2006

Geotechnical Study Associated with
2006 Annual Monitoring of the Phosphatic Clay
Liner Beneath the Southeast Landfill
in Hillsborough County, Florida

April 13, 2006

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Geotechnical, Environmental and
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April 13, 2006
File Number 05-072

Jones Edmunds
324 South Hyde Park Avenue
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Attention: Mr. Richard A. Siemering, P.E.

Subject: Geotechnical Study Associated with 2006 Annual Monitoring of the Phosphatic Clay Liner Beneath the Southeast Landfill in Hillsborough County, Florida

Gentlemen:

As requested by Jones Edmunds, Ardaman & Associates, Inc., (Ardaman) has completed a geotechnical study associated with annual monitoring of the phosphatic clay liner beneath the Southeast Landfill 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 a number of piezocone soundings and pore water pressure measurements had previously been performed by Ardaman and Madrid Engineering Group, Inc., (Madrid) in 2001/2002: (i) PC-1B and PC-1F in the Phase I area; (ii) PC-4B and PC-4C in the Phase IV area; (iii) PC-3 and PC-3B in the Phase III area; and (iv) PC-1F in the Phase I area. Specifically, the permit condition required 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, and 2005 in accordance with permit requirements were documented in Ardaman reports dated April 12, 2003, April 15, 2004, and May 27, 2005, respectively. This report contains the results of the piezocone soundings and pore water pressure measurements obtained by Ardaman in February and March 2006, and presents our interpretation of the test data.

Site Location

The Southeast Landfill 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 Southeast Landfill 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 was deposited within the settling area for a number of years during the mining operation.

A comprehensive geotechnical study was conducted by Ardaman between 1981 and 1983 to characterize the phosphatic clay deposit and to evaluate the feasibility of constructing a landfill within the waste clay settling area. Results from 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 such 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 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 clay should be pre-loaded prior to placement of the first lift of refuse. A diagram that shows the original thickness of the phosphatic clay within the settling area, as reproduced from the 1983 Ardaman report, is shown in Figure 2. As shown, the phosphatic clay deposit had an original thickness that varied between 4 and 18 feet.

Another comprehensive geotechnical study was completed by Ardaman in 1994 in association with operation permit renewal for the Southeast Landfill. Results from 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, and whether the 7-year waiting period for placements of successive refuse lifts in the landfill should be modified. Results from 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. In that report, Ardaman concluded that:

- 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 minimum waiting period of 7 years 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 1.5×10^{-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 Southeast Landfill 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 on July 5, 2005. We understand that landfilling has occurred in the Phase II and III areas subsequent to the flight date of the aerial photograph.

As part of our scope of work for the annual monitoring program stipulated in Specific Condition No. 16f of the FDEP Permit No. 35435-006-SO, Ardaman performed piezocone soundings and pore water pressure measurements at four test sites within the Phase I, III, and IV areas near test sites where the annual monitoring program had previously been performed by Ardaman in 2001 through 2005.

Four test sites for the 2006 annual monitoring program were selected by Jones Edmonds for performance of piezocone soundings and installation of piezoprobes.

(i) PC-1M in the western portion of the Phase I area
(ii) PC-1N in the eastern portion of the Phase I area
(iii) PC-3F in the Phase II area slightly to the east of the previous test sites in the Phase III area
(iv) PC-4G 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, and 2005 annual monitoring programs. As shown in Figure 3, PC-1M was located in close proximity to the previous PC-1B, PC-1H, PC-1J, and PC-1K test site locations, and PC-1N was selected adjacent to the previous PC-1, PC-1F, PC-1G, PC-1GA, PC-1I, and PC-1L test site locations. PC-3F was located approximately 250 feet southeast of the PC-3E test site location in 2005. PC-4G was selected between the PC-4E test site location in 2004 and the PC-4F test site location in 2005. The current field work and testing were performed by Ardaman in February and March 2006.

The surveyed coordinates and ground surface elevations at the current test site locations, as provided by Jones Edmonds, 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 (NGVD29). The elevation data used in the previous geotechnical studies were also based on NGVD29.

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 performance of piezocone soundings by a cone penetrometer rig from the drainage sand layer, through the phosphatic clay layer, and into the underlying natural soils. The objective of the first stage of work was to determine the depths to the top and bottom of the 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 documented the pore water pressures within the sandy soils above and below the phosphatic clay layer based on the piezocone data. The second stage of our work involved drilling of additional test holes through the refuse adjacent to the piezocone sounding locations, installation of 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.

Because methane was expected to be encountered and organic compounds might be encountered during drilling through the refuse/ash, the drilling program was performed in accordance with the health and safety plan developed by Ardaman for this project. Continuous air monitoring by an Ardaman technician with a landfill gas/methane detection meter and organic vapor analyzer was performed during 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 are 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. Prior to pushing of the piezocone 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-1M, PC-1N, PC-3F, and PC-4G) 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 are generally indicative of clayey soil types, 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

Based on results of the auger borings and piezocone soundings, the refuse/ash thicknesses at the four test sites (i.e., PC-1M, PC-1N, PC-3F, and PC-4G) in February 2006 ranged from approximately 53 feet at PC-3F to 70 feet at PC-1N, assuming that the drainage sand is 2 feet thick.

PC-1M was located in the western portion of the Phase I area. The refuse/ash thickness at PC-1M in February 2006 was estimated to be 53 feet, compared to 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 reason for the variation in refuse thickness was because the test sites were not at identical locations. PC-1K was located approximately 110 feet northwest of PC-1J and at the toe of slope between the Phase I and IV areas. The land surface elevation at PC-1K was approximately 14 feet lower than the land surface elevation at PC-1J. PC-1M is located approximately 62 feet northwest of the PC-1K test location and the land surface elevation is less than 1.5 feet higher at PC-1M than at PC-1K.

PC-1N was located in the eastern portion of the Phase I area. The refuse/ash thickness at PC-N was estimated to be 70 feet in February 2006 compared to a refuse/ash thickness of 65 to 70 feet at PC-1L in March 2005 and a thickness of 50 feet at PC-1I in March 2004. PC-1N has approximately the same land surface elevation as PC-1L but the land surface elevation at PC-1N and PC-1L is more than 15 feet higher than the land surface at the other nearby test locations at the time of testing.

The refuse/ash thickness at PC-3F in February 2006 was approximately 53 feet compared to a refuse/ash thickness at PC-3E in March 2005 of approximately 35 feet. The difference in refuse/ash thickness is accounted for by placement of refuse/ash creating a land surface elevation approximately 19.5 feet higher at PC-3F than at PC-3E in 2005.

The refuse/ash thickness at PC-4G during the current field program was approximately 55 feet. The refuse/ash thickness at PC-4F in March 2005 was slightly greater than 60 feet, compared to a refuse thickness of slightly greater than 50 feet at PC-4E in March 2004. The land surface elevation at PC-4G is roughly 10 feet lower than the elevation of PC-4F in March 2005. We understand that some filling short of a full 20-foot lift occurred in the area near these test site locations in March/April 2003.

Filling Sequence

Since 2003, filling in the landfill has generally proceeded counterclockwise from Phase IV through Phase I from west to east and from there to Phase II and Phase III. In the Phase IV area, no full lift of refuse/ash has been placed since May 1995. Approximately 13 feet of refuse/ash was placed in the vicinity of PC-4F in March/April 2003 (i.e., just prior to filling of the Phase I area) as part of the grading transition from the refuse lift placed in the adjacent Phase VI area that was completed in March 2003. Filling occurred in the vicinity of PC-1K around June 2003. The filling sequence proceeded to the eastern part of Phase I between March 2004 and March 2005. Filling occurred in the vicinity of PC-1L around July 2004. A comparison of the surveyed data in the Phase I area indicated that the thickness of the refuse lift was approximately 16 to 17 feet. The previous refuse/ash lift in the Phase I area was completed in August 1997. During our previous field program in March 2005, refuse/ash was being placed in the southern part of the Phase II area. In February 2006, refuse/ash was being placed in the

Phase II and Phase III areas near our test site location PC-3G. Placement of the previous lift in the Phase III area began in December 1990 and ended in June 1994.

Elevations and Thicknesses of Phosphatic Clay

Prior to landfill construction, the top surface of the waste phosphatic clay within the former settling area was documented to have typical elevations in the range of +121 to +123 feet (NGVD) and the bottom of the phosphatic clay reportedly occurred at typical elevations ranging from +103 to +117 feet (NGVD). As indicated previously, the original thickness of the phosphatic clay ranged from 4 to 18 feet.

Based on results of the piezocone soundings, the top and bottom elevations of the phosphatic clay and the phosphatic clay thicknesses encountered at the four test sites are summarized in Table 2. As shown, the top elevations of the phosphatic clay ranged from a low of +109.3 feet (NGVD) at PC-1M to a high of +119.0 feet (NGVD) at PC-3F. The bottom elevations of the phosphatic clay layer ranged from a low of +101.3 feet (NGVD) at PC-1M to a high of +112.5 feet (NGVD) at PC-1N.

At the location of PC-1M, the top and bottom of the phosphatic clay layer were encountered at approximately +109.3 and +101.3 feet (NGVD), respectively, for a clay thickness of about 8 feet. This thickness was close to the clay thickness (7.4 feet) documented at PC-1K in March 2005. As indicated previously, approximately 17 feet of refuse was placed south of PC-1K and PC-1M in June 2003.

The piezocone sounding for PC-1M indicates a material below the phosphatic clay with higher pore water pressure and tip resistance than the phosphatic clay. A similar layer was evident in the piezocone sounding for PC-1K in 2005. During the annual monitoring in 2001, auger borings were performed within 100 feet of piezocone PC-1B to confirm the results of the piezocone sounding. Soil samples recovered from below the phosphatic clay indicated the presence of a layer of dark brown peat that occurred at a depth of 52.5 to 57.5 feet below land surface. The peat deposit was underlain by a brown fine sand soil. The peat deposit around PC-1B was projected to have a maximum radius of approximately 200 feet. PC-1M was located approximately 75 feet from PC-1B. The peat deposit is expected to have a much higher coefficient of consolidation than the phosphatic clay and to have fully consolidated by this time.

Based on the piezocone sounding performed at the location of PC-1N, the phosphatic clay had top and bottom elevations of +116.4 and +112.5 feet (NGVD), respectively, for a phosphatic clay thickness of 3.9 feet, which is very close to the thickness (4.0 feet) documented at the nearby PC-1L in 2005. As indicated previously, approximately 17 feet of refuse was placed in the vicinity of PC-1L and PC-1N in July 2004.

The piezocone sounding performed at the location of PC-3F revealed the top elevation of the phosphatic clay at +119.0 feet (NGVD) and the bottom elevation at +111.2 feet (NGVD), for a clay thickness of 7.8 feet, The piezocone sounding performed at the location of PC-3E in 2005 revealed the top elevation of the phosphatic clay at +118.7 feet (NGVD) and the bottom elevation at +110.3 feet (NGVD), for a clay thickness of 8.3 feet. Because PC-3F is approximately 250 feet southwest of PC-3E, the clay thicknesses should not necessarily be identical. In addition, refuse/ash has been placed in this area subsequent to the March 2005 testing. The thickness of the phosphatic clay in this area has decreased from 9.3 feet at PC-3A in 2001 to 7.8 feet at PC-3F in 2006.

Based on the piezocone sounding performed at the location of PC-4G, the top and bottom elevations of the phosphatic clay were documented at +110.5 and +103.3 feet (NGVD), for a clay thickness of 7.2 feet, which is almost identical to the thickness (6.9 feet) documented at the nearby PC-4F test site in 2005.

In summary, the phosphatic clay elevations and thicknesses documented at the four test sites in 2006 (i.e., PC-1M, PC-1N, PC-3F, and PC-4G) indicated that there were changes in clay thickness of 0.1 to 0.6 feet from March 2005 to February 2006.

Piezometric Elevations on Top of Phosphatic Clay

The piezometric heads in the drainage sand layer on top of the phosphatic clay could 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 was held stationary at selected depths 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 short duration. The pore pressures were monitored for several minutes to make sure that the final readings represented the stabilized pore pressures at the selected depths.

The boring for piezocone PC-1M penetrated slightly into the phosphatic clay layer because the drainage sand layer was too dense for cone penetration. Therefore, no pore water pressure readings could be taken in the sand. The piezometeric elevation in the drainage sand at the top of the clay was approximated as the pore water pressure at the start of the piezocone sounding.

Based on the piezocone soundings performed at the four test sites, the 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 on the same figure for comparison.

As shown in Table 3 and Figure 8, results of the piezocone soundings indicated that the piezometric heads on top of the phosphatic clay at the four test site locations ranged from 1.4 to 7.7 feet. The highest piezometric elevation of +124.1 feet (NGVD), which was 7.7 feet above the top of the phosphatic clay, was observed at PC-1N. The piezometric heads in the drainage sand on top of the phosphatic clay at PC-1M, PC-3F, and PC-4G were measured to be 2.3, 1.4, and 6.5 feet above the top of the phosphatic clay, respectively. The piezometric heads in the drainage sand layer in February 2006 ranged from 0.5 feet higher to 5.5 feet lower than the previous readings obtained in March 2005.

Piezometric Elevations Below Phosphatic Clay

Based on the piezocone sounding results, the piezometric elevations in the natural soils below the phosphatic clay are summarized in Table 4. As shown, the piezometric elevations at the test site locations were documented at approximately +117 to +123 feet (NGVD) versus approximately +118 to +122 feet (NGVD) in March 2005. The piezometric elevations below the phosphatic clay at PC-1M and PC-4G were within 1 to 2 feet of the elevations at the corresponding locations PC-1K and PC-4F in March 2005. The piezometric elevations below the phosphatic clay at PC-1N and PC-3F were 4 feet higher and 5 feet lower, respectively, than the elevations at test site locations PC-1L and PC-3E in March 2005.

Piezoprobe Tests within Phosphatic Clay

The piezoprobe tests were performed by installing piezoprobes to pre-selected 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.

The dissipation of excess pore water pressures generated by piezoprobe penetrations are presented in Figures 9 to 12 for PC-1M, PC-1N, PC-3F, and PC-4G, respectively. Figures 9, 10 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. In some cases, the pore water pressure first increased to a peak level before it began to dissipate. 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.

At the location of PC-1M, with approximately 8 feet of phosphatic clay, piezoprobe tests were performed at four different depths to measure the pore water pressures near the top, middle, and bottom of the phosphatic clay layer. At the location of PC-1N, with approximately 4 feet of phosphatic clay, piezoprobe tests were performed at two different depths to measure the pore water pressures near the top and middle of the phosphatic clay layer. At the location of PC-3F, with approximately 8 feet of phosphatic clay, piezoprobe tests were performed at three different depths. At the location of PC-4G, with approximately 7 feet of phosphatic clay, piezoprobe tests were performed at two different depths to measure the pore water pressures straddling the middle of the phosphatic clay layer. Results from the piezoprobe tests are summarized in Table 5.

At the location of PC-1M, the equilibrium piezometric elevations for the four piezoprobes installed with tip elevations at +108.3, +106.3, +104.3, and +102.3 feet (NGVD) were documented to be approximately +112.7, +126.3, +130.7, and +126.1 feet (NGVD), respectively. The excess pore water pressures within the phosphatic clay layer at the tip elevations were estimated to be 0.0 (i.e., no excess pore water pressure), 10.4, 12.0, and 4.5 feet of water, respectively. Excess pore water pressures at PC-1K in 2005 ranged from 10.4 to 14.8 feet of water. As indicated previously, approximately 17 feet of refuse was placed in the vicinity of PC-1M in June 2003, which would have caused an excess pore water pressure of 22 feet of water, based on a refuse/ash weight of 80 lb/ft^3, prior to dissipation.

At the location of PC-1N, two piezoprobe tests were performed with the piezoprobe tip elevations at +116.4 and +114.9 feet (NGVD). The piezoprobe with the tip elevations at +116.4 feet (NGVD) was at the top elevation of the phosphatic clay. The piezoprobe readings indicated no excess pore water pressure. The piezometric elevation for the tip elevation at +114.9 feet (NGVD) was documented to be +130.9 feet (NGVD), corresponding to an excess pore water pressure that is equivalent to approximately 7.4 feet of water, which compares to 7.1 feet of water at the location of PC-1L in 2005. As indicated previously, approximately 17 feet of refuse was placed in the vicinity of PC-1N in July 2004. Because the phosphatic clay thickness at PC-1N (3.9 feet) was less than that at PC-1M (8.0 feet), the excess pore water pressure from refuse/ash loading would be expected to dissipate at a faster rate.

At the location of PC-3F, piezoprobe tests were performed with the piezoprobe tip elevations at +118.2, +115.4, and +114.2 feet (NGVD)). Results of the piezoprobe tests performed with the piezoprobe tip elevations at +115.4, and +114.2 feet (NGVD) were erratic. Accordingly, we were not able to interpret the piezoprobe data to obtain meaningful results. The piezoprobe test at elevation +118.2 feet (NGVD), near the top of the phosphatic clay, indicated an excess pore water pressure of 3.3 feet of water. Additional loading from the current lift is expected to cause higher excess pore water pressures toward the middle of the phosphatic clay layer. At test location PC-3E in 2005 excess pore water pressures of 0.2 to 1.8 feet of water were measured. This indicated that excess pore water pressures from the previous lift had largely dissipated.

At the location of PC-4G, two piezoprobe tests were performed within the phosphatic clay layer with the piezoprobe tip elevations at +107.8 and +105.8 feet (NGVD). The equilibrium piezometric elevations were at +132.2 and +133.3 feet (NGVD), respectively. The excess pore water pressures near the middle of the phosphatic clay were equivalent to approximately 15 feet of water. High excess pore water pressures were also observed in the Phase IV area in previous piezoprobe measurements in 2003, 2004, and 2005. The high excess pore water pressures could be attributed to refuse/ash loading in the Phase IV area. As noted previously, approximately 13 feet of refuse was added in the vicinity of PC-4F in April 2003. Excess pore water pressure decreased substantially from 19 to 32 feet of water in 2005 to approximately 15 feet of water in 2006.

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 at the Southeast Landfill was originally based on a design vertical coefficient of consolidation, c_v, of 1.5×10^{-4} cm^2/sec.

Based on the rates of excess pore water pressure dissipation shown in Figures 9 through 12, the *in situ* horizontal and vertical coefficients of consolidation, c_h and 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, $c_v = 0.05 \times c_h$. The results of the calculations are presented in Table 6.

The *in situ* vertical coefficients of consolidation from the piezoprobe measurements near the tops of PC-1M and PC-1N, and for all depths in PC-4G are equal to or greater than the design vertical coefficient of consolidation. The average c_v value for these points is 2.1×10^{-4} cm^2/sec. However, the *in situ* vertical coefficients of consolidation from the piezoprobe measurements at greater depths in the phosphatic clay layer at PC-1M and PC-1N had an average c_v value of 2.4×10^{-5} cm^2/sec. The PC-1M piezoprobe with tip elevations of +104.3 and +102.3 feet (NGVD) had computed c_v values that are atypically low. Based on annual monitoring data since 2001, these values are questionable. Although the vertical coefficients of consolidation documented at the middle and bottom of PC-1M and PC-1N were lower than the original design value of 1.5×10^{-4} cm^2/sec, the piezoprobe measurements in 2003 and 2004 at nearby test sites had computed vertical coefficients of consolidation that were consistent with the design value.

The piezoprobe pore water dissipation monitoring data for PC-3F indicates a c_v value of 3.5×10^{-5} to 1.8×10^{-4} cm^2/sec. In 2005, the *in situ* c_v value from the piezoprobe measurements at the top of PC-3E was 2.4×10^{-4} cm^2/sec and the mid-depth and bottom of the phosphatic clay layer at PC-3E had an average c_v value of 3.1×10^{-5} cm^2/sec. Although the vertical coefficients of consolidation documented at the middle and bottom of the phosphatic clay for PC-3E were lower than the original design value of 1.5×10^{-4} cm/sec, the piezoprobe measurements in 2003 and 2004 at nearby test sites had vertical coefficients of consolidation that were consistent with the design value.

In our 2005 report, because the c_v values of the phosphatic clay layer at PC-4F averaged 2.8×10^{-5} cm^2/sec for the three measurements at different depths and because c_v values lower than the design value were also documented at the nearby test site PC-4E in 2004, Ardaman recommended performance of additional field tests to measure the undrained shear strength of the clay or any excess pore water pressure in the phosphatic clay prior to placement of the next lift of refuse/ash in the Phase IV area. Because the c_v values for PC-4F exceed the design value and the excess pore water pressures have been reduced substantially, additional field tests are not required at this time.

Settlement plate data at the location of Pump Station B in the Phase VI area, as provided by Hillsborough County, are plotted in Figure 13. At this location, the initial thickness of the phosphatic clay in July 1997 was approximately 13 feet. The phosphatic clay was assumed to have settled 2 feet by the time the settlement observation was started in March 1999. As shown in Figure 12, the phosphatic clay has undergone an average settlement of about 4.6 feet over a period of 5.85 years since filling started on March 30, 2000. Assuming that an average degree of consolidation of 95 percent has been reached under double drainage conditions with an average consolidating thickness of 11 feet, the back-calculated c_v from the settlement observations is 1.7×10^{-4} cm^2/sec, which is consistent with the design value.

<u>Comparisons of Piezometric Heads</u>

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 (except for PC-3F) 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 and below the phosphatic clay at the PC-1M, PC-1N, PC-3F and PC-4G test site locations. Accordingly, under existing conditions, there should be no downward leachate migration or upward groundwater flow through the phosphatic clay layer at these locations.

Summary of Observations

The following key observations were made from the 2006 annual monitoring data based on the field work completed:

- The refuse/ash surface elevations increased by 19.5 feet at test location PC-3F compared to March 2005 at test location PC-3E. At test locations PC-1M and PC-1N, the existing grade remained unchanged. At test location PC-4G, the refuse/ash elevation was approximately 9.5 feet lower than test location PC-4F in 2003 because of the relative elevations of the landfill at two different locations within Phase IV.

- The top elevations of the phosphatic clay were as expected and generally consistent with the range of elevations obtained in 2005.

- The thicknesses of the phosphatic clay were also generally consistent with the data obtained in 2005. The phosphatic clay thicknesses at PC-1M, PC-1N, PC-3F, and PC-4G were found to 0.5 feet lower to 0.6 feet higher than the comparable thicknesses estimated in 2005.

- Excess pore water pressure was documented within the phosphatic clay layer at PC-1M, PC-1N, PC-3F, and PC-4G. The area around PC-1M was loaded in 2003 and 2004. The area around PC-1N was loaded in 2004 and 2005. The area around PC-4G was loaded in 2003. The area around PC-3F was loaded in 2005 and 2006. Because these areas have been loaded in 2003 through 2006, the excess pore water pressure was expected. Because the area around PC-3F has recently been loaded with the next lift of refuse/ash and we would expect high excess pore water pressures in the middle of the phosphatic clay layer.

- The piezometric heads above the phosphatic clay at the PC-1M, PC-1N, PC-3F, and PC-4G test site locations were measured to be 2.3, 7.7, 1.4, and 6.5 feet, respectively compared to piezometric heads above the phosphatic clay at the PC-1K, PC-1L, PC-3E, and PC-4F 2005 test site locations of 8.8, 7.2, 6.6, and 6.9 feet, respectively. Compared to 2005, the piezometric heads in 2006 ranged from 6.5 feet lower to 0.5 feet higher. On the average, the 2006 measurements were about 3 feet lower than the readings obtained in 2004.

- The maximum piezometric elevation within the phosphatic clay liner is higher than the piezometric elevation in the leachate collection and recovery system (LCRS) on top of the clay at the PC-1M, PC-1N, PC-3F and PC-4G test site locations. Accordingly, under existing conditions, there should be no downward leachate migration through the phosphatic clay layer at these locations..

- The piezometric elevations below the phosphatic clay were observed to be at approximately +117 to +123 feet (NGVD). These elevations are within a similar range to those observed in 2003, 2004, and 2005.

Recommendations

Excess pore water pressures at PC-1M and PC-1N were recorded but are not unexpected because of recent (2003-2005) refuse/ash placement in this area. However, because of the lower than expected coefficients of consolidation documented at different depths of the phosphatic clay layer at PC-1M and PC-1N, additional monitoring should be performed to measure the excess pore water pressure in the clay prior to placement of the next lift of refuse/ash.

Because of the calculated coefficient of consolidation less than the design value in PC-3E in 2005 and possibly at PC-3F in 2006, and limited pore water pressure dissipation data for PC-3F in 2006, the response of the Phase II and Phase III areas to placement of the refuse/ash lift that is currently taking place should be carefully monitored during the next event. There is no need to address scheduling of future lifts at this time.

Placement of the next full lift of refuse/ash may proceed in the portions of Phase IV where refuse/ash was last placed in 1995. In the portions of Phase IV where a partial lift of refuse/ash was placed in 2003, the difference between the thickness of refuse/ash placed in 2003 and the maximum recommended lift thickness of 20 feet may also be placed at this time. The 7-year waiting period for the placement of the next lift begins from the point in time of placing the last amount of waste in the previous lift.

The observed dissipation of excess pore water pressures and, therefore, the projected rate of consolidation and strength gain of the phosphatic clay liner are generally consistent with the filling schedule. In accordance with the design, any excess pore water pressure generated from a refuse lift should be dissipated prior to placement of the next lift of refuse.

Closure

This report has been prepared for the exclusive use of Jones Edmunds and the Hillsborough County Solid Waste Department for specific application to annual monitoring of the phosphatic clay liner at the Southeast Landfill 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.

Patrick A. Kennedy

Patrick A. Kennedy, P.E.
Project Manager

Francis K. Cheung

Francis K. Cheung, P.E.
Senior Project Engineer 04/13/06
Florida License No. 36382

Table 1

Test Site Locations and Elevations

Test Site	Area	Florida State Plane Coordinates		Approximate Ground Surface Elevation
		Northing (feet)	Easting (feet)	[feet (NGVD29)]
PC-1M	Phase I	1,250,453.47	595,948.13	+164.8
PC-1N	Phase I	1,249,919.35	597,250.64	+188.4
PC-3F	Phase III*	1,250,993.08	597,646.79	+174.2
PC-4G	Phase VI	1,250,701.58	596,061.77	+168.6

Coordinates and elevations are the averages of the coordinates and elevations for the piezocone and piezoprobe borings at each test site.

* PC-3F was actually located within the Phase II area and adjacent to the Phase III area.

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-1M	02/17/06	+164.8	55.5	+109.3	63.5	+101.3	8.0
	PC-1N	02/28/06	+188.4	72.0	+116.4	75.9	+112.5	3.9
Phase III	PC-3F	02/21/06	+174.2	55.2	+119.0	63.0	+111.2	7.8
Phase IV	PC-4G	02/17/06	+168.6	58.1	+110.5	65.3	+103.3	7.2

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-1M	02/17/06	+164.8	+111.6	+109.3	2.3
	PC-1N	02/28/06	+188.4	+124.1	+116.4	7.7
Phase III	PC-3F	02/21/06	+174.2	+117.7	+116.3	1.4
Phase IV	PC-4G	02/17/06	+168.6	+117.0	+110.5	6.5

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-1M	02/17/06	+164.8	+123.0
	PC-1N	02/28/06	+188.4	+122.5
Phase III	PC-3F	02/21/06	+174.2	+116.9
Phase IV	PC-4G	02/17/06	+168.6	+119.0

Table 5

Pore Water Pressure within the Phosphatic Clay

Area	Test Site	Ground Surface Elevation	Top of Clay Elevation	Bottom of Clay Elevation	Piezoprobe Tip Below Ground Surface (feet)	Elevation of Piezoprobe Tip	Stabilized Pore Water Pressure at Piezoprobe Tip After Dissipation (feet of H_2O)	Piezometric Elevation at Piezoprobe Tip Level	Piezometric Elevation on Top of Phosphatic Clay	Piezometric Elevation Below Phosphatic Clay	Excess Pore Water Pressure at Piezoprobe Tip Level (feet of H_2O)
Phase I	PC-1M	+164.8	+109.3	+101.3	56.5 58.5 60.5 62.5	+108.3 +106.3 +104.3 +102.3	4.4 20.0 26.4 23.8	+112.7 +126.3 +130.7 +126.1	+111.6	+123.0	0.0 10.4 12.0 4.5
	PC-1N	+188.4	+116.4	+112.5	72.0 73.5	+116.4 +114.9	7.7 16.0	+124.1 +130.9	+124.1	+122.5	0.0 7.4
Phase III	PC-3F	+174.2	+116.3	+111.2	56.0	+118.2	3.3	+121.5	+117.7	+116.9	3.9
Phase IV	PC-4G	+168.6	+110.5	+103.3	60.8 62.8	+107.8 +105.8	24.4 27.5	+132.2 +133.3	+117.0	+119.0	14.5 15.0

All elevations are feet (NGVD).

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 (cm^2/sec)	Vertical Coefficient of Consolidation, c_v* (cm^2/sec)
Phase I	PC-1M	PC-1M	56.5 58.5 60.5 62.5	+108.3 +106.3 +104.3 +102.3	3.0×10^{-3} 1.0×10^{-3} 4.7×10^{-5} 1.6×10^{-4}	1.5×10^{-4} 5.1×10^{-5} 2.3×10^{-6} 7.6×10^{-6}
	PC-1N	PC-1N	72.0 73.5	+116.4 +114.9	5.1×10^{-3} 6.7×10^{-4}	2.5×10^{-4} 3.3×10^{-5}
Phase III	PC-3F	PC-3F	56.0	+118.2	7.2×10^{-4} 3.7×10^{-3}	3.5×10^{-5} 1.8×10^{-4}
Phase IV	PC-4G	PC-4G	57.8 60.8 62.8	+110.8 +107.8 +105.8	5.1×10^{-3} 4.2×10^{-3} 4.0×10^{-3}	2.5×10^{-4} 2.1×10^{-4} 1.9×10^{-4}

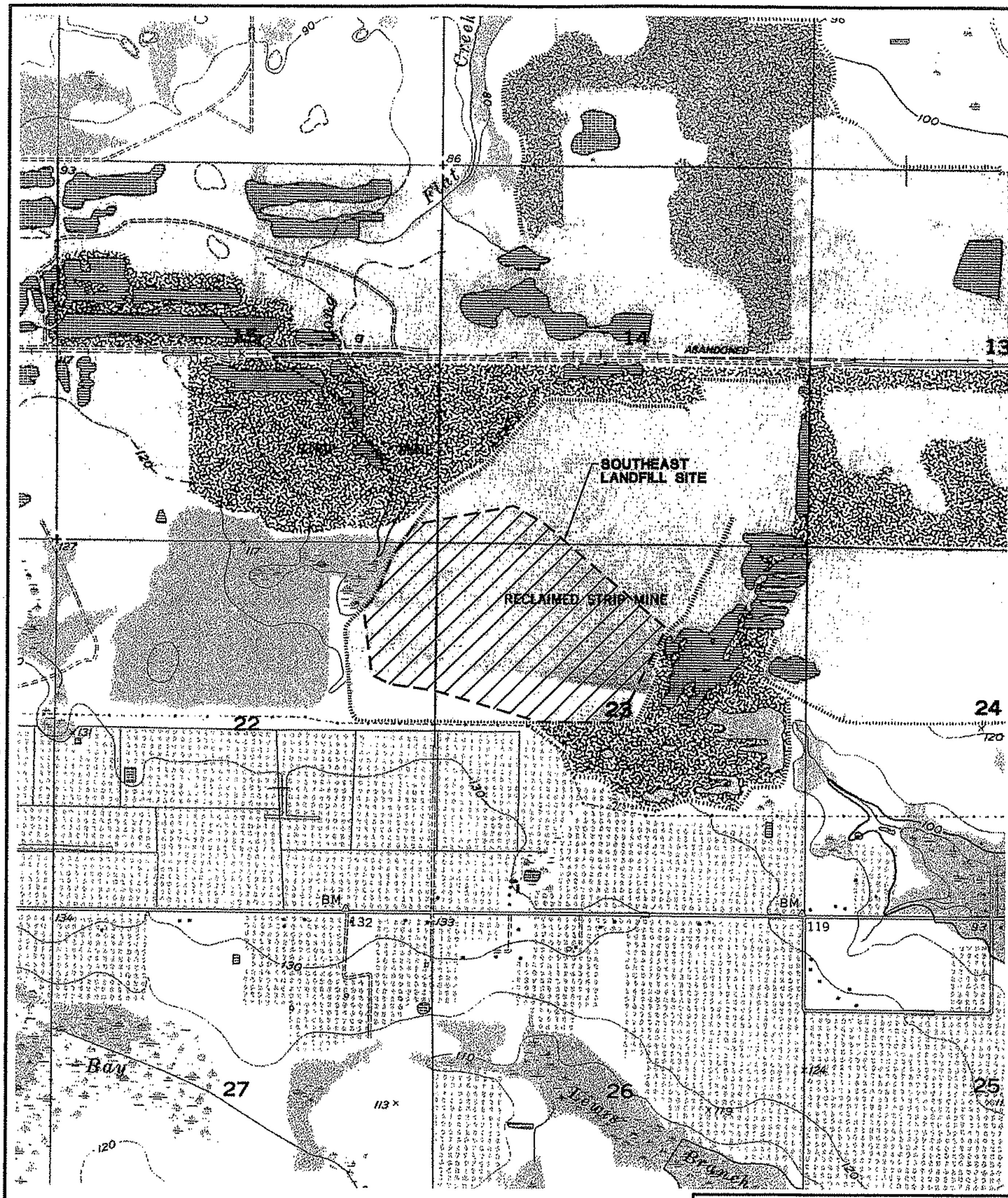
* c_v using empirical relationship recommended by Baligh and Levadoux, 1986.

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-1M	02/17/06	+164.8	+111.6	+130.7	+123.0	Within Clay
	PC-1N	02/28/06	+188.4	+124.1	+130.9	+122.5	Within Clay
Phase III	PC-3F	02/21/06	+174.2	+117.7	–	+116.9	In Middle of Clay*
Phase IV	PC-4G	02/17/06	+168.6	+117.0	+133.3	+119.0	Within Clay

*Maximum head not measured but expected to be in the middle of the clay because the area was recently loaded.



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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 ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA		
DRAWN BY: RAAR	CHECKED BY:	DATE: 03/24/05
FILE NO. 05-072	APPROVED BY: FTC	FIGURE: 1

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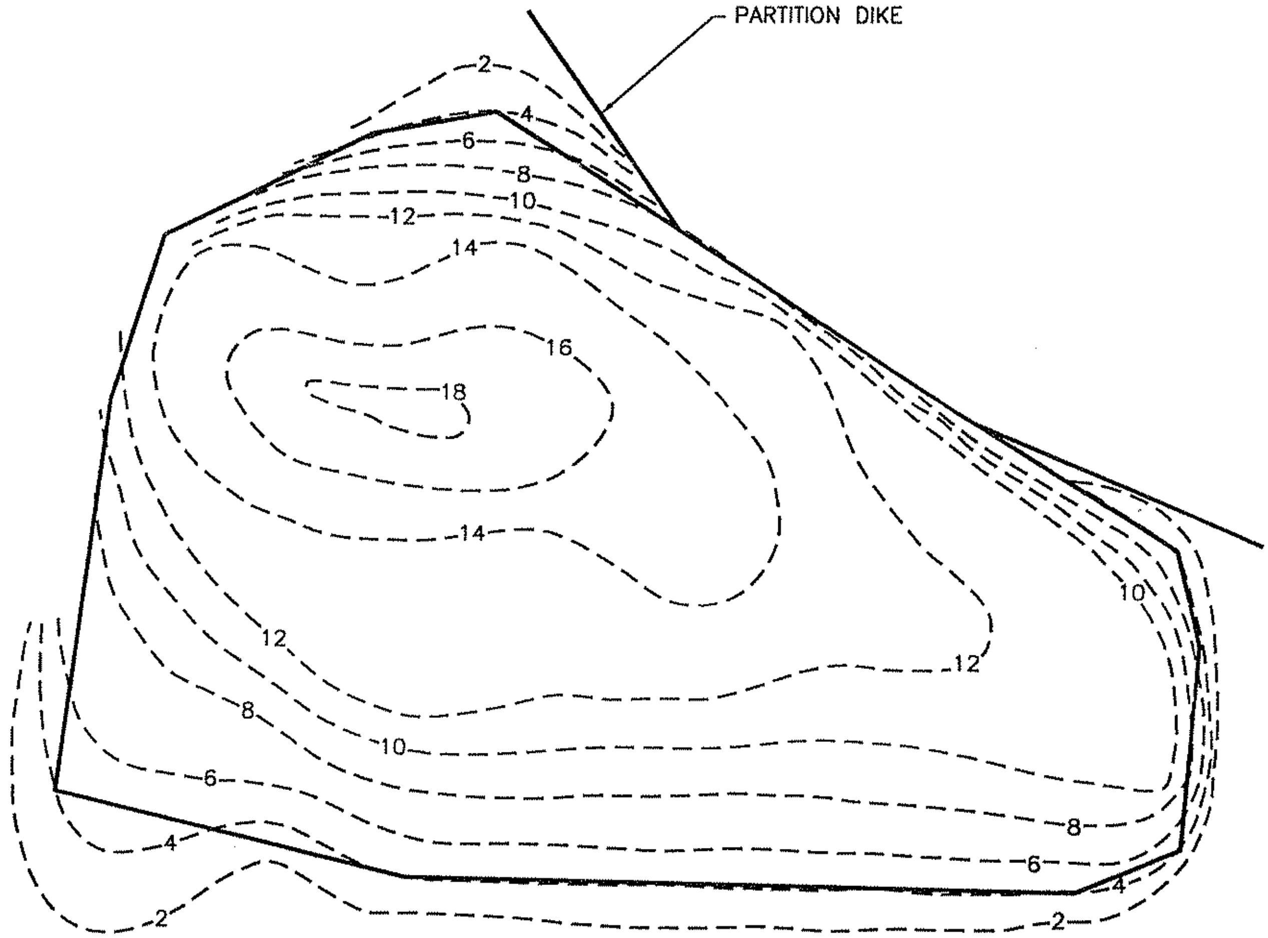

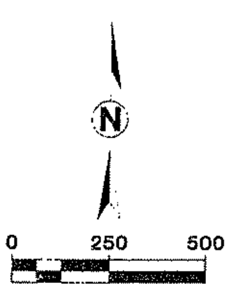

LEGEND

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 ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA		
DRAWN BY: RAAR	CHECKED BY:	DATE: 03/24/06
FILE NO. 05-072	APPROVED BY: FVC	FIGURE: 2

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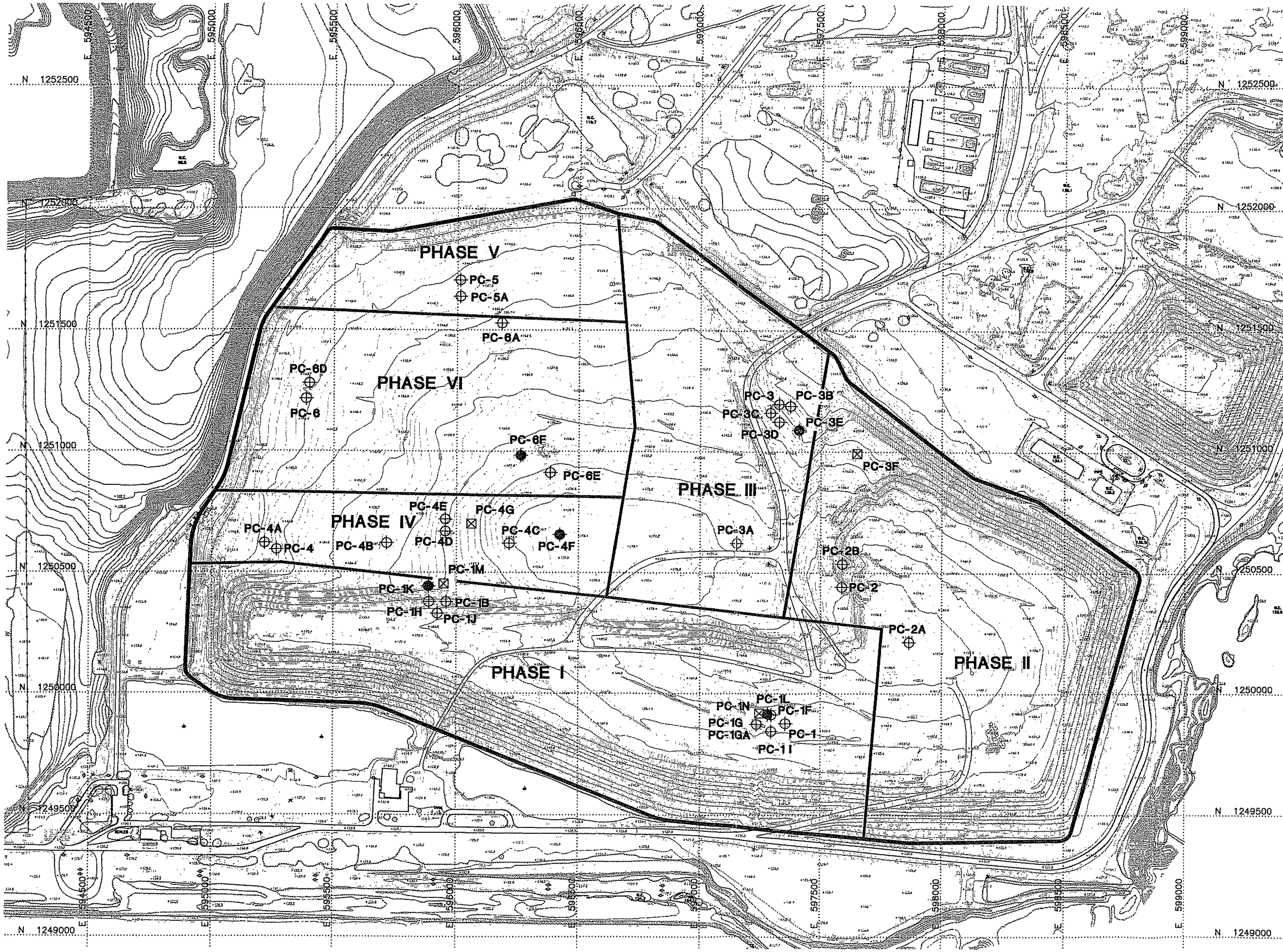

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SCALE: 1"=400'

NOTE: THE TOPOGRAPHY IS BASED ON PHOTOGRAMMETRY FROM AN AERIAL PHOTOGRAPH DATED JULY 5, 2005.

LEGEND

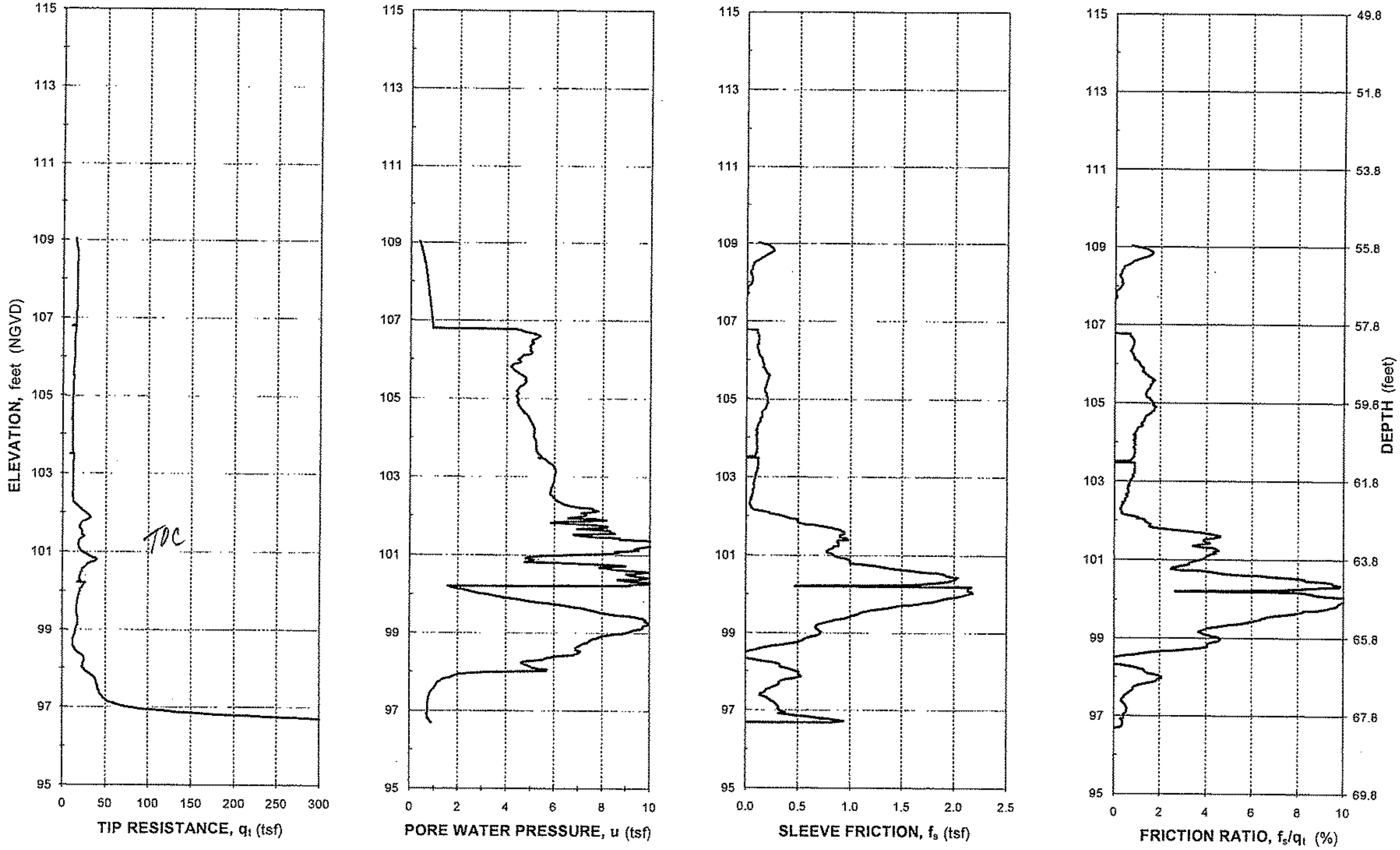
- ⊕ TEST SITES IN 2001-2004
- ● TEST SITES IN 2005
- ⊠ TEST SITES IN 2006

LANDFILL LAYOUT AND FIELD TEST LOCATION MAP

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

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AOA	CHECKED BY: PAB	DATE: 03/24/06
FILE NO. 05-072	APPROVED BY: FKCC	FIGURE: 3



Piezocone soundings were performed on 02/17/2006.

CONE PENETRATION TEST RESULTS FOR PC-1M		
Ardaman & Associates, Inc. Geotechnical, Environmental and Material Consultants		
GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA		
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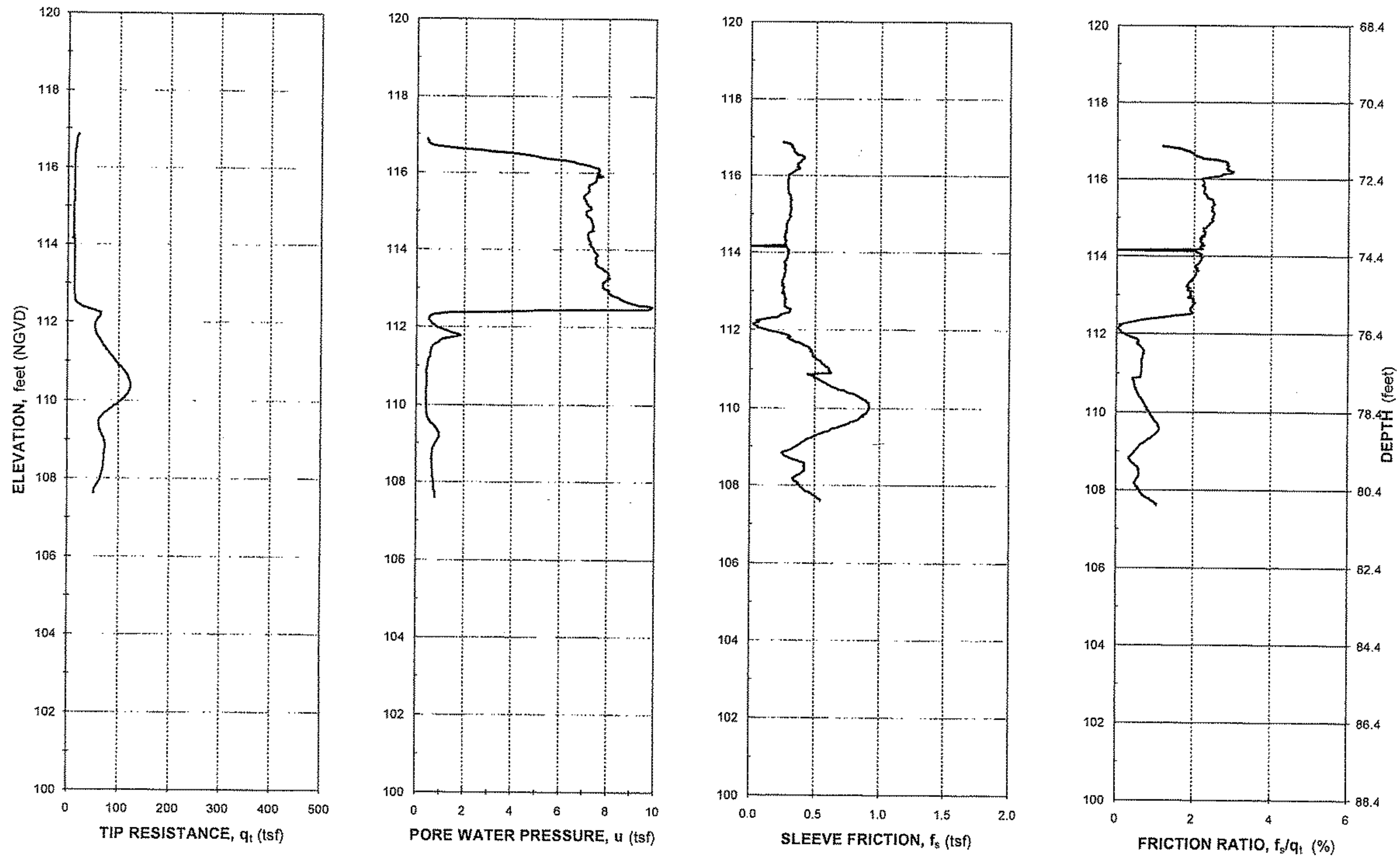
FILE NO. 05-072	APPROVED BY: PKC	FIGURE: 4



Piezocone soundings were performed on 02/28/2006.

CONE PENETRATION TEST RESULTS FOR PC-1N

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

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

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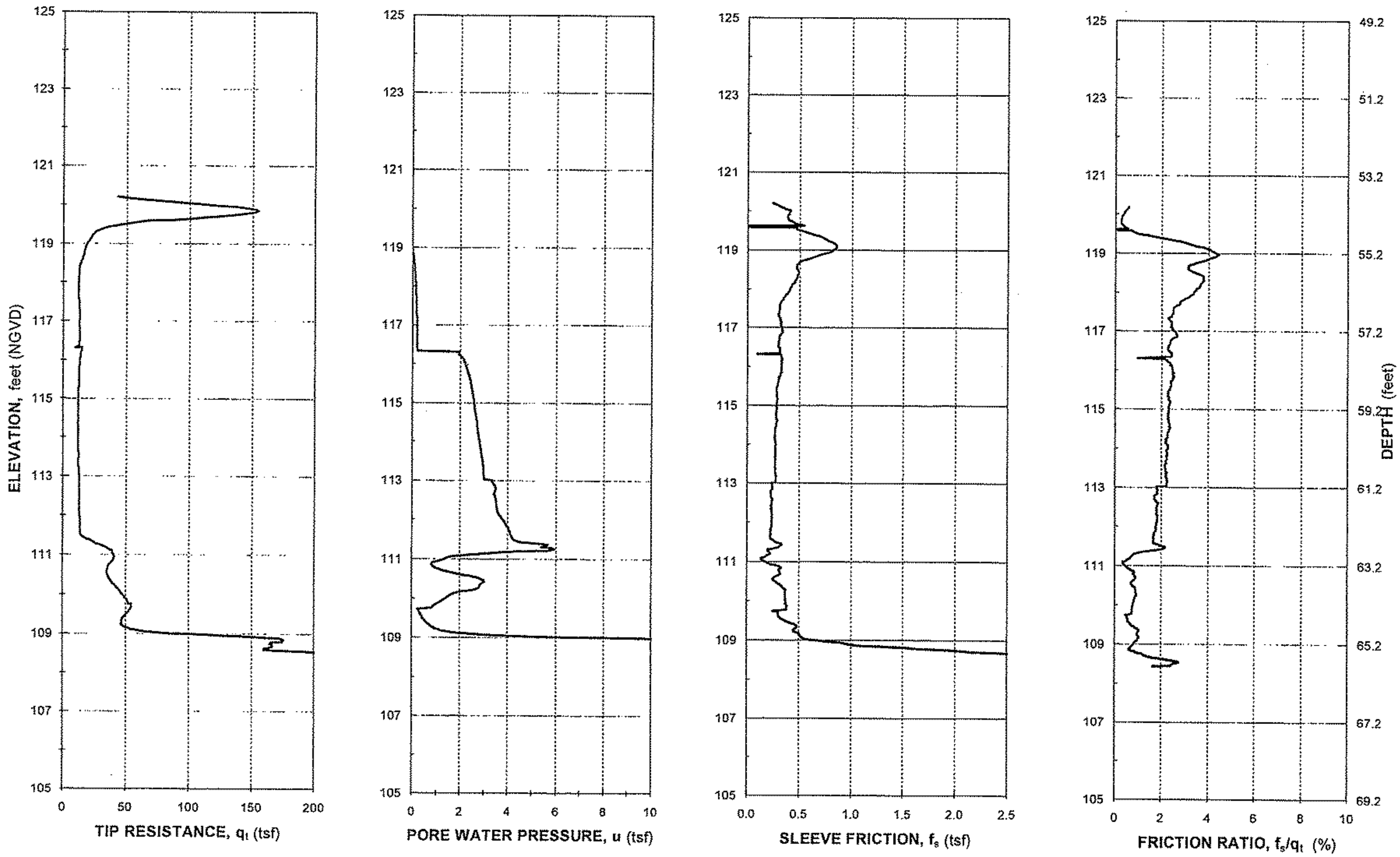

Piezocone soundings were performed on 02/21/2006.

CONE PENETRATION TEST RESULTS FOR PC-3F

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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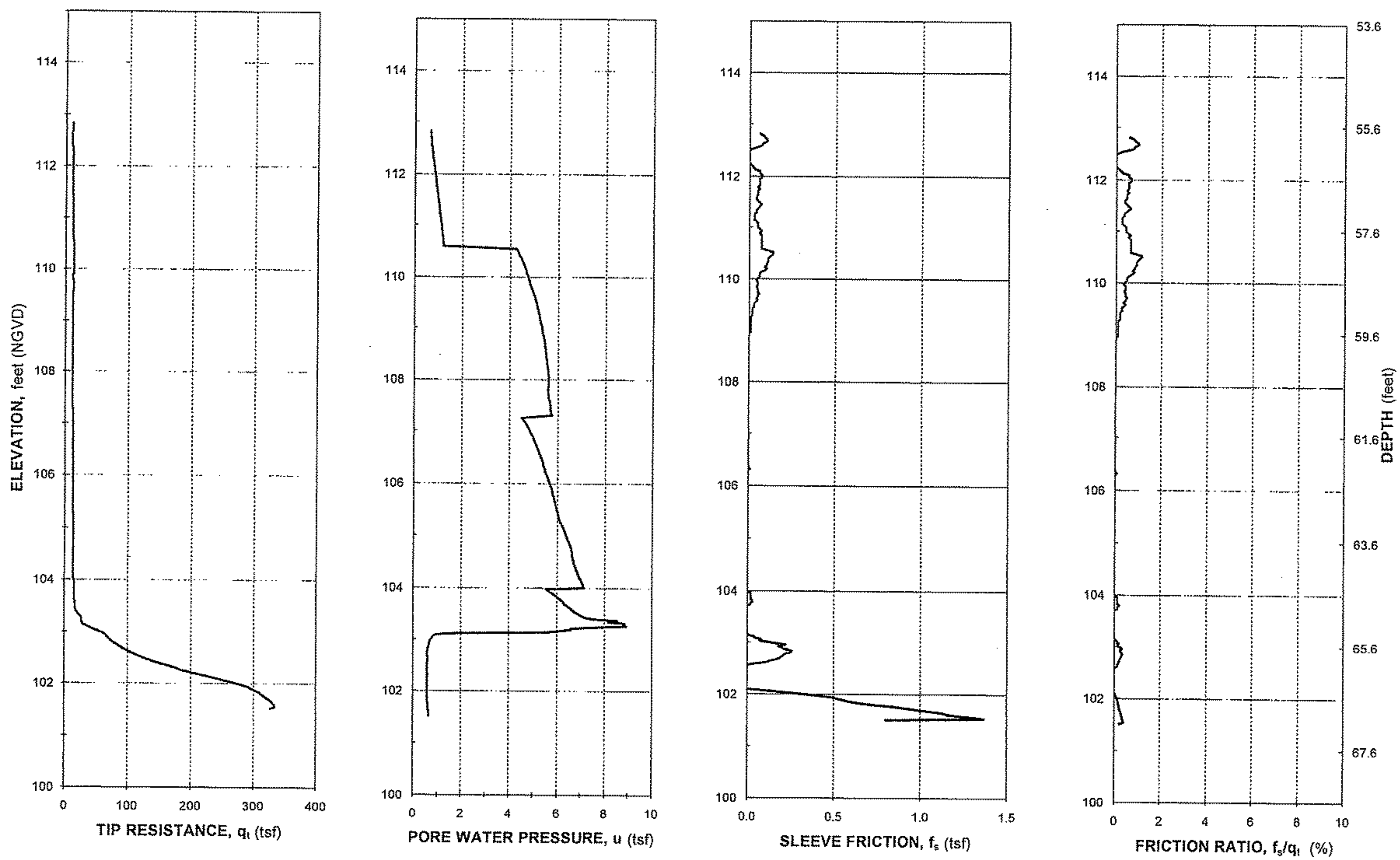
FILE NO. 05-072	APPROVED BY: PKC	FIGURE: 6



Piezocone soundings were performed on 02/17/2006.

CONE PENETRATION TEST RESULTS FOR PC-4G

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

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

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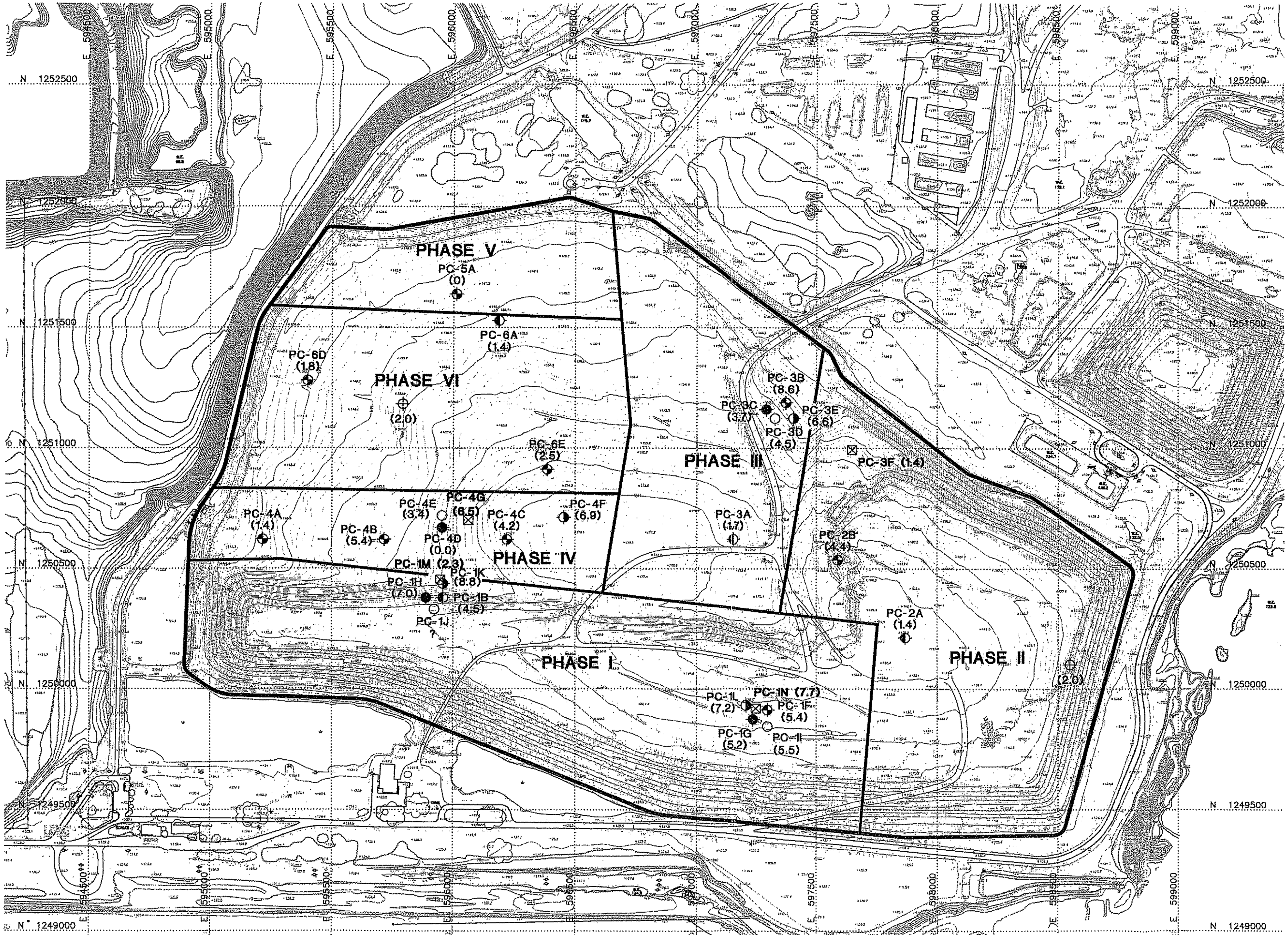
FILE NO. 05-072	APPROVED BY: FKC	FIGURE: 7



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NOTE: THE TOPOGRAPHY IS BASED ON PHOTOGRAMMETRY FROM AN AERIAL PHOTOGRAPH DATED JULY 5, 2005.

LEGEND

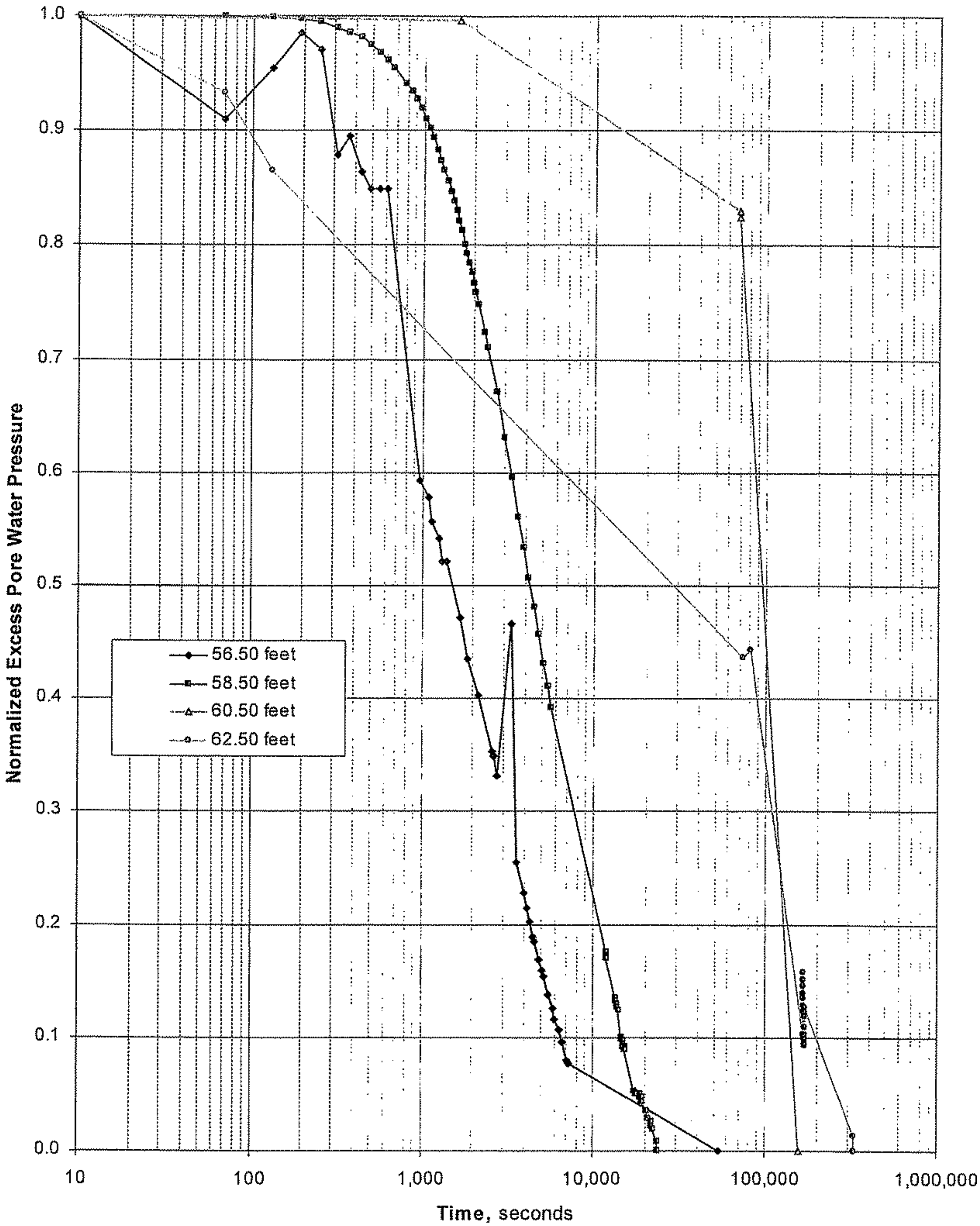
- ARDAMAN (1994)
- ARDAMAN (2001)
- ARDAMAN (2002)
- ARDAMAN (2003)
- ARDAMAN (2004)
- ARDAMAN (2005)
- TEST SITES IN 2006
- **(2.0)** PIEZOMETRIC LEVEL ABOVE TOP OF CLAY IN FEET

LEACHATE LEVELS ON TOP OF PHOSPHATIC CLAY

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

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AOA	CHECKED BY: [signature]	DATE: 03/31/06
FILE NO. 05-072	APPROVED BY: FKC	FIGURE: 8



EXCESS PORE WATER PRESSURE DISSIPATION AT PC-1M

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

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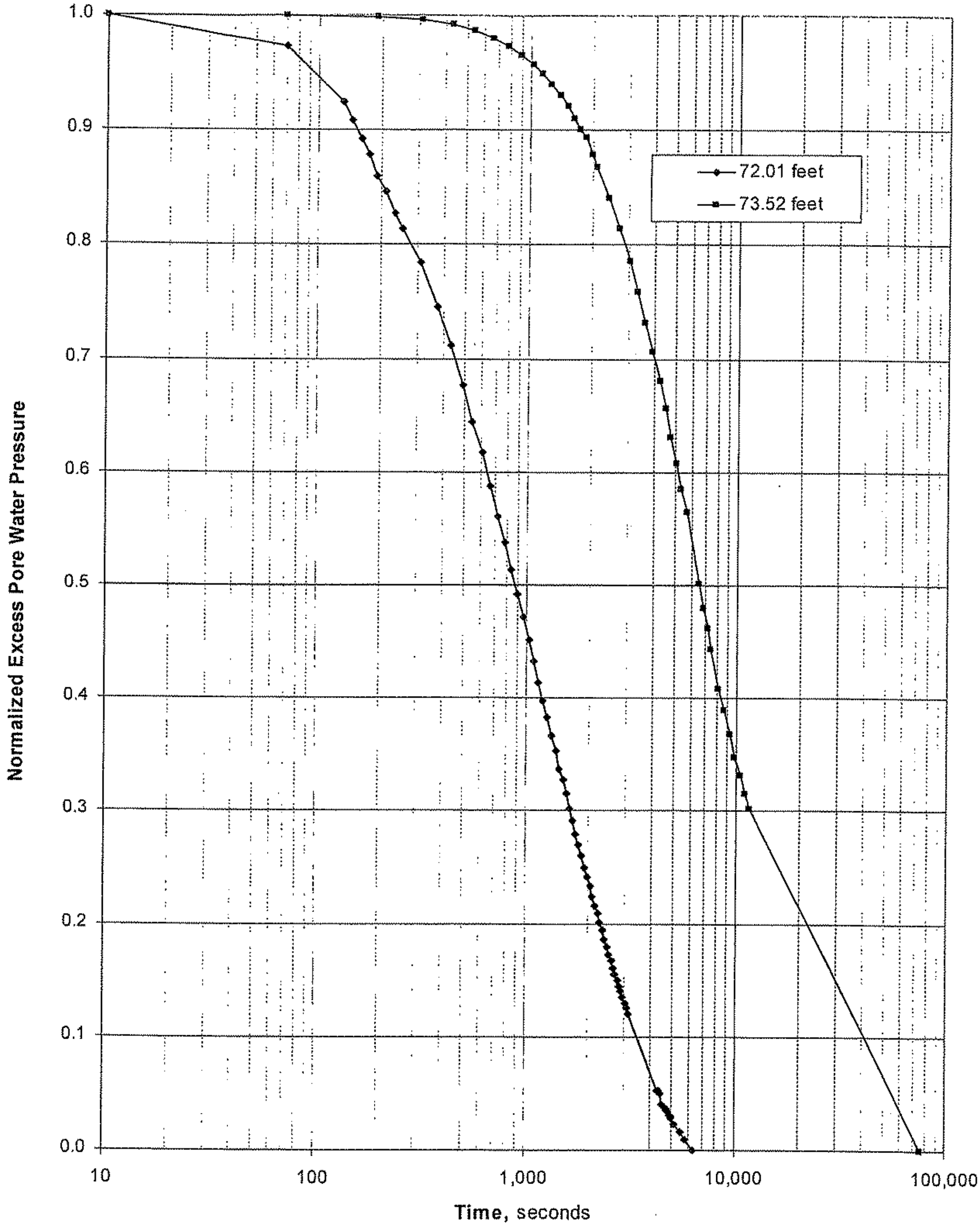
FILE NO. 05-072	APPROVED BY: FEC	FIGURE: 9



EXCESS PORE WATER PRESSURE DISSIPATION AT PC-1N
Ardaman & Associates, Inc Geotechnical, Environmental and Material Consultants
GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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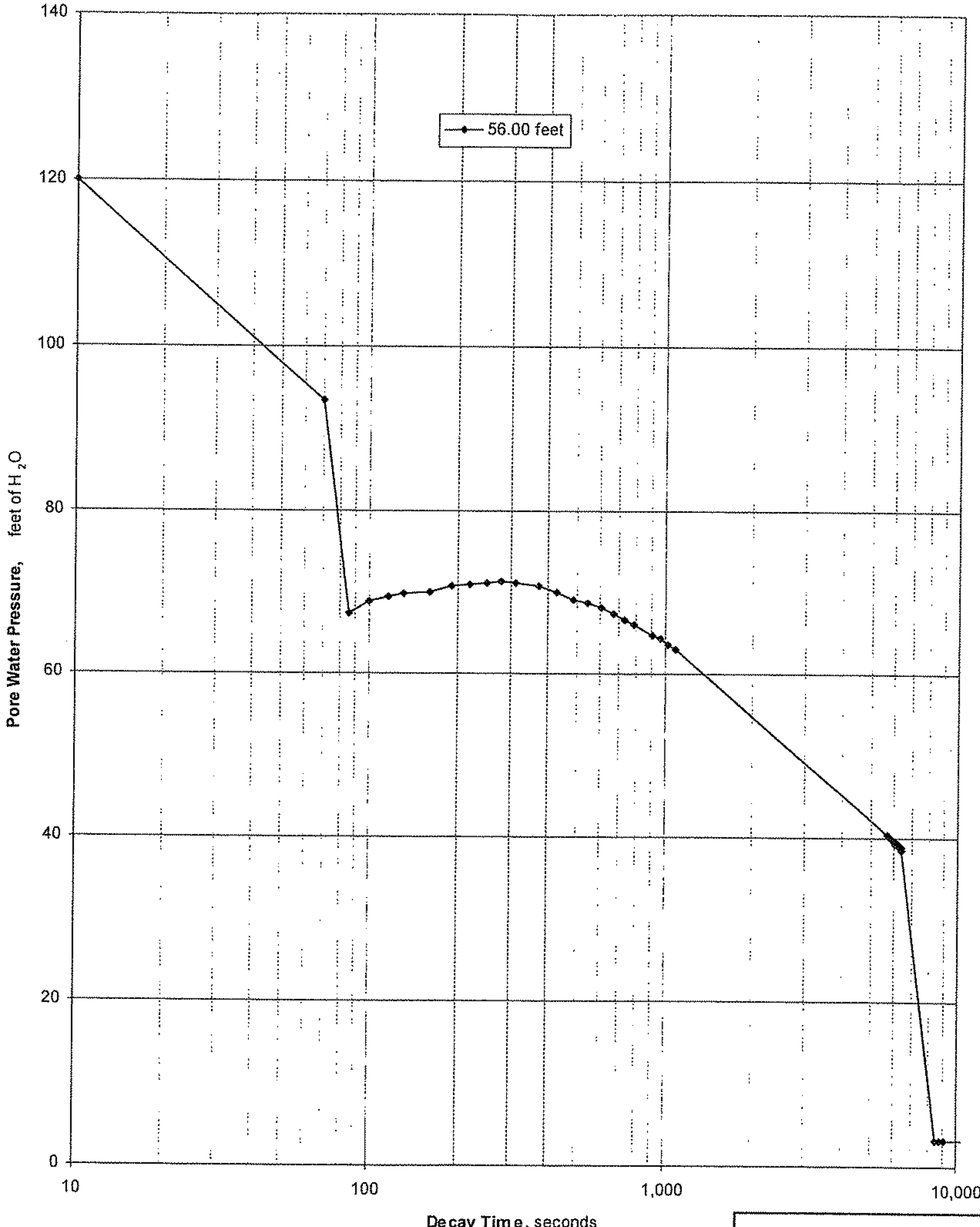
FILE NO. 05-072	APPROVED BY: FKC	FIGURE: 10



PORE WATER PRESSURE DISSIPATION AT PC-3F

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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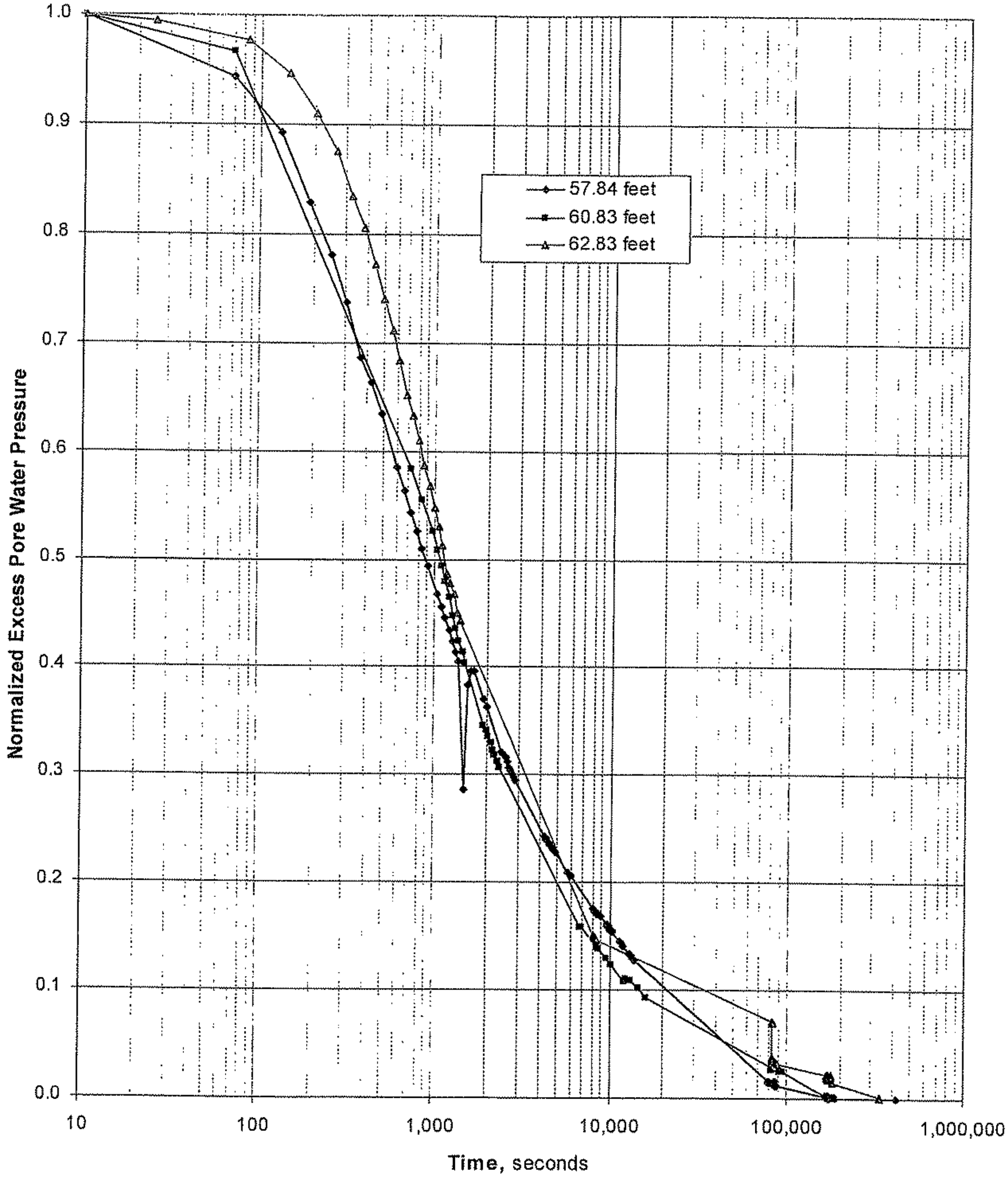

EXCESS PORE WATER PRESSURE DISSIPATION AT PC-4G

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

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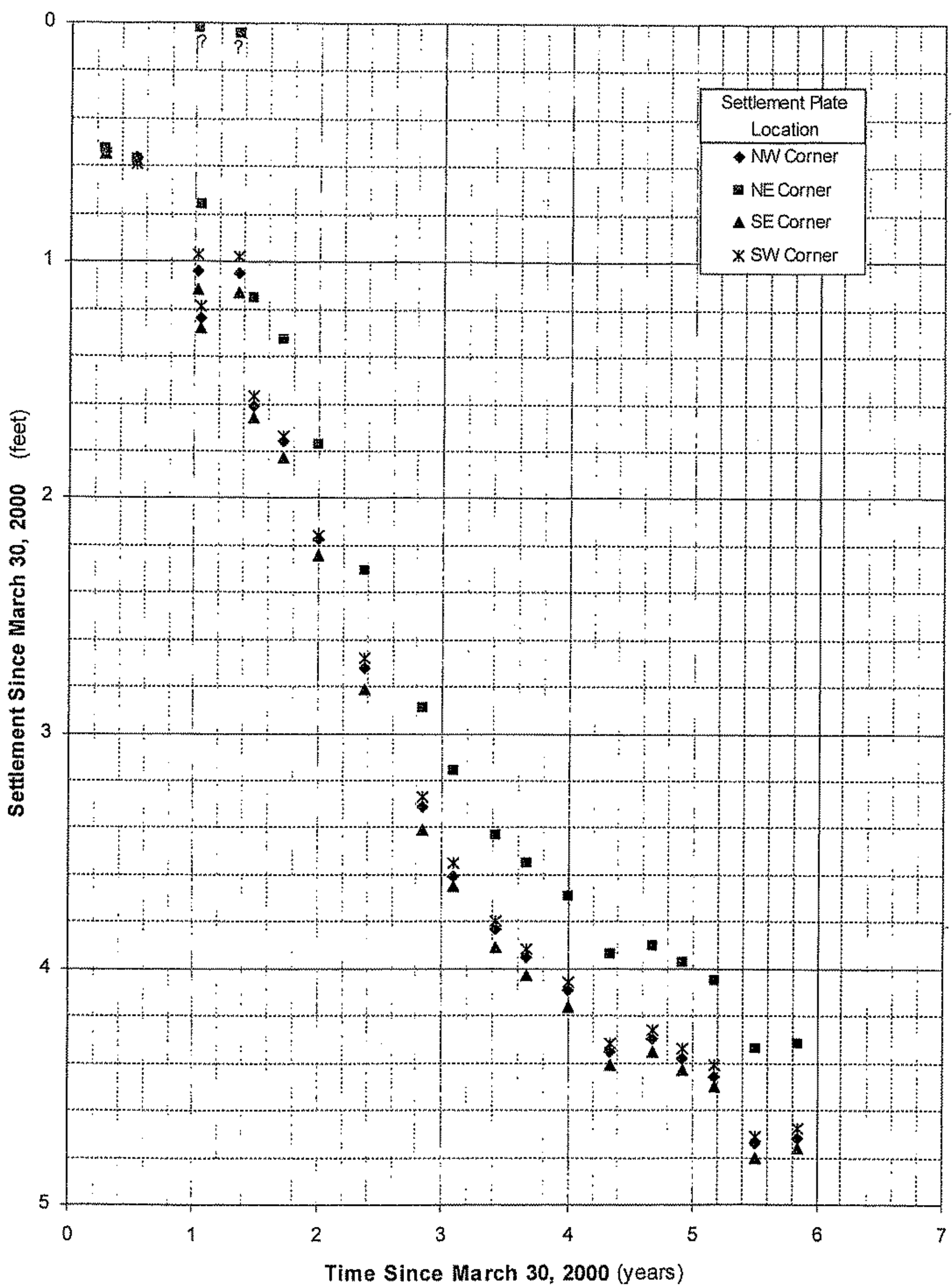
FILE NO. 05-072	APPROVED BY: PKC	FIGURE: 12

SETTLEMENT VS. TIME



SETTLEMENT OF TOP OF PHOSPHATIC CLAY AT PUMP STATION B SUMP

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

GEOTECHNICAL STUDY ASSOCIATED WITH ANNUAL MONITORING
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: PAK	CHECKED BY:	DATE: 03/29/2006
FILE NO. 05-072	APPROVED BY: FKO	FIGURE: 13