layer and within the sand layers located above and below the phosphatic clay layer.

At the location of PC-1Q, we estimated the piezometric elevation on top of the phosphatic clay to be +114.5 feet (NGVD), based on the piezocone pore pressure measurement that was measured just above the surface of the phosphatic clay. At the location of PC-1R, we estimated the piezometric elevation on top of the phosphatic clay to be +119.9 feet (NGVD), based on the final equilibrium hydraulic head recorded in the piezoprobe installed within the drainage sand layer above the waste phosphatic clay.

At the location of PC-3H, using the final equilibrium hydraulic head reading of the piezoprobe that was installed just above the surface of the phosphatic clay layer, the piezometric elevation in the drainage sand layer was estimated to be +122.0 feet (NGVD), corresponding to approximately 3 feet above the top of the waste phosphatic clay.

The pore water pressure transducer in the piezocone at the location of PC-4I did not provide any useful data. The collected readings indicated that the transducer either malfunctioned or was damaged in the field as the piezocone penetrated through the drainage sand layer. Accordingly, piezometric elevation data above the waste phosphatic clay could not be obtained at this location. The piezometric elevation on top of the waste phosphatic clay was documented at +119.8 feet (NGVD), which corresponded to approximately 7.6 feet above the waste phosphatic clay surface.

The estimated piezometric heads in the drainage sand layer on top of the phosphatic clay are summarized in Table 3 and are further displayed in Figure 8. Piezometric heads documented from previous studies are also shown in the figure for comparison. As shown in Table 3 and Figure 8, the piezometric heads on top of the phosphatic clay at the four test site locations were estimated to range from 2.8 to 5.1 feet. The highest piezometric elevation of +122 feet (NGVD), corresponding to 3.0 feet above the top of the phosphatic clay, was observed at PC-3H. The latest piezometric elevations in the drainage sand layer were comparable to previous data.

Piezometric Elevations below Phosphatic Clay

The piezometric elevations below the phosphatic clay were estimated based on the piezocone sounding results, the final equilibrium hydraulic head readings in piezoprobes, and previous data. These elevations are summarized in Table 4. As shown, the piezometric elevations at the test site locations ranged from approximately +120 to +127 feet (NGVD). Except for the reading obtained at PC-1R, the latest piezometric elevations were comparable to previous measurements documented in February 2007. The latest piezometric elevation at PC-1R was documented at +120.5 feet (NGVD), which was lower than the piezometric elevation of approximately +128.0 feet (NGVD) at PC-1P in February 2007, but was comparable to the piezometric elevation of approximately +122.5 feet (NGVD) at PC-1N in February 2006.

Piezoprobe Tests within Phosphatic Clay

The piezoprobe tests were performed by installing piezoprobes to various depths within the phosphatic clay and holding them stationary until the excess pore water pressure generated from probe penetration completely dissipated, and the measured pore water pressure reached the actual pore water pressure before probe penetration.

Dissipation of excess pore water pressures generated by piezoprobe penetrations are presented in Figures 9 through 12 for PC-1Q, PC-1R, PC-3H, and PC-4I, respectively. Figures 9, 10, 11, and 12 are in the form of normalized excess pore pressure (i.e., the ratio of excess pore water pressure at any time to the maximum excess pore water pressure after piezoprobe penetration) versus time. As shown in the figures, all pore water pressures reached equilibrium conditions at the end of the monitoring periods. The rate of dissipation of excess pore water pressure generated by piezoprobe penetration can be used to estimate the *in situ* coefficient of consolidation of the phosphatic clay.

As indicated previously, the waste phosphatic clay thicknesses at the locations of PC-1Q, PC-1R, PC-3H, and PC-4I were approximately 6.5, 8.5, 7.0, and 8.0 feet, respectively. At each of these locations, piezoprobes were installed at three different depths for monitoring of pore water pressure dissipation. Results from the piezoprobe tests are summarized in Table 5.

At the location of PC-1Q, the equilibrium piezometric elevations for the three piezoprobes installed with tip elevations at +108.1, +105.9, and +103.9 feet (NGVD) were documented to be approximately +117.9, +115.8, and +118.9 feet (NGVD), respectively. The piezoprobe with the tip elevation at +108.1 feet (NGVD) was 1.3 feet below the top elevation of the phosphatic clay. At this tip elevation, the piezoprobe readings indicated an excess pore water pressure of 3.2 feet of water. The excess pore water pressure for the tip elevation at +105.9 and +103.9 feet (NGVD) was documented to be 0.6 and 3.3 feet, respectively. Therefore, it appears that approximately 1 to 3 feet of excess pore water pressure remains within the waste phosphatic

clay at this test site location. The maximum excess pore water pressure at PC-1O in 2007 was documented to be approximately 0 to 2 feet.

At the location of PC-1R, three piezoprobe tests were performed with the piezoprobe tip elevations at +114.1, +112.9 and +110.0 feet (NGVD). The piezoprobe with the tip elevation at +114.1 feet (NGVD) was 3 feet below the top elevation of the phosphatic clay. At this tip elevation, the piezoprobe readings indicated an excess pore water pressure of 3.6 feet of water. The excess pore water pressure for the tip elevation at +112.9 and +110.0 feet (NGVD) was documented to be 5.4 and 0.9 feet, respectively. Therefore, it appears that approximately 1 to 5 feet of excess pore water pressure remains within the phosphatic clay at this test site location. The maximum excess pore water pressure at PC-1P in 2007 was documented to be approximately 2 to 6 feet.

At the location of PC-3H, piezoprobe tests were performed with the piezoprobe tip elevations at +117.3, +115.5, and +113.0 feet (NGVD). The data indicated that the excess pore water pressure within the waste phosphatic clay was as much as 18 feet. In February 2007, the maximum excess water pressure in this area was also documented to be approximately 18 feet. As indicated previously, refuse was last placed in the Phase III area in the early part of 2007.

At the location of PC-4I, three piezoprobe tests were performed within the phosphatic clay layer with the piezoprobe tip elevations at +109.0, +105.8, and +103.6 (NGVD). The recorded data indicated that the equilibrium piezometric elevations were much low than expected, and did not produce any usable data.

In summary, the piezoprobe data indicated some residual excess pore water pressure remained at the locations of PC-1Q and PC-1R, and high excess pore water pressure was documented at PC-3H. For comparison, a 20-foot lift of refuse/ash is expected to generate about 20 to 25 feet of excess pore water pressure.

Coefficient of Consolidation

The coefficient of consolidation governs the rate of strength increase in the phosphatic clay upon loading from a refuse lift and thus the waiting period between placements of successive refuse lifts in an area. The filling schedule and the recommended waiting period between placement of successive refuse/ash lifts at the SCLF was originally based on a design vertical coefficient of consolidation (c_v) of 1.5×10^{-4} cm^2/sec, a phosphatic clay thickness of 12 feet, and a double drainage condition.

Based on the rates of excess pore water pressure dissipation shown in Figures 9 through 12, the *in situ* vertical coefficient of consolidation of the waste phosphatic clay, c_v, were calculated based on empirical relationships proposed by Baligh and Levadoux (*ASCE Journal of Geotechnical Engineering*, Vol. 112, No. 7, July, 1986). Based on the equations in this reference, $c_v \approx 0.05 \times c_h$, where c_h represents the horizontal coefficient of consolidation derived from piezoprobe readings. Results of the calculations are presented in Table 6.

As shown in Table 6, the coefficients of consolidation calculated from the piezoprobe data for 2008 were, in general, slightly lower than the design coefficient of consolidation and previous measured values. Settlement plate data at the location of Pump Station B in the Phase VI area between 1999 and 2008, as provided by Hillsborough County, are plotted in Figure 13. At this location, the initial thickness of the waste phosphatic clay was approximately 13 feet. As shown

in Figure 13, the waste phosphatic clay has undergone a settlement of about 5.4 feet over a period of approximately 10 years, with a current thickness of approximately 7.6 feet. Assuming that an average degree of consolidation of 95 percent has been reached and an average waste phosphatic clay thickness of approximately 10 feet, the average coefficient of consolidation was back-calculated to be on the order of $1.0x10^{-4}$ cm^2/sec, which is lower than the design coefficient of consolidation of $1.5x10^{-4}$ cm^2/sec. The data also indicated that the coefficient of consolidation has decreased with time. A lower coefficient of consolidation will require a longer waiting period between placement of successive refuse lifts. However, the thickness of the waste phosphatic clay has reduced over the years, which will decrease the waiting period.

Comparisons of Piezometric Heads

The piezometric elevations in the materials directly above and below the phosphatic clay layer, as documented from the piezocone soundings, are summarized in Table 7. The piezometric heads within the phosphatic clay layer are also shown on the same table for comparison.

The piezometric head within the phosphatic clay will be highest after loading of a new refuse/ash lift and will decrease gradually as excess pore water pressure dissipates. If the piezometric head within the phosphatic clay is higher than the piezometric head on top of the phosphatic clay, there will be no downward migration of leachate. Once the excess pore water pressure from landfill loading dissipates, the flow direction through the phosphatic clay will be a function of the piezometric head difference across the phosphatic clay. If the piezometric elevation in the natural soils below the phosphatic clay is higher than the piezometric elevation on top of the phosphatic clay, upward flow will occur. Conversely, if the piezometric elevation in the natural soils is lower than the piezometric elevation on top of the phosphatic clay, leachate will migrate downward, at a rate governed by the piezometric head difference, and the hydraulic conductivity and thickness of the phosphatic clay deposit.

As shown in Table 7, the existing maximum piezometric heads within the phosphatic clay layer are higher than the piezometric heads in the materials above the phosphatic clay at PC-1Q, PC-1R, and PC-3H. At PC-4I, the piezometric head below clay layer is higher than the piezometric head in the materials above the phosphatic clay. Accordingly, under existing condition, there should be no downward leachate migration through the phosphatic clay layer at the four test site locations in 2008.

Summary of Observations

The following key observations were derived from the 2008 annual monitoring data:

- The average refuse/ash thicknesses were estimated to be on the order of 60 feet in the Phase III area, 60 to 70 feet in the Phase IV areas, and 65 to 70 feet in the Phase I areas.

- A comparison of the original (i.e., before landfill construction) and latest phosphatic clay elevations indicated that the top elevations of the phosphatic clay have become lower, which was a result of compression of the phosphatic clay layer from landfill loading. However, the bottom elevations have remained approximately within the same range as the original elevations before landfill construction.

- The waste phosphatic clay thicknesses documented in July/August 2008 at PC-1Q, PC-3H, and PC-4I were within 1 foot of those documented in February 2007. The small differences

could be attributed to settlement of the phosphatic clay layer or the test sites not being at identical locations from year to year. At PC-1R, the thickness of the clay layer appeared to have increased by 4.9 feet. A review of the elevation data indicated that the bottom elevation of the waste phosphatic clay at PC-1R was 6.5 feet lower than that documented at PC-1P in 2007 and 3.9 feet lower than that documented at PC-1N in 2006. The greater waste phosphatic clay thickness could be explained by a localized depression on natural ground before discharge of the waste phosphatic clay slurry into the former settling area.

- The piezoprobe data indicated some residual excess pore water pressure at PC-1Q and PC-1R, and high excess pore water pressure at PC-3G. In the Phase I area, there was an average excess pore water pressure of 2 to 3 feet within the waste phosphatic clay. This represents an average consolidation on the order of 90 percent, considering that a 20-foot lift of refuse/ash would generate approximately 20 to 25 feet of excess pore water pressure. The higher excess pore water pressure at PC-3G is probably a result of refuse/ash placement in the Phase III area through the early part of 2007.

- The piezometric heads above the phosphatic clay at the PC-1Q, PC-1R, and PC-3H test site locations were approximately 3 to 5 feet above the clay surface.

- The piezometric head differences indicated no downward leachate migration through the phosphatic clay layer at the test site locations.

- The coefficients of consolidation calculated from the piezoprobe data for 2008 were, in general, slightly lower than the design coefficient of consolidation and previous measured values. Settlement plate data at the location of Pump Station B in the Phase VI area between 1999 and 2008 indicated an average coefficient of consolidation on the order of 1.0×10^{-4} cm^2/sec, which is lower than the design coefficient of consolidation of 1.5×10^{-4} cm^2/sec. The settlement plate data also indicated that the coefficient of consolidation has decreased with time. A lower coefficient of consolidation will require a longer waiting period between placement of successive refuse lifts. However, the thickness of the waste phosphatic clay has reduced over the years, which will decrease the waiting period.

Recommendations

The latest filling within the Phase I area occurred between March 2003 and March 2005. At the locations of PC-1Q and PC-1R, there was an average excess pore water pressure of 2 to 3 feet within the waste phosphatic clay. This represents an average consolidation on the order of 90 percent, considering that a 20-foot lift of refuse/ash would generate approximately 20 to 25 feet of excess pore water pressure. The measured pore pressure values are not unexpected and should allow placement of waste in the Phase I area. Nevertheless, Ardaman recommends performance of stability analyses to evaluate the implications of the residual pore water pressure in the Phase I area before placement of the next lift of refuse. The high excess pore water pressure at PC3-G is not unexpected because of the refuse/ash placement in the Phase III area through the early part of 2007.

As proposed in the FDEP permit renewal application, undisturbed sampling and laboratory strength testing of the waste phosphatic clay in the Phase I area will be performed. Based on the measured undrained shear strengths, slope stability analyses should be performed to determine the factor of safety of the landfill slope after placement of the next 20-foot thick lift of

refuse/ash in the Phase I area. This will provide further information regarding the Phase I area. Similar sampling and testing programs are also recommended before placement of refuse in future phases, as recently proposed in the FDEP permit renewal application.

Closure

This report has been prepared for the exclusive use of Jones Edmunds and the Hillsborough County Solid Waste Management Department for specific application to annual monitoring of the phosphatic clay liner at the SCLF in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

Ardaman appreciates the opportunity to assist you on this project. Please contact us if you have any questions concerning this report or need additional information.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.
Certificate of Authorization No. 5950

Jeyisanker Mathiyaparanam, E.I.
Assistant Project Engineer

Francis K. Cheung, P.E. 12/19/08
Senior Project Engineer
Florida License No. 36382

Enclosure

Table 1

Test Site Locations and Elevations

Test Site	Area	Florida State Plane Coordinates		Approximate Ground Surface Elevation
		Northing (feet)	Easting (feet)	(feet, NGVD)
PC-1Q	Phase I West	1,250,367.50	595,957.96	+183.4
PC-1R	Phase I East	1,249,876.43	597,307.38	+185.6
PC-3H	Phase III	1,251,197.51	597,350.45	+179.5
PC-4I	Phase IV	1,250,612.87	596,044.42	+180.3

Coordinates and elevations were provided by Jones Edmunds based on a survey performed by Pickett & Associates, Inc.

Table 2

Top and Bottom Elevations of Phosphatic Clay

Area	Test Site	Date	Approximate Ground Surface Elevation (feet, NGVD)	Top of Clay		Bottom of Clay		Clay Thickness (feet)
				Depth (feet bls)	Elevation (feet, NGVD)	Depth (feet bls)	Elevation (feet, NGVD)	
Phase I	PC-1Q	07/18/08	+183.4	74.0	+109.4	80.5	+102.9	6.5
	PC-1R	07/18/08	+185.6	68.5	+117.1	77.0	+108.6	8.5
Phase III	PC-3H	07/18/08	+179.5	60.5	+119.0	67.5	+112.0	7.0
Phase IV	PC-4I	07/18/08	+180.3	70.0	+110.3	78.0	+102.3	8.0

Table 3

Piezometric Levels on Top of Phosphatic Clay

Area	Test Site	Date	Ground Surface Elevation (feet, NGVD)	Piezometric Elevation on Top of Phosphatic Clay (feet, NGVD)	Top of Clay Elevation (feet, NGVD)	Piezometric Head on Top of Phosphatic Clay (feet)
Phase I	PC-1Q	08/18/08	+183.4	+114.5	+109.4	5.1
	PC-1R	09/10/08	+185.6	+119.9	+117.1	2.8
Phase III	PC-3H	09/10/08	+179.5	+122.0	+119.0	3.0
Phase IV	PC-4I	07/23/08	+180.3	*	+110.3	*

* The pore water pressure transducer in the piezocone at the location of PC-4I did not produce any usable data. The collected readings indicated that the transducer either malfunctioned or was damaged in the field as the piezocone penetrated through the drainage sand layer. The piezometric elevation at a nearby test site in 2007 was documented at +119.8 feet (NGVD), which corresponded to approximately 7.6 feet above the waste phosphatic clay surface.

Table 4

Piezometric Elevations Below Phosphatic Clay

Area	Test Site	Date	Ground Surface Elevation (feet, NGVD)	Piezometric Elevation Below Phosphatic Clay (feet , NGVD)
Phase I	PC-1Q	08/18/08	+183.4	+125.0
	PC-1R	09/10/08	+185.6	+120.5
Phase III	PC-3H	09/10/08	+179.5	+123.2
Phase IV	PC-4I	08/18/08	+180.3	+127.1

Table 5

Pore Water Pressure within the Phosphatic Clay

Area	Test Site	Ground Surface Elevation (ft, NGVD)	Top of Clay Elevation (ft, NGVD)	Bottom of Clay Elevation (ft, NGVD)	Piezoprobe Tip Below Ground Surface (feet)	Elevation of Piezoprobe Tip (ft, NGVD)	Stabilized Pore Water Pressure at Piezoprobe Tip After Dissipation (feet of H_2O)	Piezometric Elevation at Piezoprobe Tip Level (ft, NGVD)	Piezometric Elevation on Top of Phosphatic Clay (ft, NGVD)	Piezometric Elevation Below Phosphatic Clay (ft, NGVD)	Excess Pore Water Pressure at Piezoprobe Tip Level (feet of H_2O)
Phase I	PC-1Q	+183.4	+109.4	+102.9	75.3 77.5 79.5	+108.1 +105.9 +103.9	9.8 9.9 15.0	+117.9 +115.8 +118.9	+114.5	+115.8	3.2 0.6 3.3
	PC-1R	+185.6	+117.1	+108.6	71.5 72.7 75.6	+114.1 +112.9 +110.0	9.6 12.9 11.3	+123.7 +125.8 +121.3	+119.9	+120.5	3.6 5.4 0.9
Phase III	PC-3H	+179.5	+119.0	+112.0	62.2 64.0 66.5	+117.3 +115.5 +113.0	3.5 22.0 28.5	+120.8* +137.5 +141.5	+122.0	+123.2	-1.5 (?)* 14.9 18.4
Phase IV	PC-4I	+180.3	+110.3	+102.3	71.3 74.5 76.7	+109.0 +105.8 +103.6	0.9 6.0 20.0	+109.9 +111.8 +123.6	**	+127.1	** ** **

* The reading is erroneous and may have been caused by the presence of drainage sand near the top part of the waste phosphatic clay at this test location.

** The piezocone appeared to have been damaged during penetration and, thus, no piezometric elevation could be obtained on top of the waste phosphatic clay to allow calculations of excess pore water pressures at this test location.

Table 6

Computed Coefficients of Consolidation from Piezoprobe Tests

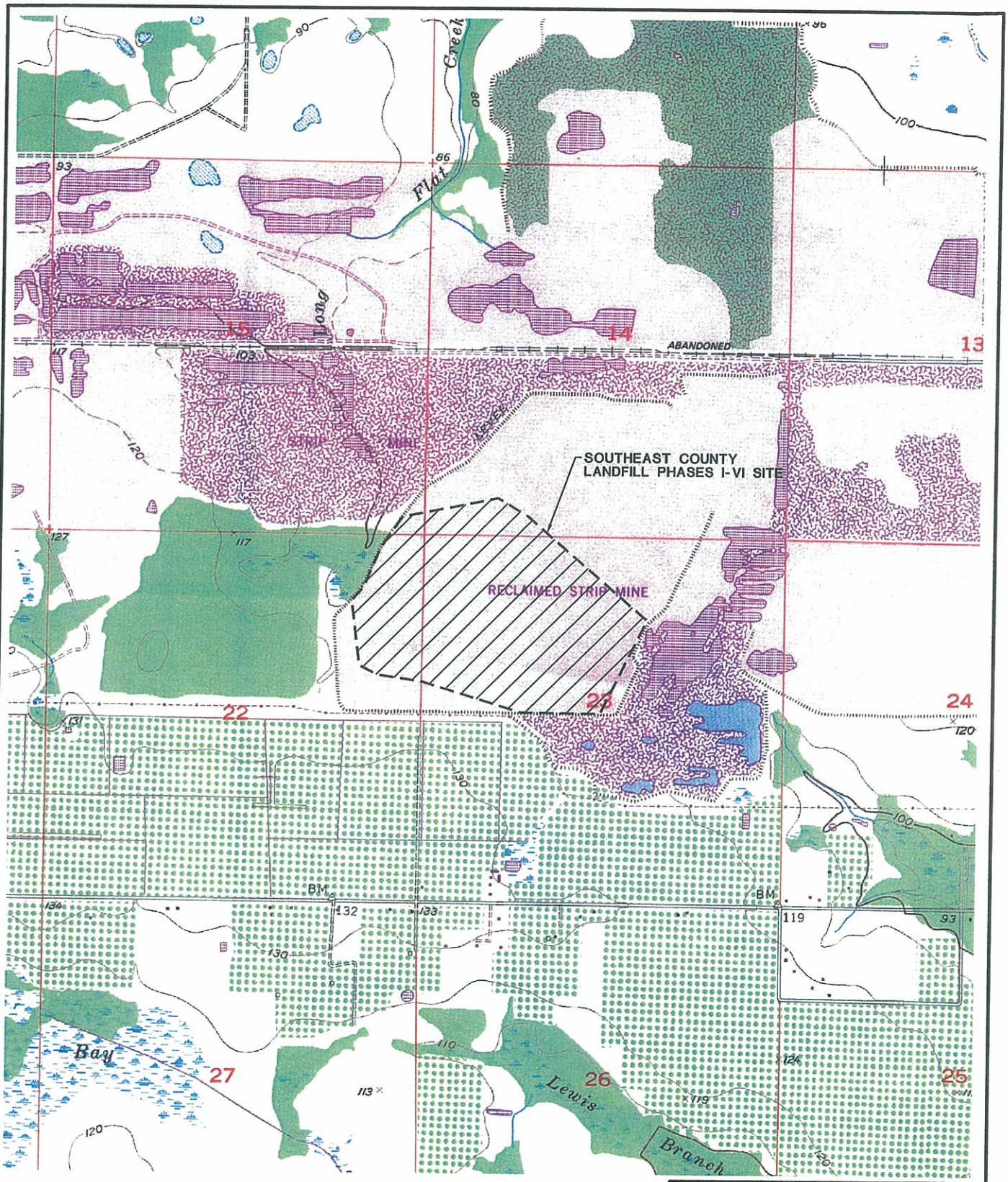
Area	Test Site	Piezoprobe Designation	Piezoprobe Tip Depth Below Ground Surface (feet)	Elevation of Piezoprobe Tip (feet, NGVD)	Horizontal Coefficient of Consolidation, c_h, from Piezoprobe Readings (cm^2/sec)	Vertical Coefficient of Consolidation, c_v*, of Waste Phosphatic Clay (cm^2/sec)
Phase I	PC-1Q	PC-1Q	75.3 77.5 79.5	+108.1 +105.9 +103.9	$1.5x10^{-2}$ $3.0x10^{-3}$ $9.1x10^{-3}$	$7.5x10^{-4}$ $1.5x10^{-4}$ $4.6x10^{-4}$
	PC-1R	PC-1R	71.5 72.7 75.6	+114.1 +112.9 +110.0	$5.9x10^{-4}$ $1.7x10^{-4}$ $1.8x10^{-4}$	$3.0x10^{-5}$ $8.5x10^{-6}$ $9.1x10^{-6}$
Phase III	PC-3H	PC-3H	62.2 64.0 66.5	+117.3 +115.5 +113.0	$4.6x10^{-2}$ $2.4x10^{-3}$ $1.0x10^{-3}$	$2.3x10^{-3}$ $1.2x10^{-4}$ $5.1x10^{-5}$
Phase IV	PC-4I	PC-4I	71.3 74.5 76.7	+109.0 +105.8 +103.6	$9.9x10^{-3}$ $4.0x10^{-3}$ $4.0x10^{-3}$	$4.9x10^{-4}$ $2.0x10^{-4}$ $2.0x10^{-4}$

* c_v using empirical relationship recommended by Baligh and Levadoux (1986).

Table 7

Comparisons of Piezometric Elevations

Area	Test Site	Date	Ground Surface Elevation (feet, NGVD)	Piezometric Elevation on Top of Phosphatic Clay (feet, NGVD)	Existing Maximum Piezometric Elevation Within Clay (feet, NGVD)	Piezometric Elevation Below Phosphatic Clay (feet, NGVD)	Location of Existing Maximum Head
Phase I	PC-1Q	07/24/08	+183.4	+114.5	+118.9	+115.8	Within Clay
	PC-1R	08/20/08	+185.6	+119.9	+125.8	+120.5	Within Clay
Phase III	PC-3H	08/06/08	+179.5	+122.0	+141.5	+123.2	Within Clay
Phase IV	PC-4I	07/23/08	+180.3	-	+123.6	+127.1	Below Clay



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0 1000 2000

SCALE: — 1"=2000'

TOWNSHIP 31 SOUTH
RANGE 21 EAST
SECTIONS 14, 15, 22 AND 23

OBTAINED FROM U.S.G.S. QUAD MAP:
LITHIA, FLORIDA 1955
(PHOTOREVISED 1987)

SITE LOCATION MAP

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

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: CD	CHECKED BY: M.S	DATE: 09/22/08
FILE NO. 08-032	APPROVED BY: FKC	FIGURE: 1

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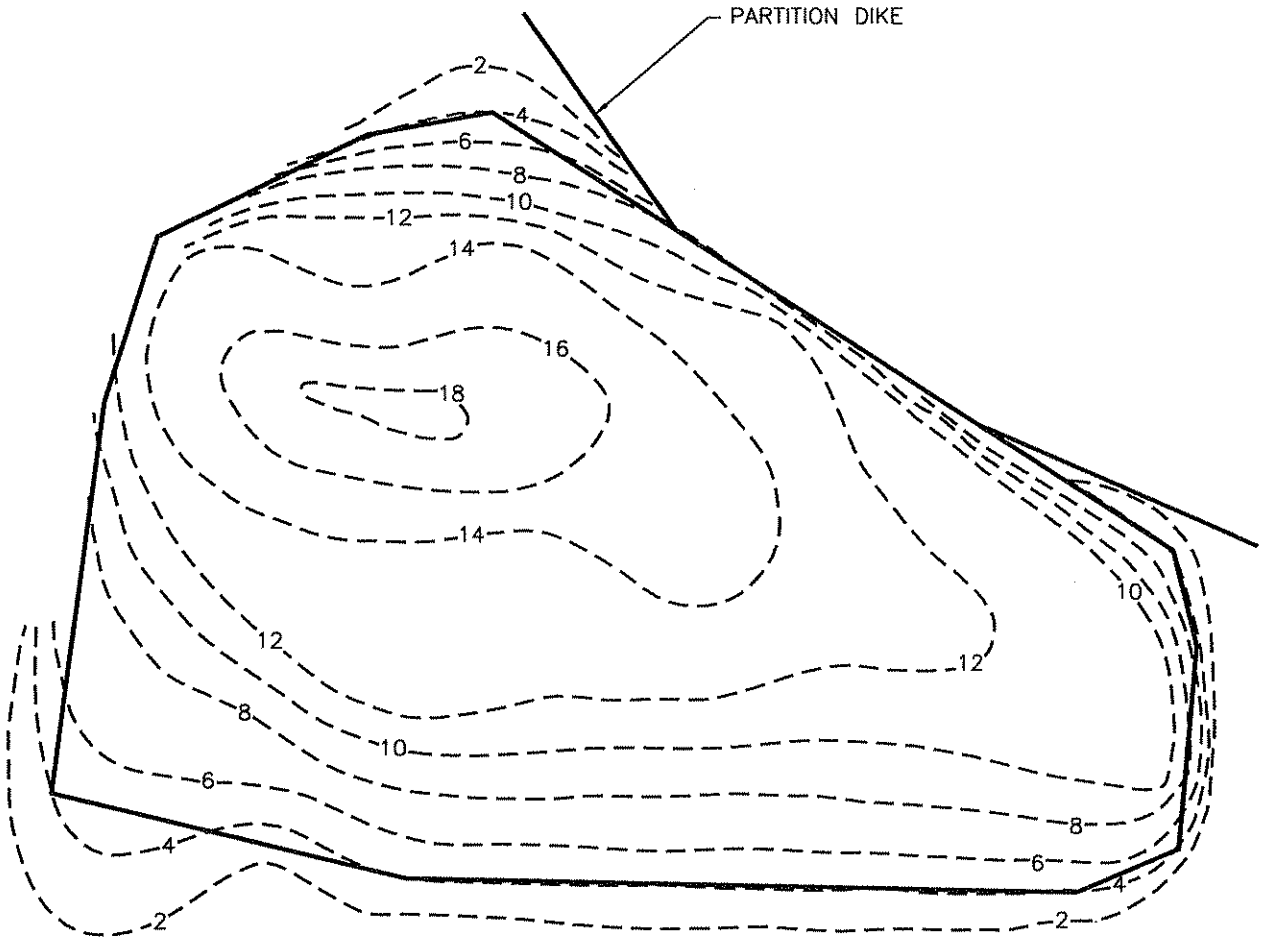

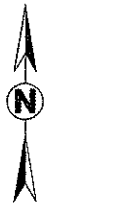

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LEGEND

10 **CONTOURS OF CLAY THICKNESS IN FEET**

NOTE: REPRODUCED FROM ARDAMAN & ASSOCIATES' 1981–1983 STUDY

THICKNESS OF PHOSPHATIC CLAY BEFORE LANDFILL CONSTRUCTION

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

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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FILE NO. 08-032	APPROVED BY: FKC	FIGURE: 2

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Plotted: Sep 22, 2008 – 11:55am by jey.mathiyaparanam

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SCALE: —1"=400'

NOTE: THE TOPOGRAPHY IS BASED ON PHOTOGRAMMETRY FROM AN AERIAL PHOTOGRAPH DATED JULY 2008.

LEGEND

- ⊕ TEST SITES IN 2001–2004
- ◉ TEST SITES IN 2005
- ⊠ TEST SITES IN 2006
- ▲ TEST SITES IN 2007
- ⊕ TEST SITES IN 2008

LANDFILL LAYOUT AND FIELD TEST LOCATION MAP

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GEOTECHNICAL STUDY ASSOCIATED WITH 2007 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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FILE NO. 08-032	APPROVED BY:	FIGURE: 3

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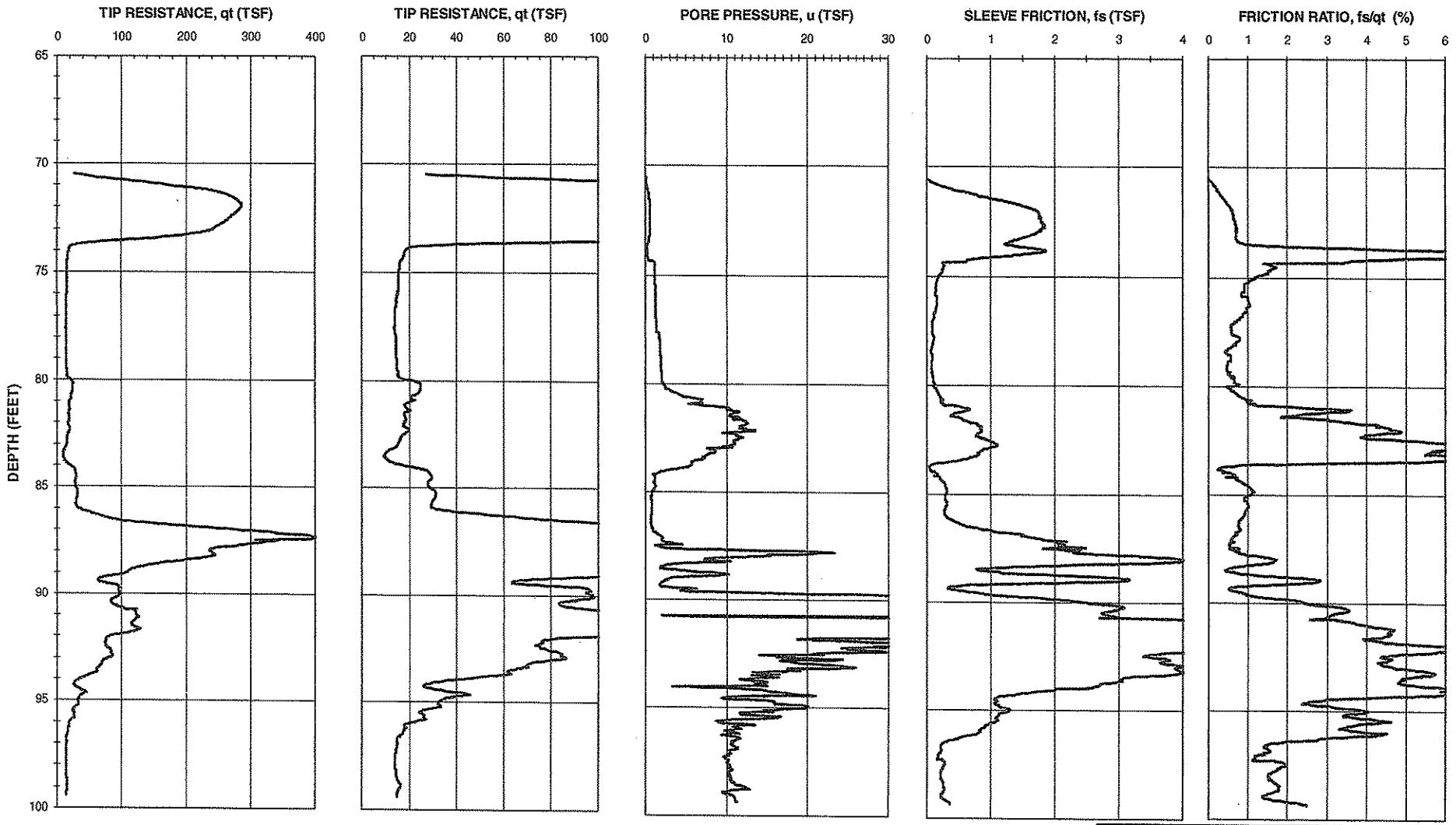

CONE PENETRATION TEST RESULTS FOR PC-1Q

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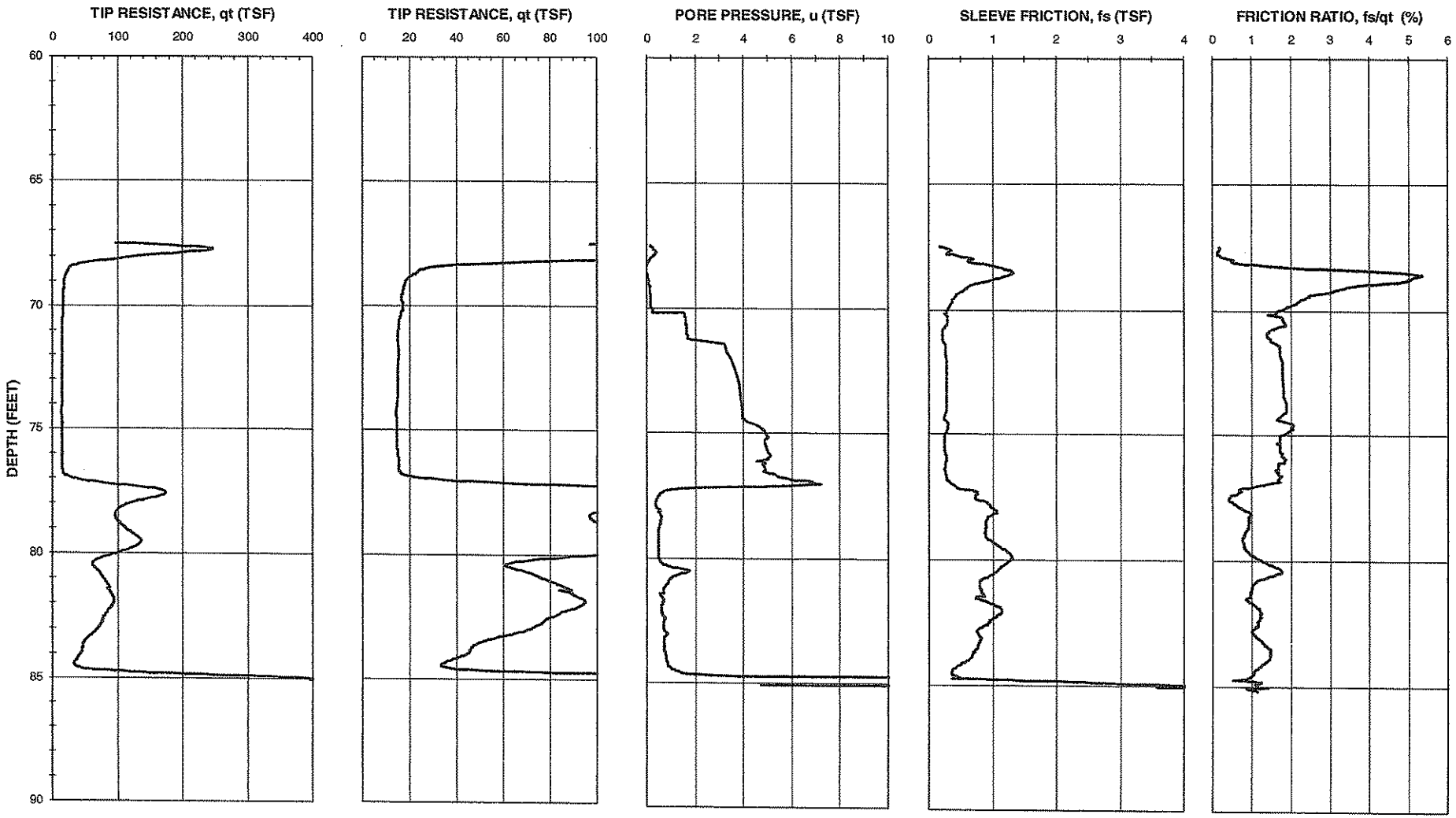
GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: JM	CHECKED BY:	DATE: 09/24/2008
FILE NO. 08-032	APPROVED BY: FCC	FIGURE: 4



CONE PENETRATION TEST RESULTS FOR PC-1R

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

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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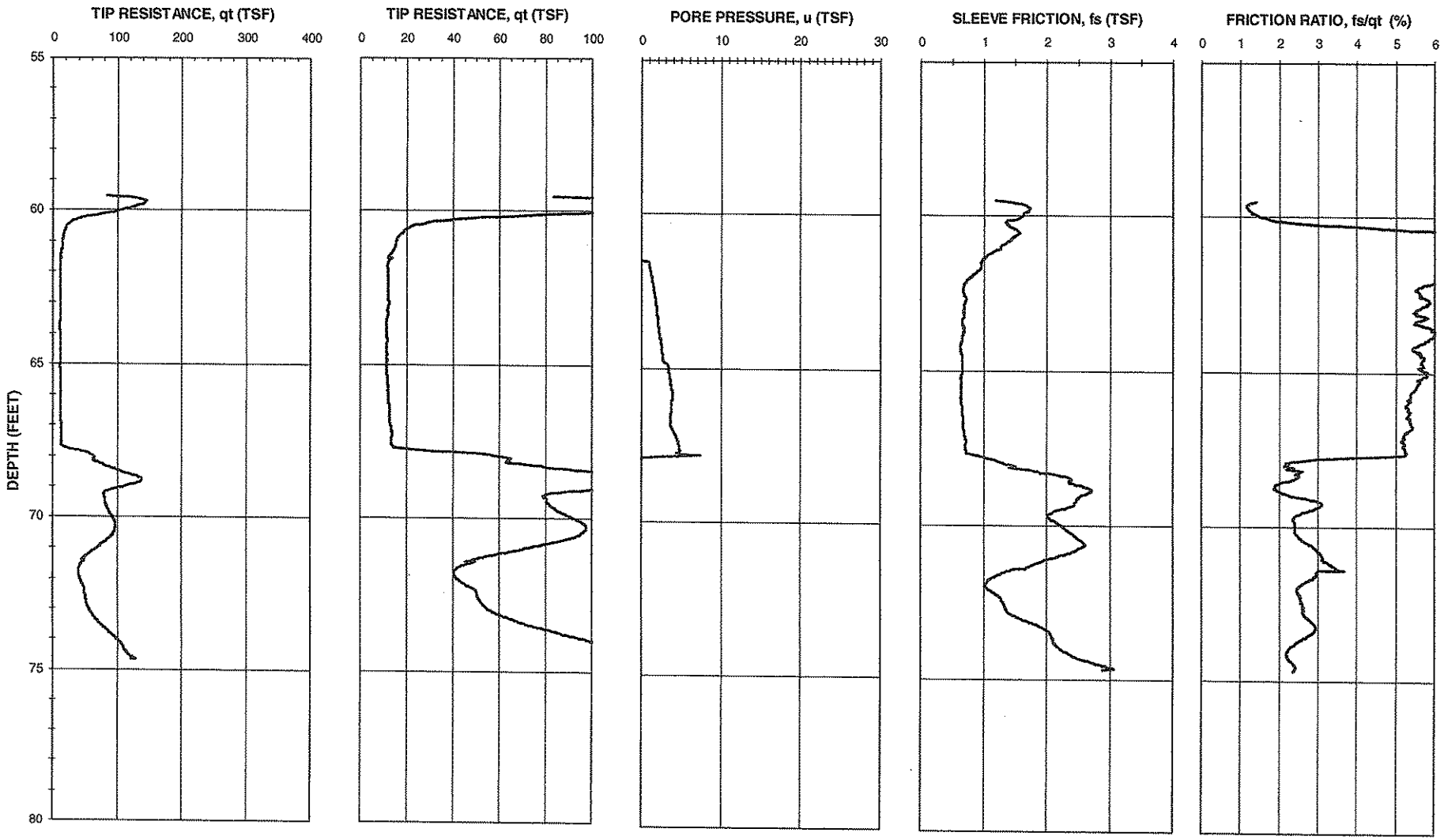
FILE NO. 08-032	APPROVED BY: FKC	FIGURE: 5



CONE PENETRATION TEST RESULTS FOR PC-3H

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Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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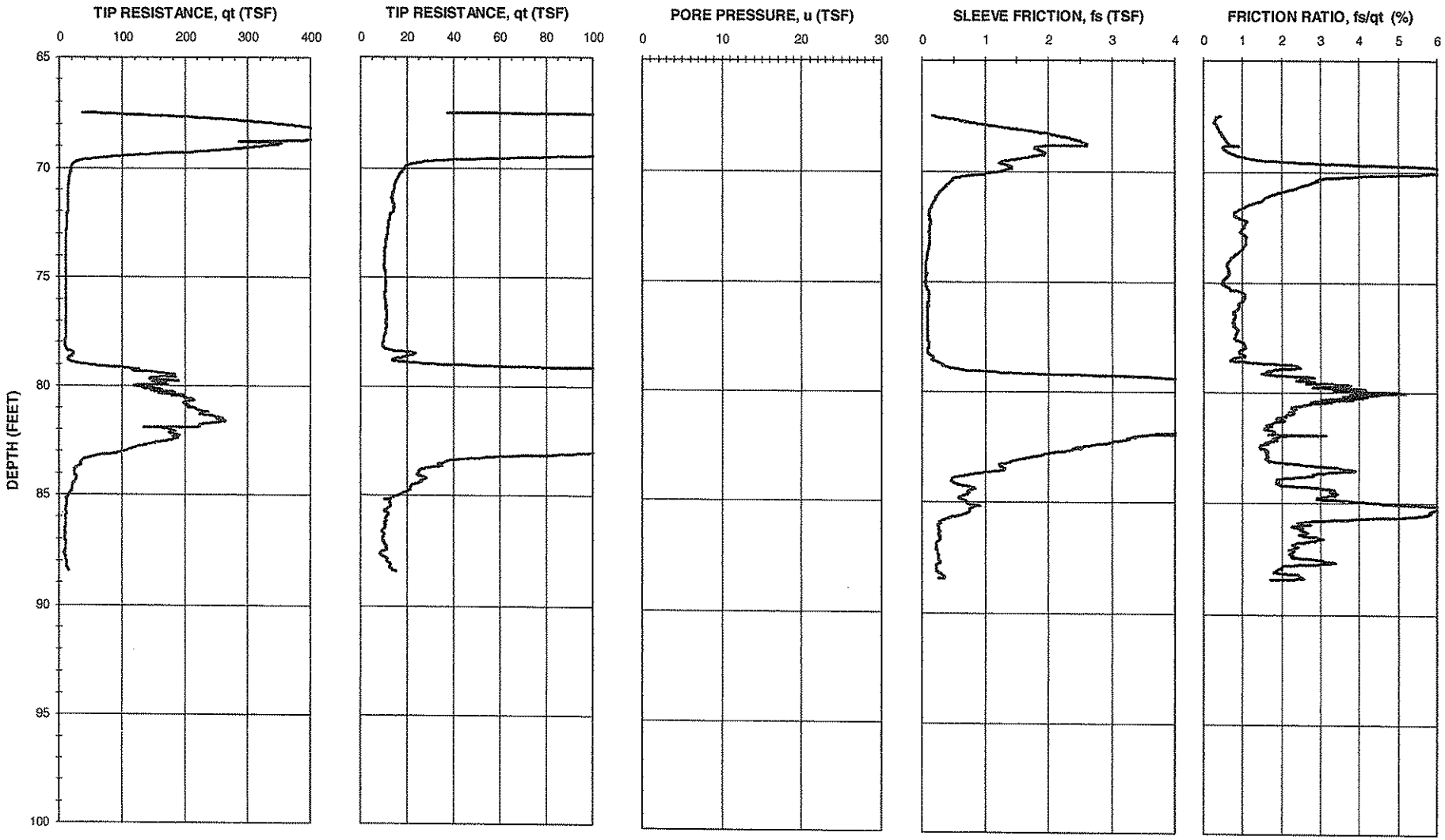

CONE PENETRATION TEST RESULTS FOR PC-4I

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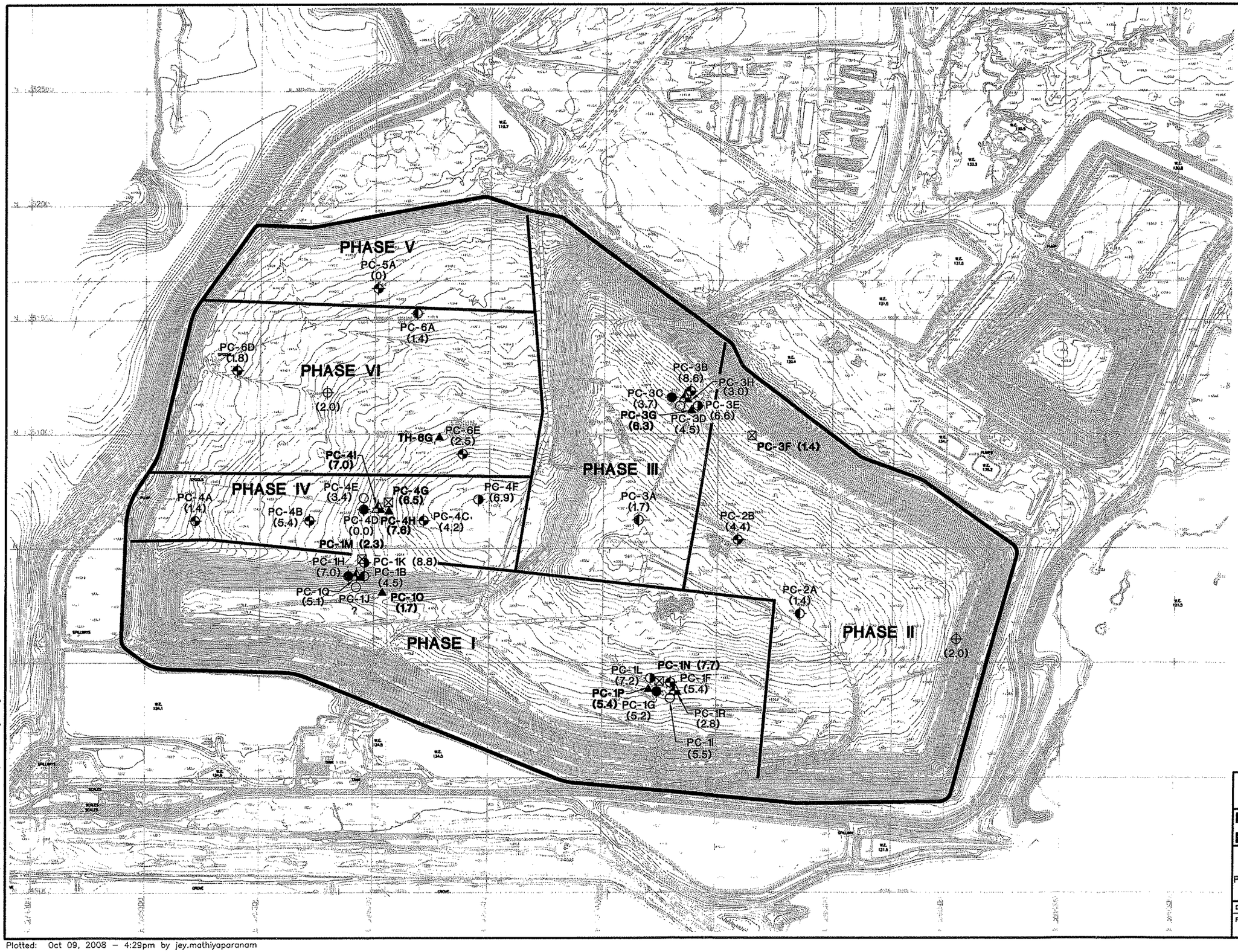
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: JM	CHECKED BY:	DATE: 09/24/2008
FILE NO. 08-032	APPROVED BY: FICC	FIGURE: 7



NOTE: THE TOPOGRAPHY IS BASED ON PHOTOGRAMMETRY FROM AN AERIAL PHOTOGRAPH DATED JULY 25, 2006.

LEGEND

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- ◐ ARDAMAN (2001)
- ◈ ARDAMAN (2002)
- ● ARDAMAN (2003)
- ○ ARDAMAN (2004)
- ◑ ARDAMAN (2005)
- ⊠ TEST SITES IN 2006
- ▲ TEST SITES IN 2007
- ⚠ TEST SITES IN 2008
- **(2.0)** PIEZOMETRIC LEVEL ABOVE TOP OF CLAY IN FEET

LEACHATE LEVELS ON TOP OF PHOSPHATIC CLAY

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Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH 2007 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: CD	CHECKED BY: MJT	DATE: 04/11/07
FILE NO. 08-212	APPROVED BY: PLCC	FIGURE: 8

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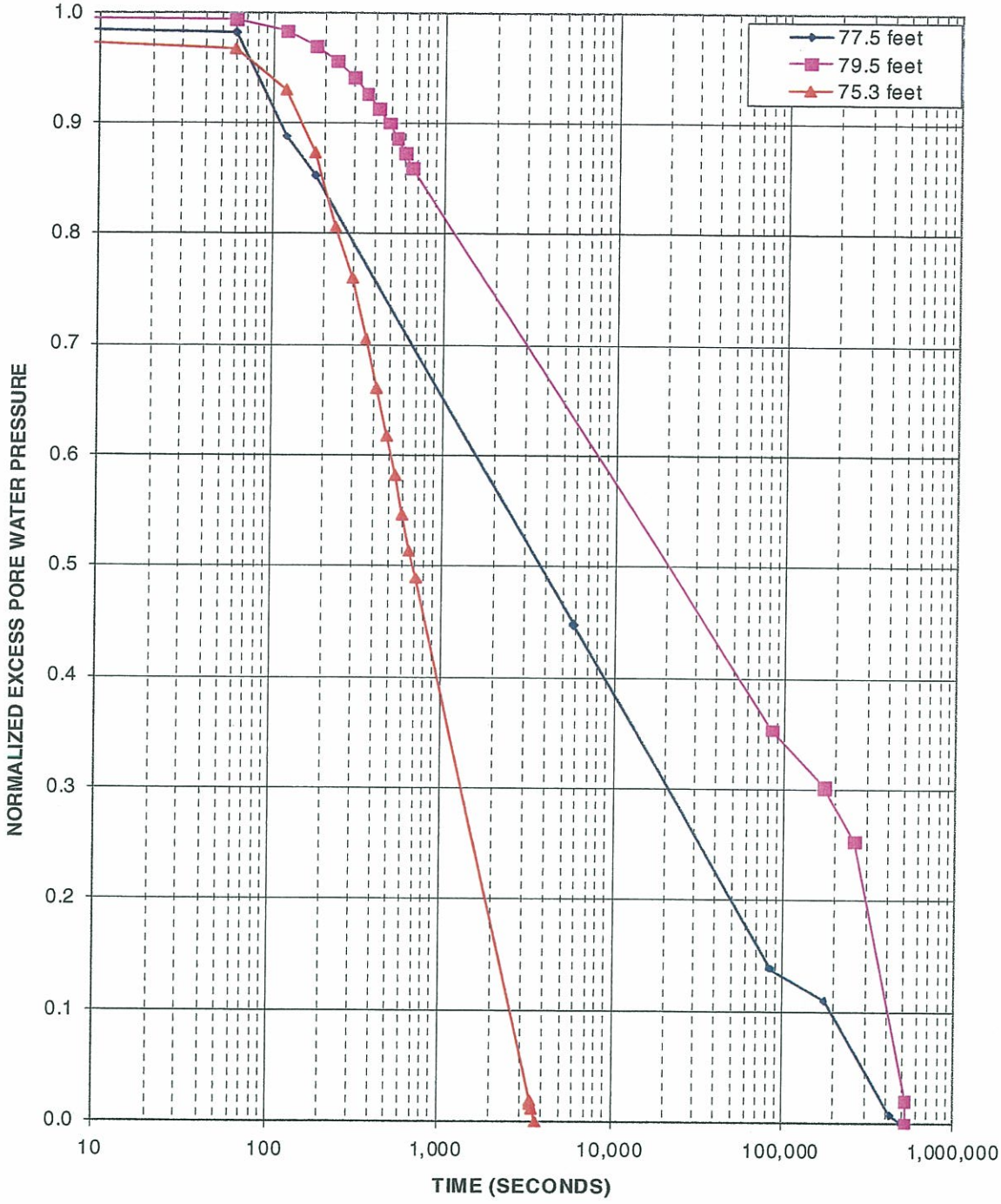

EXCESS PORE WATER PRESSURE DISSIPATION AT PC-1Q

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GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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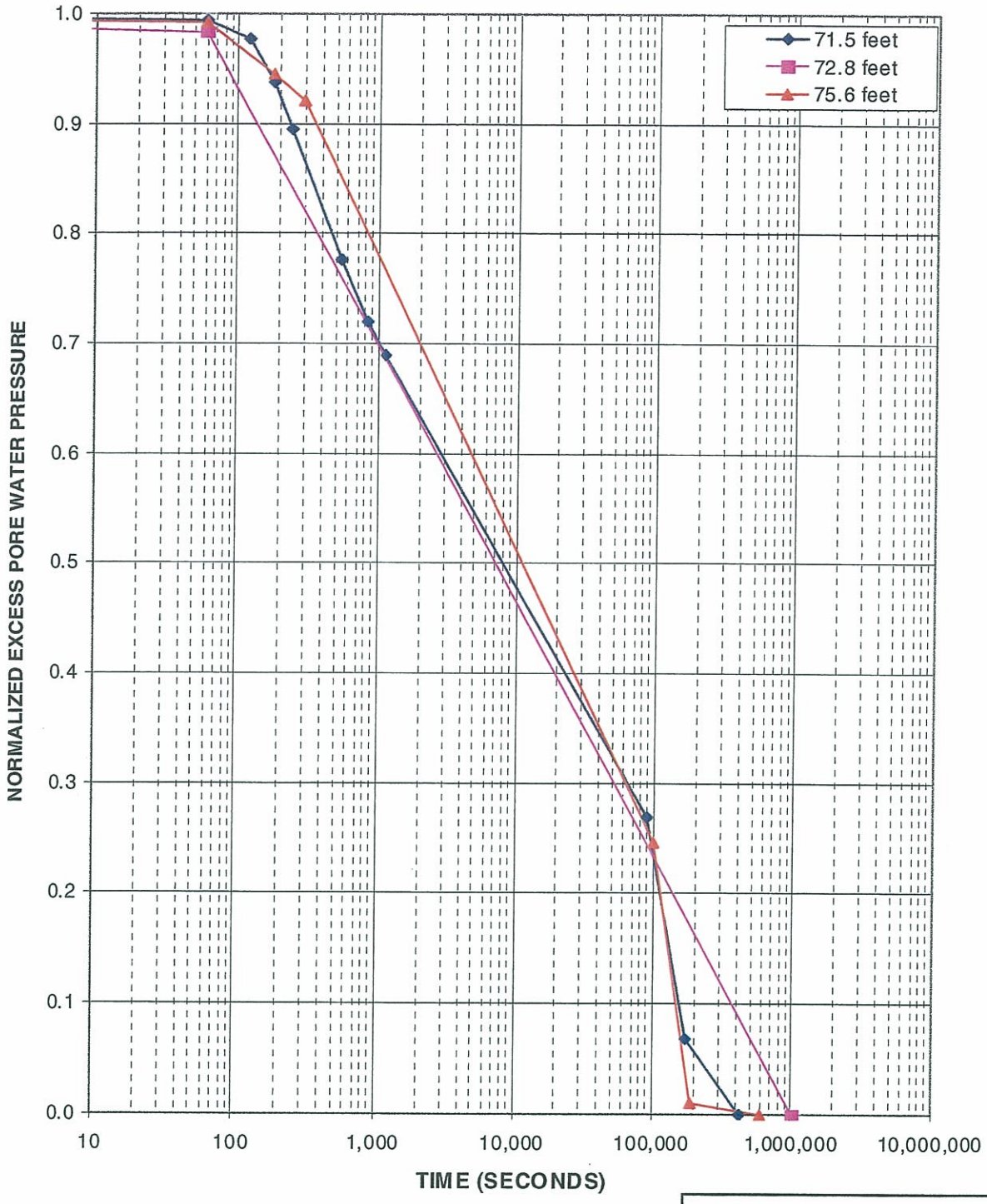

EXCESS PORE WATER PRESSURE DISSIPATION AT PC-1R

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Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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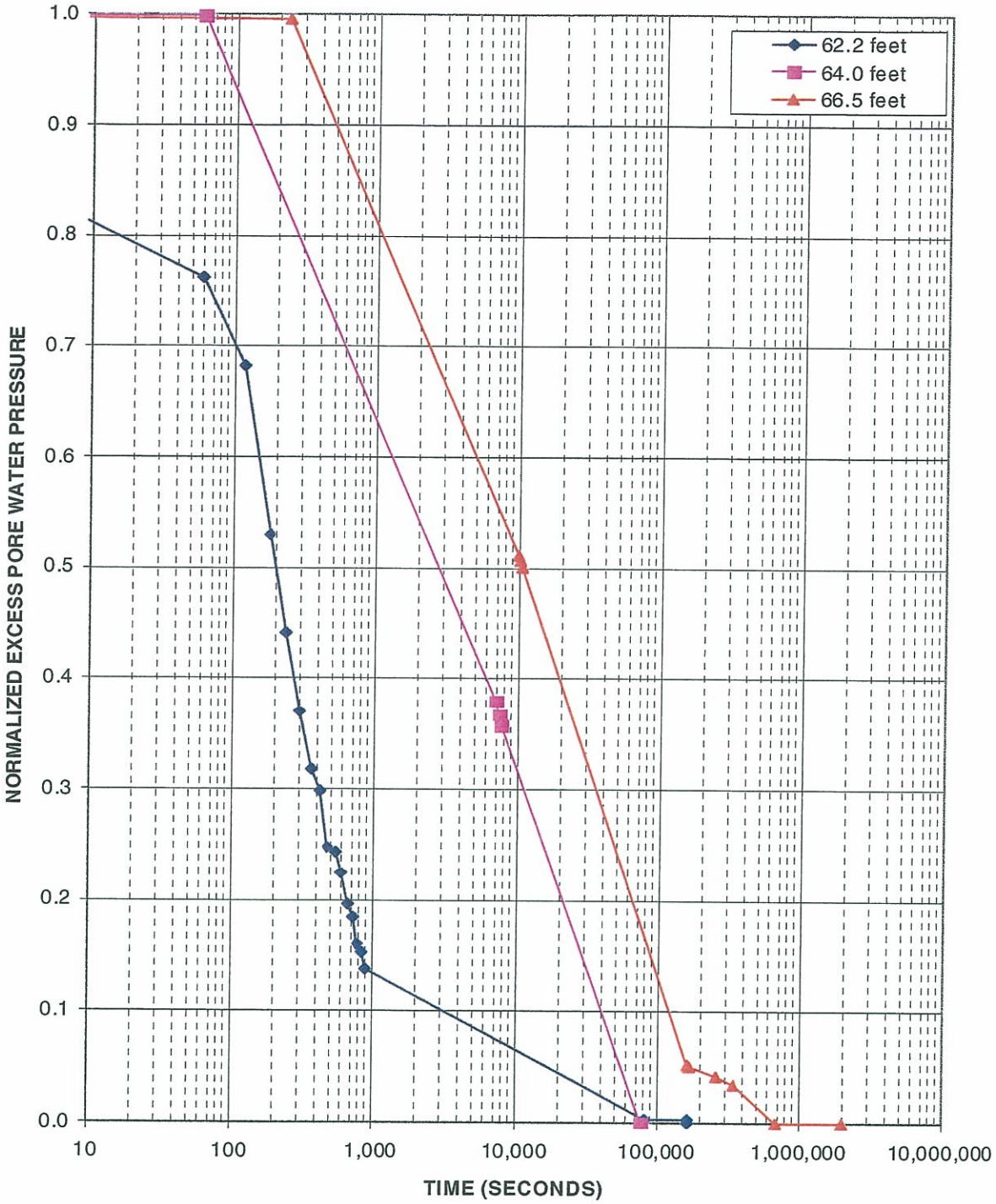
FILE NO. 08-032	APPROVED BY: FEC	FIGURE: 10



PORE WATER PRESSURE DISSIPATION AT PC-3H

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Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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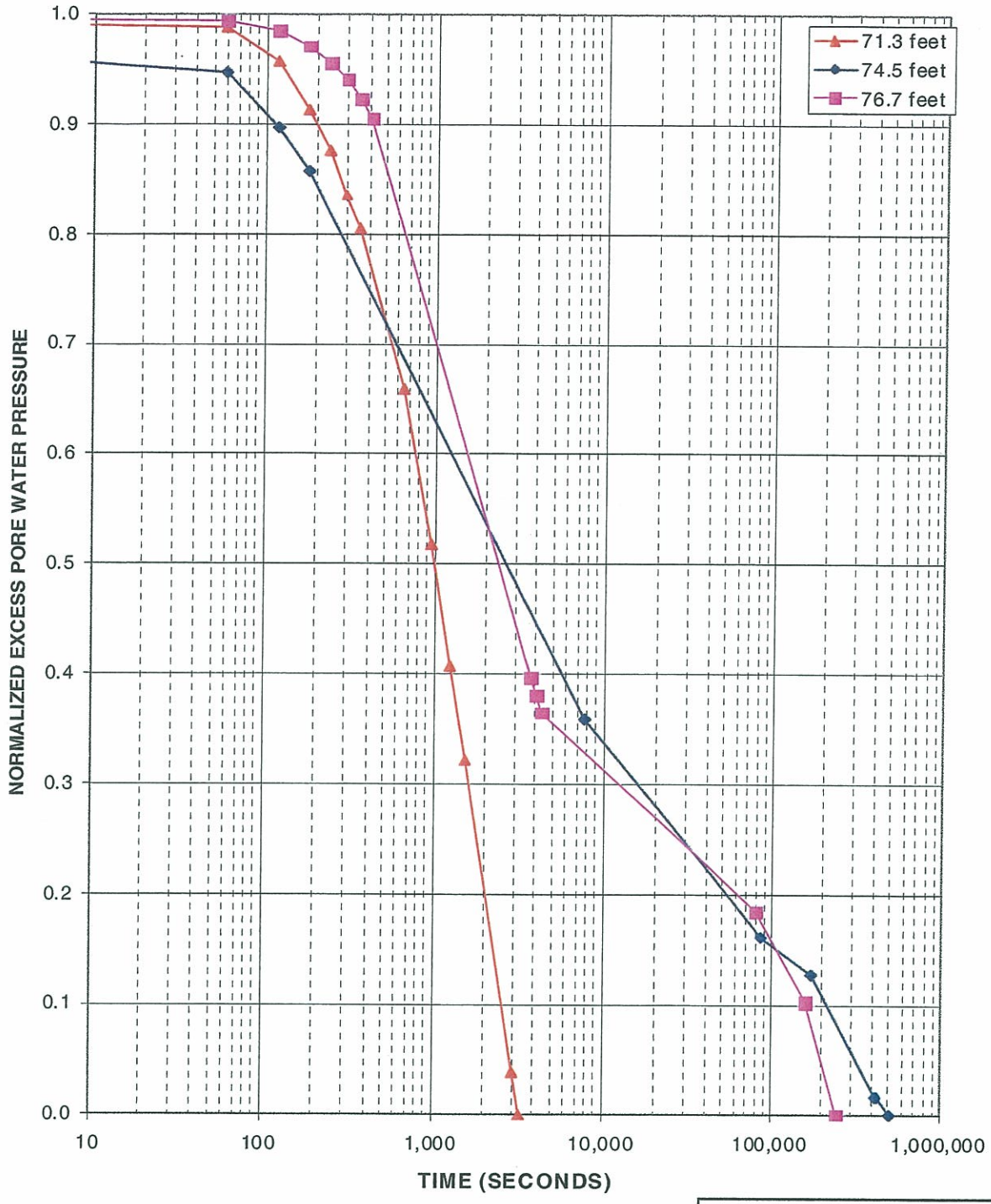
FILE NO. 08-032	APPROVED BY:	FIGURE: 11



EXCESS PORE WATER PRESSURE DISSIPATION AT PC-4I

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GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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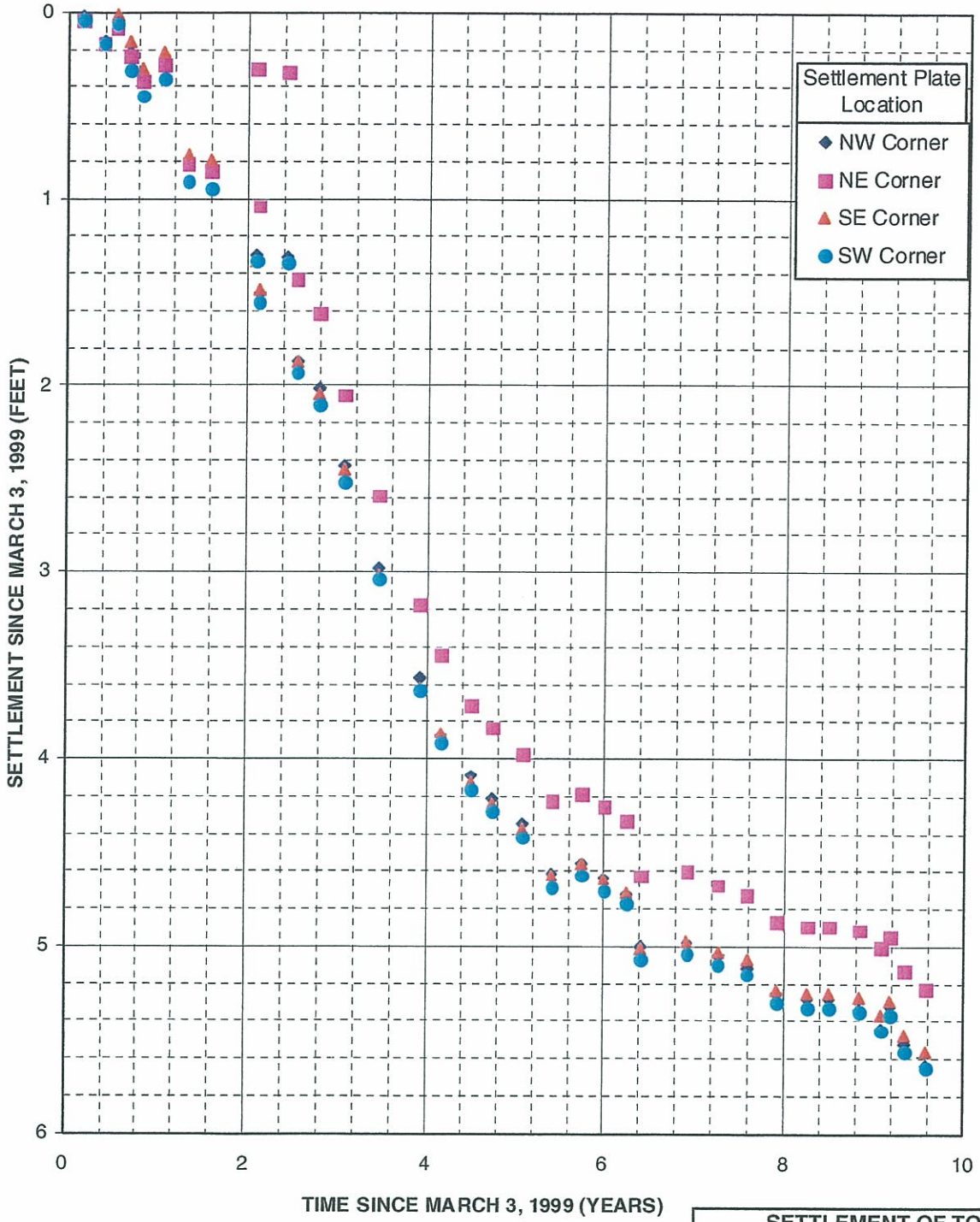
FILE NO. 08-032	APPROVED BY: FCC	FIGURE: 12



SETTLEMENT OF TOP OF PHOSPHATIC CLAY AT PUMP STATION B SUMP

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GEOTECHNICAL STUDY ASSOCIATED WITH 2008 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: JM	CHECKED BY: M.J	DATE: 09/24/2008
FILE NO. 08-032	APPROVED BY: FKC	FIGURE: 13