2007

Geotechnical Study Associated with 2007 Annual Monitoring of the Phosphatic Clay Liner Beneath the Southeast Landfill in Hillsborough County, Florida

May 16, 2007

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File Number 06-212

Jones Edmunds
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Attention: Mr. Joseph H. O'Neill, P.E.
Solid Waste Department Manager

Subject: Geotechnical Study Associated with 2007 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 and 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, 2005, and 2006 in accordance with permit requirements were documented in Ardaman reports dated April 12, 2003, April 15, 2004, May 27, 2005, and April 13, 2006, respectively. This report contains the results of the piezocone soundings and pore water pressure measurements obtained by Ardaman in February and March 2007, 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. 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 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 1.5×10^{-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 25, 2006. We understand that landfilling has occurred in the Phase III and IV 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 between 2001 and 2006.

Four test sites for the 2007 annual monitoring program were selected by Jones Edmonds for performance of piezocone soundings and installation of piezoprobes.

(i) PC-1O in the western portion of the Phase I area.
(ii) PC-1P in the eastern portion of the Phase I area.
(iii) PC-3G in the Phase III area.
(iv) PC-4H 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,and 2006 annual monitoring programs. As shown in Figure 3, PC-1O was located approximately 110 to 120 feet east/southeast of the previous PC-1B and PC-1J test site locations, and PC-1P was selected west of the previous PC-N, PC-1G, and PC-1GA test site locations. PC-3G was located between the previous PC-3D and PC-3E test site locations. PC-4H was selected south of the PC-4G test site location in 2006. The current field work and testing were performed by Ardaman in February and March 2007.

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 (NGVD29). 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 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 attempted to document 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.

The drilling program was performed in accordance with a health and safety plan developed by Ardaman 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.

For the 2007 annual monitoring program, in addition to the piezocone soundings and piezoprobe installations, Hillsborough County has requested Ardaman to perform soil borings and obtain undisturbed samples of the phosphatic clay from the Phase I and VI areas for direct measurements of undrained shear strengths of the phosphatic clay. In the Phase I area, the undisturbed phosphatic clay samples were recovered from two borings designated TH-1O and TH-1P that were located adjacent to PC-1O and PC-1P, respectively. In the Phase VI area, the undisturbed samples were retrieved from a soil boring designated TH-6G, as shown in Figure 3. Previous laboratory measurements of undrained shear strengths of the phosphatic clay were performed as part of the original geotechnical investigation completed in 1983 and the follow-up study completed in 1994. Sampling of the phosphatic clay was never performed during the previous annual monitoring programs.

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-1O, PC-1P, PC-3G, and PC-4H) 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

The refuse/ash thicknesses at the four piezocone test sites (i.e., PC-1O, PC-1P, PC-3G, and PC-4H) varied from approximately 61 feet at PC-4H to 69 feet at PC-1O in February and March 2007, assuming that the drainage sand layer has a thickness of 2 feet.

PC-1O was located in the western portion of the Phase I area. The refuse/ash thickness at PC-1O was estimated to be 69 feet in February 2007, compared to a refuse/ash thickness of 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-1P was located in the eastern portion of the Phase I area. The refuse/ash thickness at PC-P was estimated to be 67 feet in February 2007, compared to a refuse/ash thickness of 70 feet at PC-1N in February 2006, and a thickness of 66 feet at PC-1L in March 2005. PC-1P has approximately the same land surface elevation as PC-1N and PC-1L. We understand that no refuse/ash was placed in the Phase I area since August 2004.

The refuse/ash thickness at PC-3G was documented to be approximately 63 feet in February 2007, compared to a thickness of approximately 53 feet at PC-3F in February 2006, and approximately 34 feet at PC-3E in March 2005. The difference in refuse/ash thickness was a result of refuse/ash placement in the Phase III area that occurred between August 2005 and August 2006

The refuse/ash thickness at PC-4H during the current field program was estimated at 61 feet, compared to a refuse/ash thickness of 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-4H was approximately 6.5 feet higher than the landfill surface elevation at PC-4G. We understand that some filling, short of a full 20-foot lift, occurred near these test site locations in March/April 2003. The Phase IV area was being filled at the time of our field exploration in February/March of 2007.

Based on a review of the latest topographic survey, the piezocone soundings performed in the Phase I, III, and IV areas, and the soil borings performed in the Phase I and VI areas, the average refuse/ash thicknesses were estimated to be on the order of 50 feet in the Phase V and VI areas, 60 feet in the Phase III and IV areas, and 70 feet in the Phase I and II areas.

Piezocone Tip Resistance

The undrained shear strengths of the phosphatic clay can be inferred from the tip resistance recorded during penetration of the piezocone. Based on the tip resistance readings shown in Figures 4 through 7, the undrained shear strengths of the phosphatic clay were estimated to be on the order of 700 to 800 psf at PC-1O, 1,100 to 1,200 psf at PC-1P, 700 to 800 psf at PC-3G, and 1,100 to 1,200 psf at PC-4H.

Filling Schedule

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., less than 4 years ago), and that in the Phase II area was placed between August 2004 and August 2005 (i.e., less than 3 years ago), corresponding to 7 to 8 years after the prior 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., less than 2 years ago), corresponding to more than 14 years after the prior 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, corresponding to approximately 12 years after the prior refuse/ash lift was placed.

We understand that the landfill operation has moved or will be moving shortly into the Phase V and VI areas. Filling of the next lift of refuse/ash in the Phase V and VI areas is projected to occur over a period of approximately 2 years.

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 2007 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 +112.2 feet (NGVD) at PC-4H to a high of +118.7 feet (NGVD) at PC-1P, whereas the bottom elevations of the phosphatic clay layer ranged from a low of +104.6 feet (NGVD) at PC-4H to a high of +115.1 feet (NGVD) at PC-1P.

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 prior to landfill construction.

At the location of PC-1O, the top and bottom of the phosphatic clay layer were encountered at approximately +113.2 and +105.7 feet (NGVD), respectively, for a clay thickness of about 7.5 feet. This thickness is relatively close to the clay thickness of 8.0 feet documented at PC-1M in February 2006.

Based on the piezocone sounding performed at the location of PC-1P, the phosphatic clay had top and bottom elevations of +118.7 and +115.1 feet (NGVD), respectively, for a phosphatic clay thickness of 3.6 feet, which is close to the thickness of 3.9 feet documented at the nearby PC-1N in 2006.

The piezocone sounding performed at the location of PC-3G revealed the top elevation of the phosphatic clay at +118.9 feet (NGVD) and the bottom elevation at +111.7 feet (NGVD), for a clay thickness of 7.2 feet. The piezocone sounding at PC-3F performed in 2006 was located approximately 300 feet east of PC-3G and, therefore, would not make a good comparison to the current phosphatic clay thickness. The piezocone sounding at PC-3E performed in 2005, which was located in close proximity to PC-3G, indicated that the top elevation of the phosphatic clay at +118.7 feet (NGVD) and the bottom elevation at +110.4 feet (NGVD), for a clay thickness of 8.3 feet.

Based on the piezocone sounding performed at the location of PC-4H, the top and bottom elevations of the phosphatic clay were documented at +112.2 and +104.6 feet (NGVD), for a clay thickness of 7.6 feet, which is fairly close to the thickness of 7.2 feet documented at the nearby PC-4G test site in 2006.

Compared to the latest known thicknesses at corresponding nearby test sites, the phosphatic clay thicknesses documented at the four current test sites in February/March 2007 indicated that the clay thicknesses near PC-1O, PC-1P, and PC-3G have decreased by 0.3 to 1.1 feet, whereas the clay thickness near PC-4H 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.

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.

The above technique, which has been used in previous annual monitoring programs, could not be used this time to obtain piezometric elevations in the drainage sand layer at PC-1O and PC-1P because the drainage sand layer was too dense for the piezocone to penetrate. At the location of PC-1O, we estimated the piezometric elevation on top of the phosphatic clay to be above +114.9 feet (NGVD), based on the final equilibrium hydraulic head reading of the first piezoprobe that was installed just below the surface of the phosphatic clay. At the location of PC-1P, we estimated the piezometric elevation on top of the phosphatic clay to be +124.1 feet (NGVD), based on the piezometric elevations previously documented in 2005 and 2006.

At the location of PC-3G, using the technique described above and also considering the final equilibrium hydraulic head reading of the first piezoprobe that was installed below the surface of the phosphatic clay, the piezometric elevation in the drainage sand layer was estimated to be above +125.2 feet (NGVD).

At the location of PC-4H, the piezocone malfunctioned and failed to record pore water pressure. Accordingly, the piezometric elevation in the drainage sand layer was documented using a piezoprobe installed within the drainage sand layer. Based on the final equilibrium hydraulic head recorded in the piezoprobe, the piezometric elevation in the drainage sand layer was estimated at +119.8 feet (NGVD).

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 on 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 1.7 to 7.6 feet. The highest piezometric elevation of +125.2 feet (NGVD), corresponding to 6.3 feet above the top of the phosphatic clay, was observed at PC-3G. 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 +119 to +128 feet (NGVD). Except for the reading obtained at PC-4H, the latest piezometric elevations were comparable to previous measurements. The latest piezometric elevation at PC-4H was documented at +125.7 feet (NGVD), whereas the elevations fluctuated within a narrow range of +117 to +119 feet (NGVD) between 2003 and 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-1O, PC-1P, PC-3G, and PC-4H, 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.

At the location of PC-1O, with approximately 7.5 feet of phosphatic clay, piezoprobe tests were performed at three different depths to measure pore water pressure. At the location of PC-1P, with approximately 3.6 feet of phosphatic clay, piezoprobe tests were performed at two different depths. At the location of PC-3G, with approximately 7.2 feet of phosphatic clay, piezoprobe tests were performed at two different depths. At the location of PC-4H, with approximately 7.6 feet of phosphatic clay, piezoprobe tests were performed at three different depths. Results from the piezoprobe tests are summarized in Table 5.

At the location of PC-1O, the equilibrium piezometric elevations for the three piezoprobes installed with tip elevations at +112.2, +110.2, and +109.2 feet (NGVD) were documented to be approximately +114.9, +120.0, and +119.7 feet (NGVD), respectively. These elevations indicated minimal excess pore water pressure at the piezoprobe depths. The maximum excess pore water pressure at PC-1M in 2006 was documented to be as high as 12 feet of water.

At the location of PC-1P, two piezoprobe tests were performed with the piezoprobe tip elevations at +117.4 and +116.4 feet (NGVD). The piezoprobe with the tip elevation at +117.4 feet (NGVD) was 1.3 feet below the top elevation of the phosphatic clay. The piezoprobe readings indicated an excess pore water pressure of 1.9 feet of water. The piezometric elevation for the tip elevation at +116.4 feet (NGVD) was documented to be +132.9 feet (NGVD), corresponding to an excess pore water pressure that is equivalent to approximately 6.3 feet of water, which was slightly lower than the 7.4 feet of excess pore water pressure documented at PC-1N in 2006.

At the location of PC-3G, piezoprobe tests were performed with the piezoprobe tip elevations at +118.9 and +116.9 feet (NGVD)). Results of the piezoprobe test performed with the piezoprobe tip elevation at +118.9 feet (NGVD), which is near the top of the phosphatic clay, showed minimal excess pore water pressure. The piezoprobe test installed at +116.9 feet (NGVD), which was at 2 feet below the top of the phosphatic clay, indicated an excess pore water pressure of approximately 18 feet of water. The high excess pore water pressure is expected because of recent refuse/ash placement in the Phase III area.

At the location of PC-4H, three piezoprobe tests were performed within the phosphatic clay layer with the piezoprobe tip elevations at +112.2, +110.2, and +109.2 (NGVD). The equilibrium piezometric elevations were at +120.1, +137.1, and +136.8 feet (NGVD), respectively. The excess pore water pressures near the middle of the phosphatic clay were equivalent to approximately 15 feet of water. The high excess pore water pressures could be attributed to recent refuse/ash loading in the Phase IV area.

In summary, the piezoprobe data indicated minimal excess pore water pressure at PC-1O, some residual excess pore water pressure at PC-1P, and high excess pore water pressure at PC-3G and PC-4H. 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 Southeast Landfill 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 double drainage condition.

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, $c_v \approx 0.05 \times c_h$. Results of the calculations are presented in Table 6.

As shown in Table 6, the coefficients of consolidation calculated from the piezoprobe data for 2007 were generally much lower than the design coefficient of consolidation and previous data, and the excess pore water pressure data revealed by the piezoprobe. An incremental consolidation test performed recently by Ardaman on an undisturbed sample of phosphatic clay collected from TH-6G at a depth of 55 to 57 feet indicated a coefficient of vertical consolidation of 9×10^{-5} to 2×10^{-4} cm^2/sec for vertical effective stress ranging from 1 to 2.5 tsf, corresponding to a landfill height of approximately 25 to 65 feet. Settlement plate data at the location of Pump Station B in the Phase VI area provided by Hillsborough County in 2006 also supported a coefficient of consolidation of 1.7×10^{-4} cm^2/sec, which is consistent with the design value.

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-1P, PC-3G, and PC-4H. Accordingly, under existing conditions, there should be no downward leachate migration through the phosphatic clay layer at these locations.

Summary of Observations

The following key observations were made from the 2007 annual monitoring data:

- The average refuse/ash thicknesses were estimated to be on the order of 50 feet in the Phase V and VI areas, 60 feet in the Phase III and IV areas, and 70 feet in the Phase I and II areas.

- The top elevations of the phosphatic clay were as expected and generally consistent with the range of elevations obtained in 2006.

- 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 prior to landfill construction.

- Compared to the 2006 data, the phosphatic clay thicknesses documented in February/March 2007 near PC-1O, PC-1P, and PC-3G decreased by 0.3 to 1.1 feet, whereas the clay thickness near PC-4H increased by 0.4 feet. These differences are considered minimal, and could be attributed to settlement of the phosphatic clay layer or the test sites not being at identical locations from year to year.

- Based on the piezocone tip resistance, the undrained shear strengths of the phosphatic clay were estimated to be on the order of 700 to 800 psf at PC-1O, 1,100 to 1,200 psf at PC-1P, 700 to 800 psf at PC-3G, and 1,100 to 1,200 psf at PC-4H.

- The piezoprobe data indicated minimal excess pore water pressure at PC-1O, some residual excess pore water pressure at PC-1P, and high excess pore water pressure at PC-3G and PC-4H. The high excess pore water pressure at PC-3G and PC-4H is expected because of recent refuse/ash placement in the Phase III area.

- The piezometric heads above the phosphatic clay at the PC-1O, PC-1P, PC-3G, and PC-4H test site locations were measured to be 0.3, 5.4, 6.3, and 7.6 feet, respectively.

- Except for the reading obtained at PC-4H, the latest piezometric elevations below the phosphatic clay were comparable to previous measurements. The latest piezometric elevation at PC-4H was documented at +125.7 feet (NGVD), whereas the elevations fluctuated within a narrow range of +117 to +119 feet (NGVD) between 2003 and 2006.

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

Recommendations

In reviewing all available data, including direct measurements of undrained shear strengths from undisturbed samples of phosphatic clay recovered from TH-1O, TH-1P, and TH-6G as well as the piezocone tip resistance, the phosphatic clay in the Phase V and VI areas as well as along the south side of the Phase I area (where the phosphatic clay is relatively thin) appeared to be fully consolidated with an undrained shear strength reaching maximum values derived from the existing overburden loads. Stability analyses of the next lift of refuse indicated that the Phase V and VI area and the south side of Phase I area are ready to receive the next lift of refuse. However, we recommend the west slope of the Phase I area not be filled at this time because the undrained shear strengths documented at PC-1O and TH-1O remained relatively low. Additional testing and studies are recommended for this area.

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, P.E. 05/16/2007
Project Manager
Florida License No. 48430

Francis K. Cheung, P.E. 05/16/07
Senior Project Engineer
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, NGVD)
PC-1O	Phase I West	1,250,305.27	596037.84	+184.2
PC-1P	Phase I East	1,249,883.45	597,202.13	+187.6
PC-3G	Phase III	1,251,102.84	597,378.53	+182.9
PC-4H	Phase IV	1,250,660.75	596,063.88	+175.2

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-1O	02/20/07	+184.2	71.0	+113.2	78.5	+105.7	7.5
	PC-1P	02/16/07	+187.6	68.9	+118.7	72.5	+115.1	3.6
Phase III	PC-3G	02/19/07	+182.9	64.0	+118.9	71.8	+111.7	7.2
Phase IV	PC-4H	03/02/07	+175.2	63.0	+112.2	70.6	+104.6	7.6

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-1O	02/20/07	+184.2	+114.9	+113.2	1.7
	PC-1P	02/16/07	+188.4	+124.1	+118.7	5.4
Phase III	PC-3G	03/06/07	+174.2	+125.2	+118.9	6.3
Phase IV	PC-4H	03/05/07	+168.6	+119.8	+112.2	7.6

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-1O	02/20/07	+184.2	+123.0
	PC-1P	02/16/07	+188.4	+128.0
Phase III	PC-3G	02/19/07	+182.9	+119.2
Phase IV	PC-4H	03/19/07	+175.2	+125.7

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-1O	+184.2	+113.2	+105.7	72.0 74.0 75.0	+112.2 +110.2 +109.2	2.7 9.8 10.5	+114.9 +120.0 +119.7	+114.9	+123.0	-1.1 (?) 1.9 0.4
	PC-1P	+188.4	+118.7	+115.1	71.0 72.0	+117.4 +116.4	10.0 16.5	+127.4 +132.9	+124.1	+128.0	1.9 6.3
Phase III	PC-3G	+182.9	+118.9	+111.7	64.0 66.0	+118.9 +116.9	6.5 24.9	+125.4 +141.8	+125.2	+119.2	0.2 18.3
Phase IV	PC-4H	+175.6	+112.2	+104.6	63.0 65.0 66.0	+112.2 +110.2 +109.2	7.9 26.9 27.6	+120.1 +137.1 +136.8	+119.8	+125.7	0.3 15.7 14.6

Table 6

Computed Coefficients of Consolidation from Piezoprobe Tests

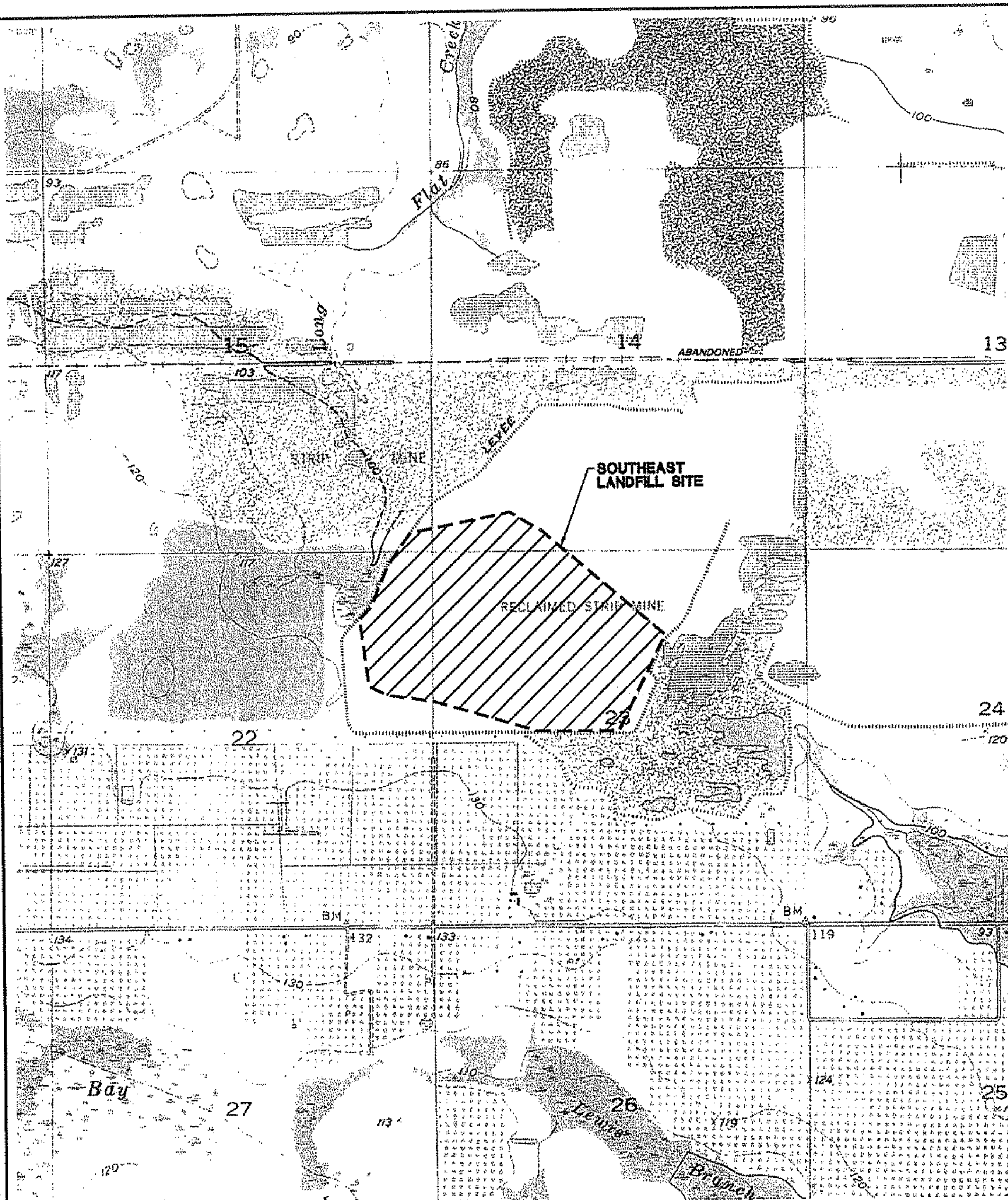
Area	Test Site	Piezoprobe Designation	Piezoprobe Tip Depth Below Ground Surface (feet)	Elevation of Piezoprobe Tip (feet, NGVD)	Horizontal Coefficient of Consolidation, c_h (cm^2/sec)	Vertical Coefficient of Consolidation, c_v* (cm^2/sec)
Phase I	PC-1O	PC-1O	74.0 75.0	+110.2 +109.2	2.5×10^{-3} 8.7×10^{-4}	1.2×10^{-4} 4.3×10^{-5}
	PC-1P	PC-1P	71.0 72.0	+119.4 +118.4	4.3×10^{-4} 3.0×10^{-4}	2.1×10^{-5} 1.5×10^{-5}
Phase III	PC-3G	PC-3G	64.0 66.0	+110.2 +108.2	1.7×10^{-4} 2.0×10^{-4}	8.1×10^{-6} 9.9×10^{-6}
Phase IV	PC-4H	PC-4H	63.0 65.0 66.0	+105.6 +103.6 +102.6	8.2×10^{-4} 1.4×10^{-4} 4.2×10^{-4}	4.0×10^{-5} 6.8×10^{-6} 2.1×10^{-5}

* 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-1O	03/07/07	+184.2	+114.9	+120.0	+123.0	Below Clay
	PC-1P	03/07/07	+188.4	+124.1	+132.9	+128.0	Within Clay
Phase III	PC-3G	03/06/07	+182.9	+125.2	+141.8	+119.2	Within Clay
Phase IV	PC-4H	03/05/07	+175.2	+119.8	+137.1	+125.7	Within Clay



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)

0 1000 2000

SCALE: — 1"=2000'

SITE LOCATION MAP

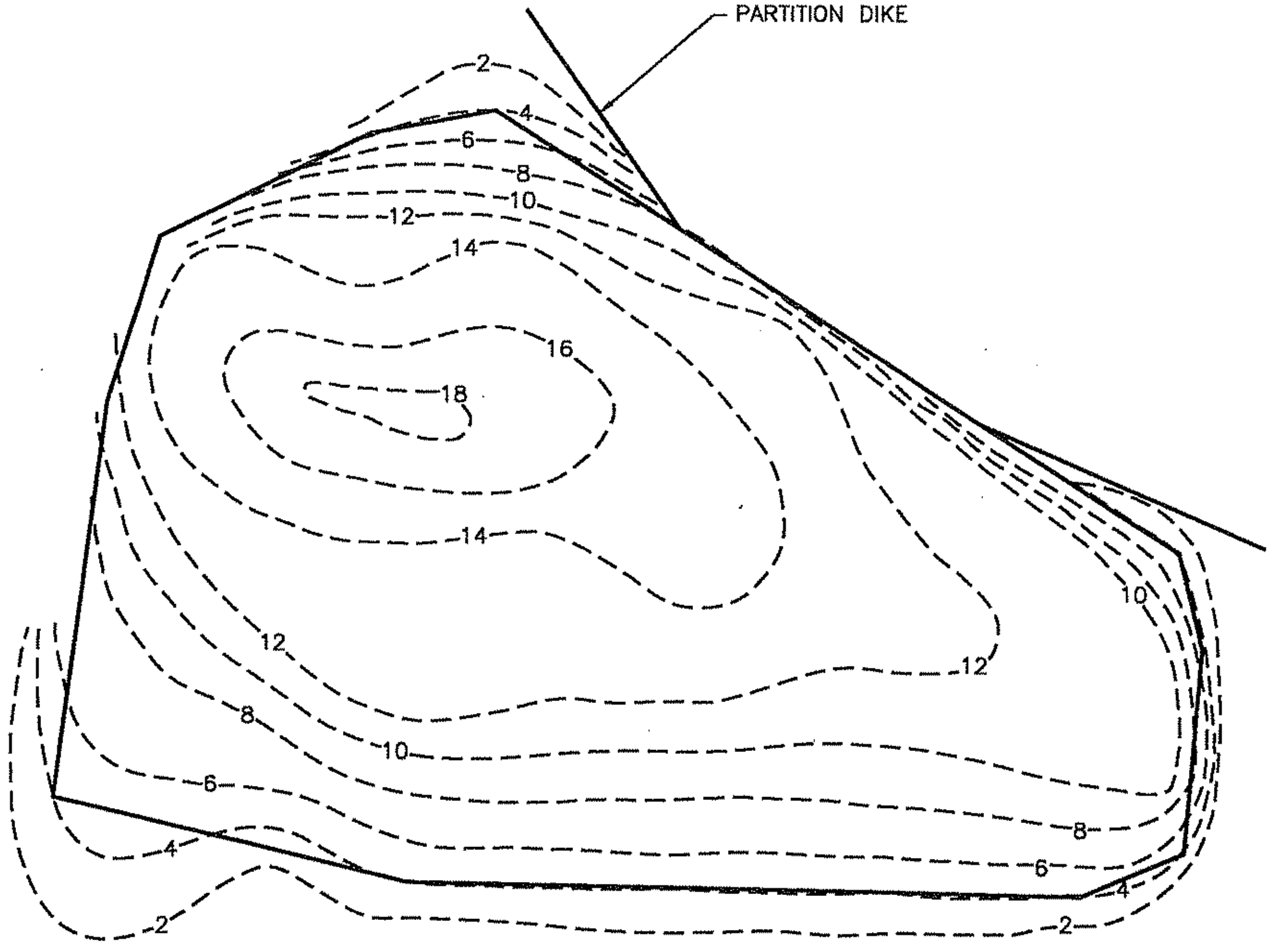
Ardaman & Associates, Inc
Geotechnical, Environmental and Materials Consultants

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

DRAWN BY: CD	CHECKED BY:	DATE: 03/30/07
FILE NO. 06-212	APPROVED BY:	FIGURE: 1

T:\Corporate\06\06-212\06-212-01.dwg

Plotted: Apr 10, 2007 – 3:59pm by chris.drew



N

0 250 500

LEGEND

10 CONTOURS OF CLAY THICKNESS IN FEET

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

THICKNESS OF PHOSPHATIC CLAY BEFORE LANDFILL CONSTRUCTION

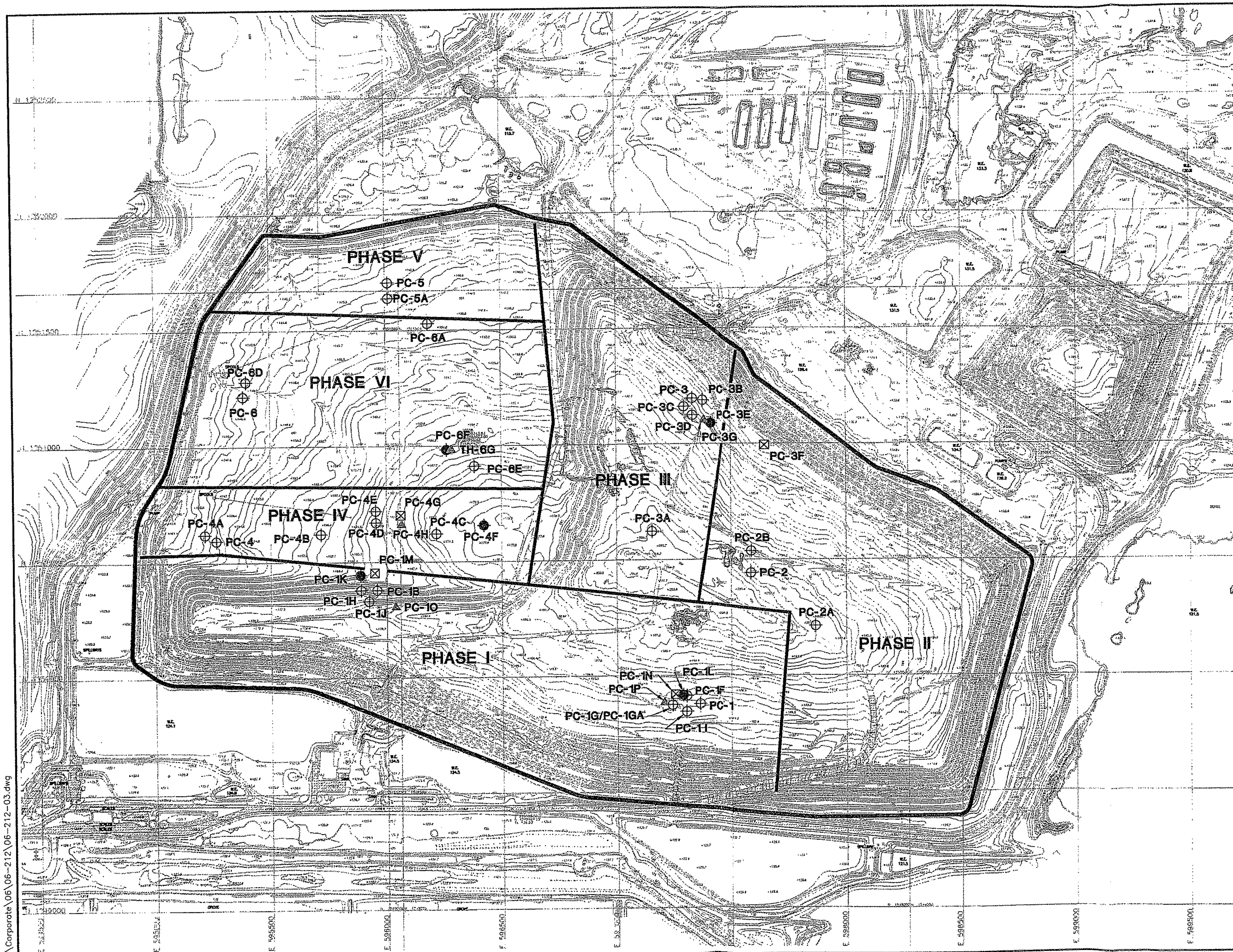
Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

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

DRAWN BY: CD	CHECKED BY:	DATE: 03/30/07
FILE NO. 06-212	APPROVED BY:	FIGURE: 2

T:\Corporate\06\06-212\06-212-02.dwg

Plotted: Apr 10, 2007 - 4:01pm by chris.drew



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SCALE: 1"=400'

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

LEGEND

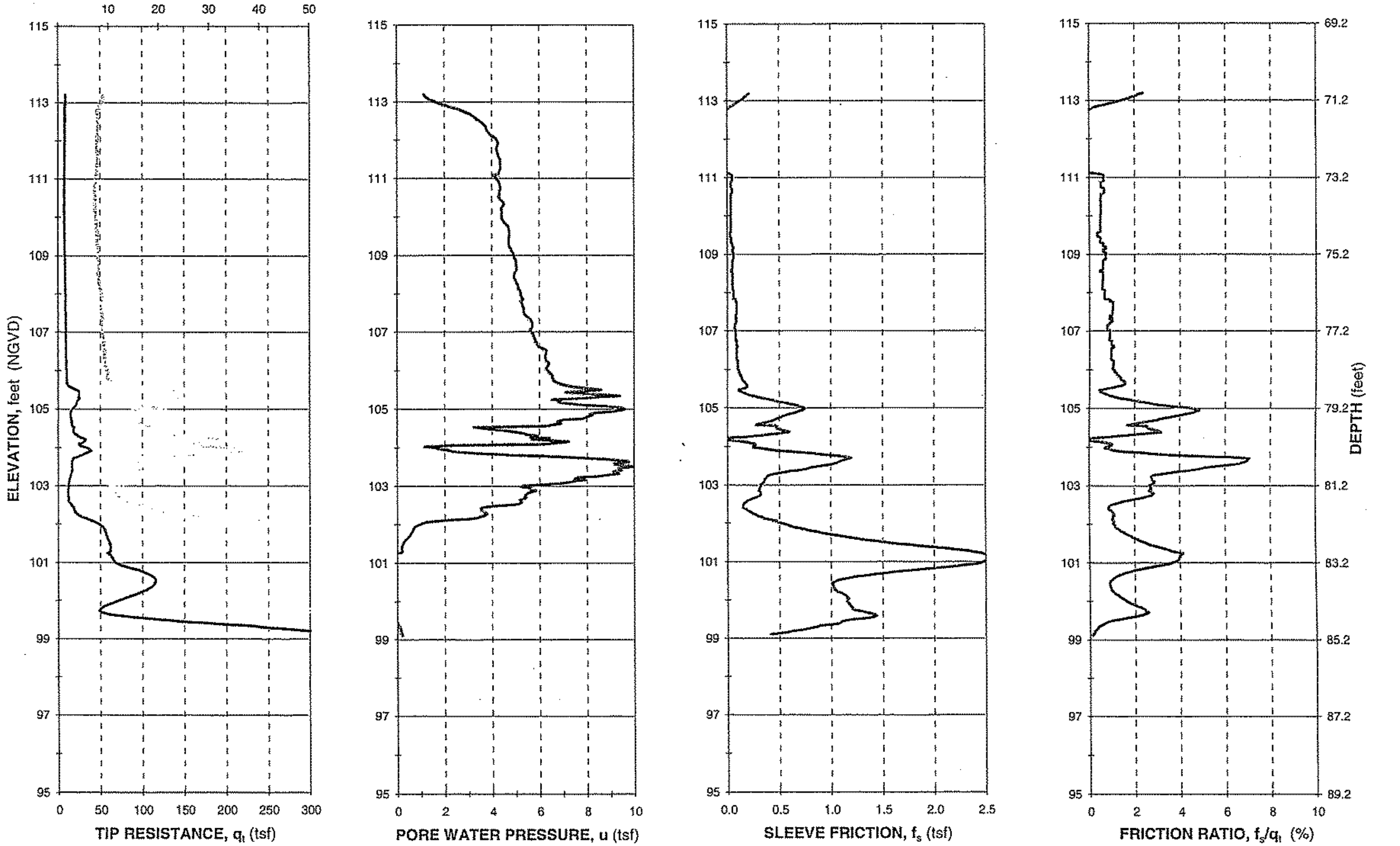
- ⊕ TEST SITES IN 2001–2004
- ● TEST SITES IN 2005
- ⊠ TEST SITES IN 2006
- ▲ TEST SITES IN 2007

LANDFILL LAYOUT AND FIELD TEST LOCATION MAP

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

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

DRAWN BY: AOA	CHECKED BY:	DATE: 03/30/07
FILE NO. 06-212	APPROVED BY:	FIGURE: 3



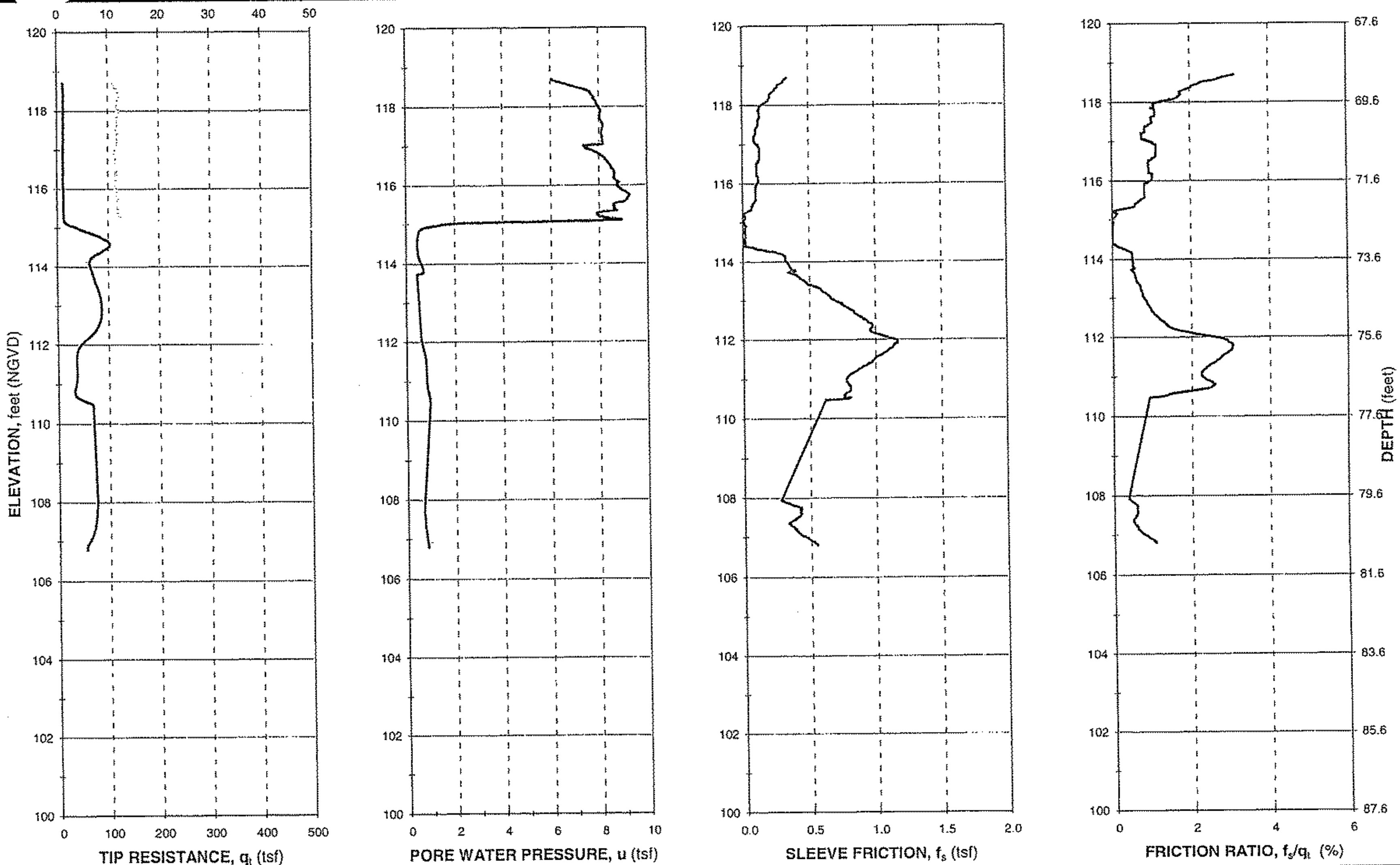
Piezocone soundings were performed on 02/20/2007.

CONE PENETRATION TEST RESULTS FOR PC-1O

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

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 4



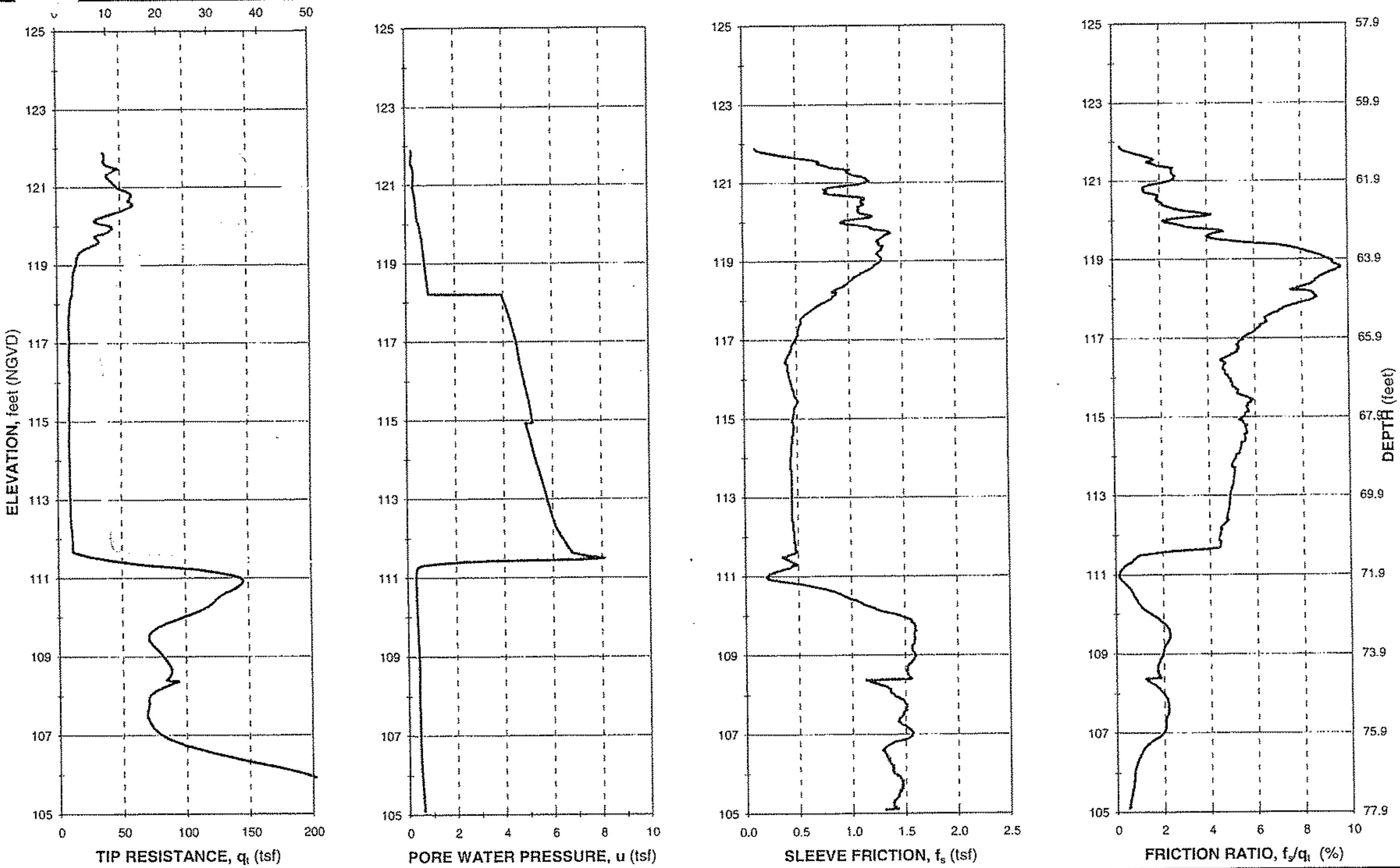
Piezocone soundings were performed on 02/16/2007.

CONE PENETRATION TEST RESULTS FOR PC-1P

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 5



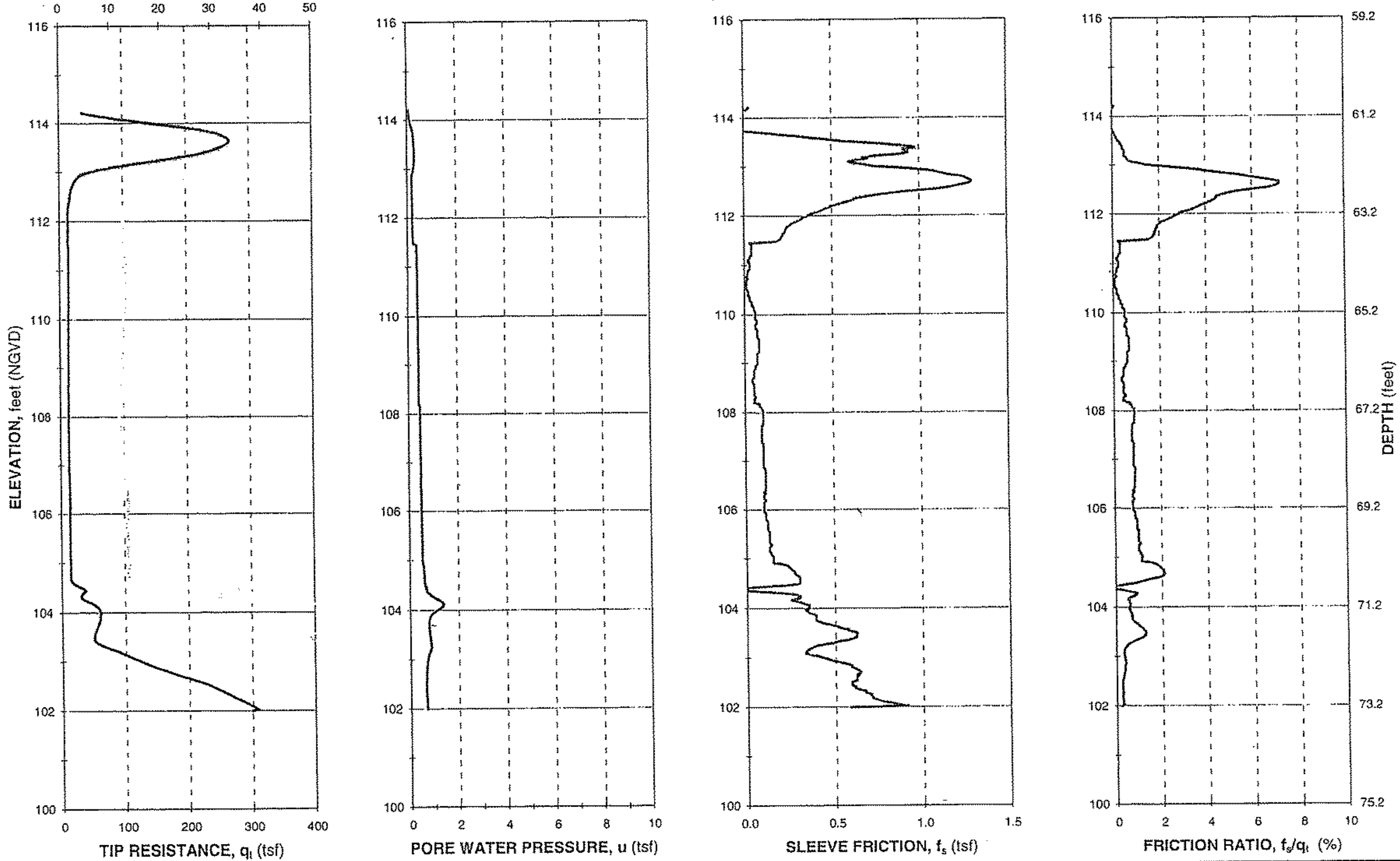
Piezocone soundings were performed on 02/19/2007.

CONE PENETRATION TEST RESULTS FOR PC-3G

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 6



Piezocone soundings were performed on 03/02/2007.

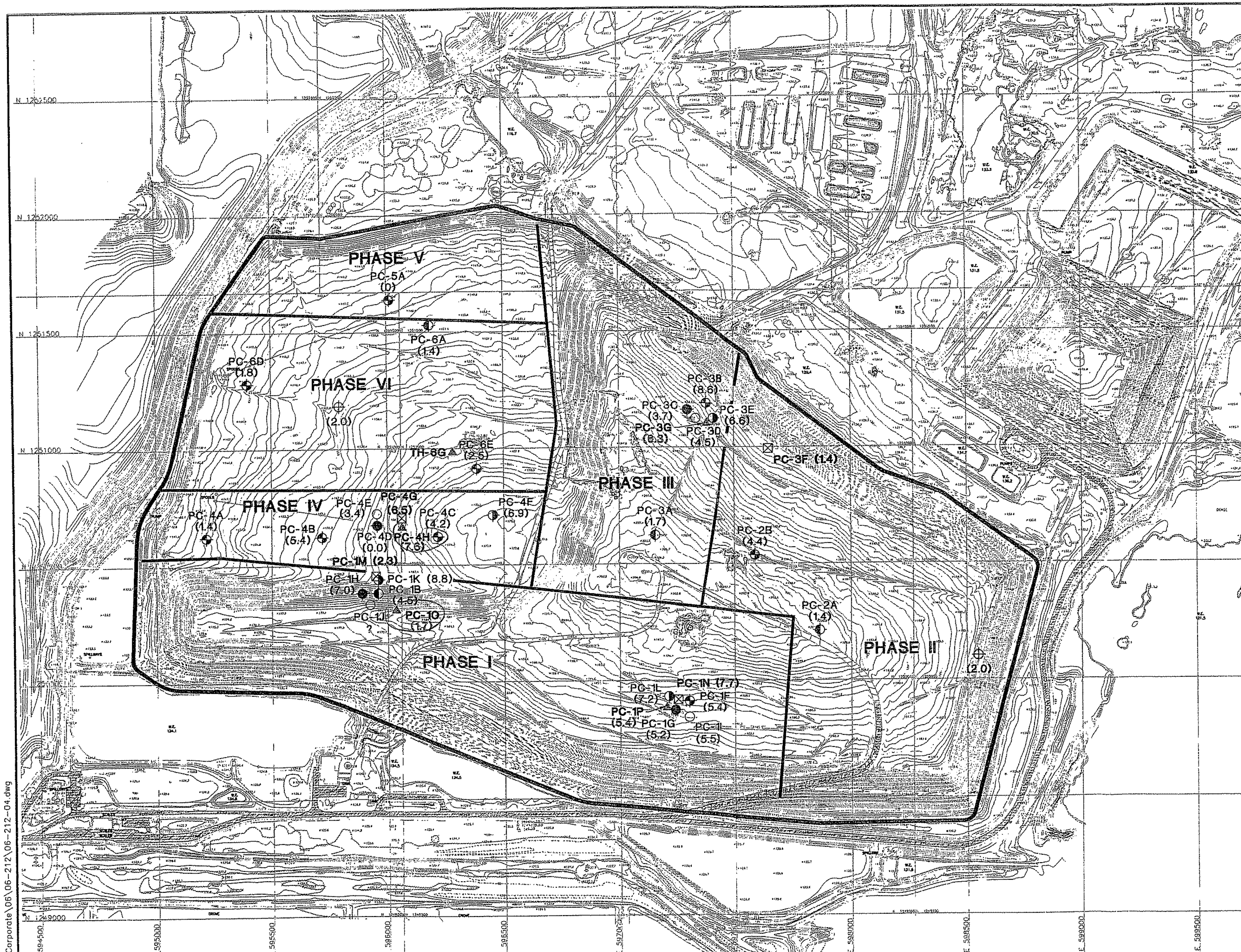
Pore water pressure readings are not valid because probe dried out.

CONE PENETRATION TEST RESULTS FOR PC-4H

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

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 7



N

0 200 400

SCALE: 1"=400'

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

LEGEND

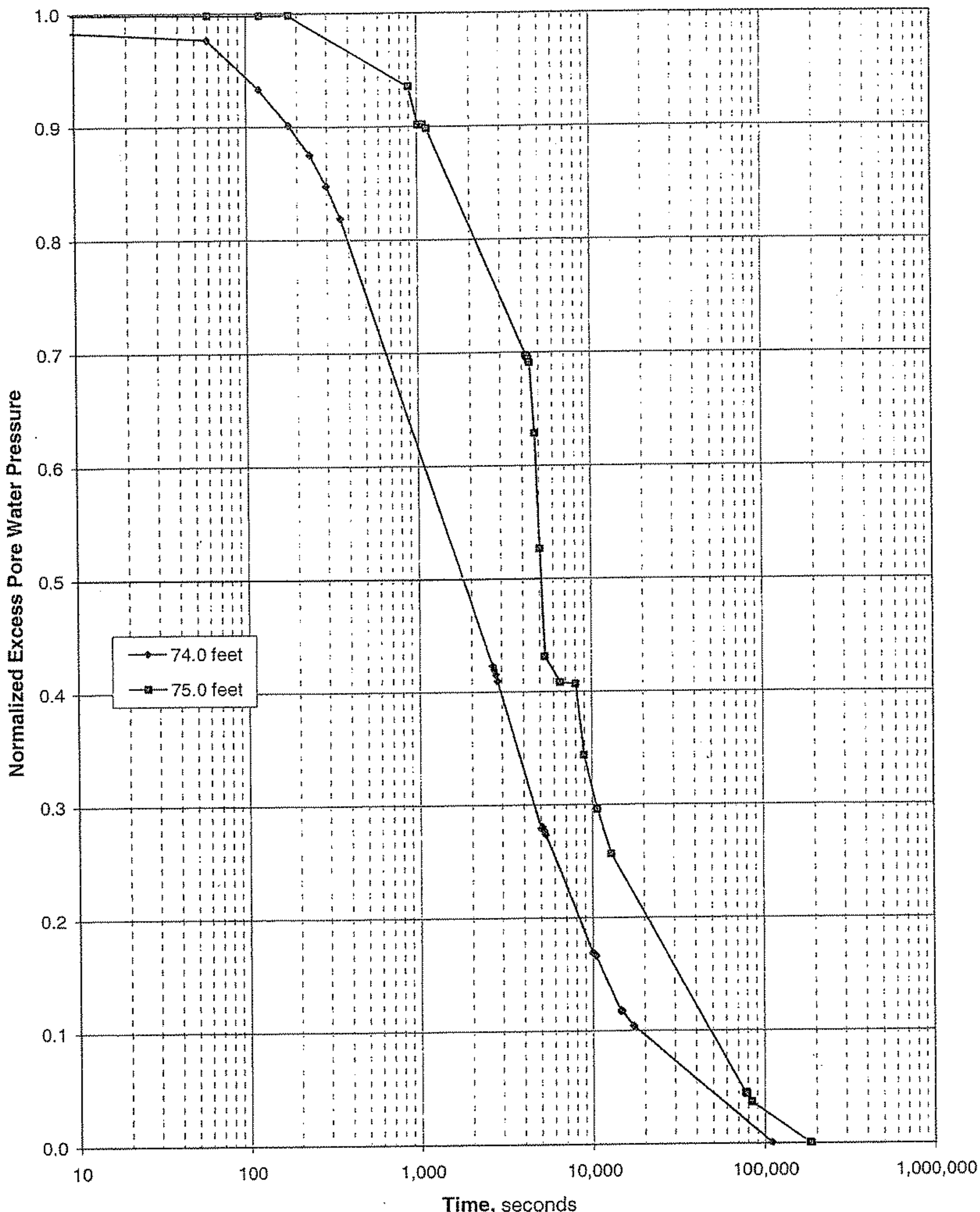
- ARDAMAN (1994)
- ARDAMAN (2001)
- ARDAMAN (2002)
- ARDAMAN (2003)
- ARDAMAN (2004)
- ARDAMAN (2005)
- ☒ TEST SITES IN 2006
- ▲ TEST SITES IN 2007
- **(2.0)** PIEZOMETRIC LEVEL ABOVE TOP OF CLAY IN FEET

LEACHATE LEVELS ON TOP OF PHOSPHATIC CLAY

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

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

DRAWN BY: CD	CHECKED BY:	DATE: 04/11/07
FILE NO. 06-212	APPROVED BY:	FIGURE: 8

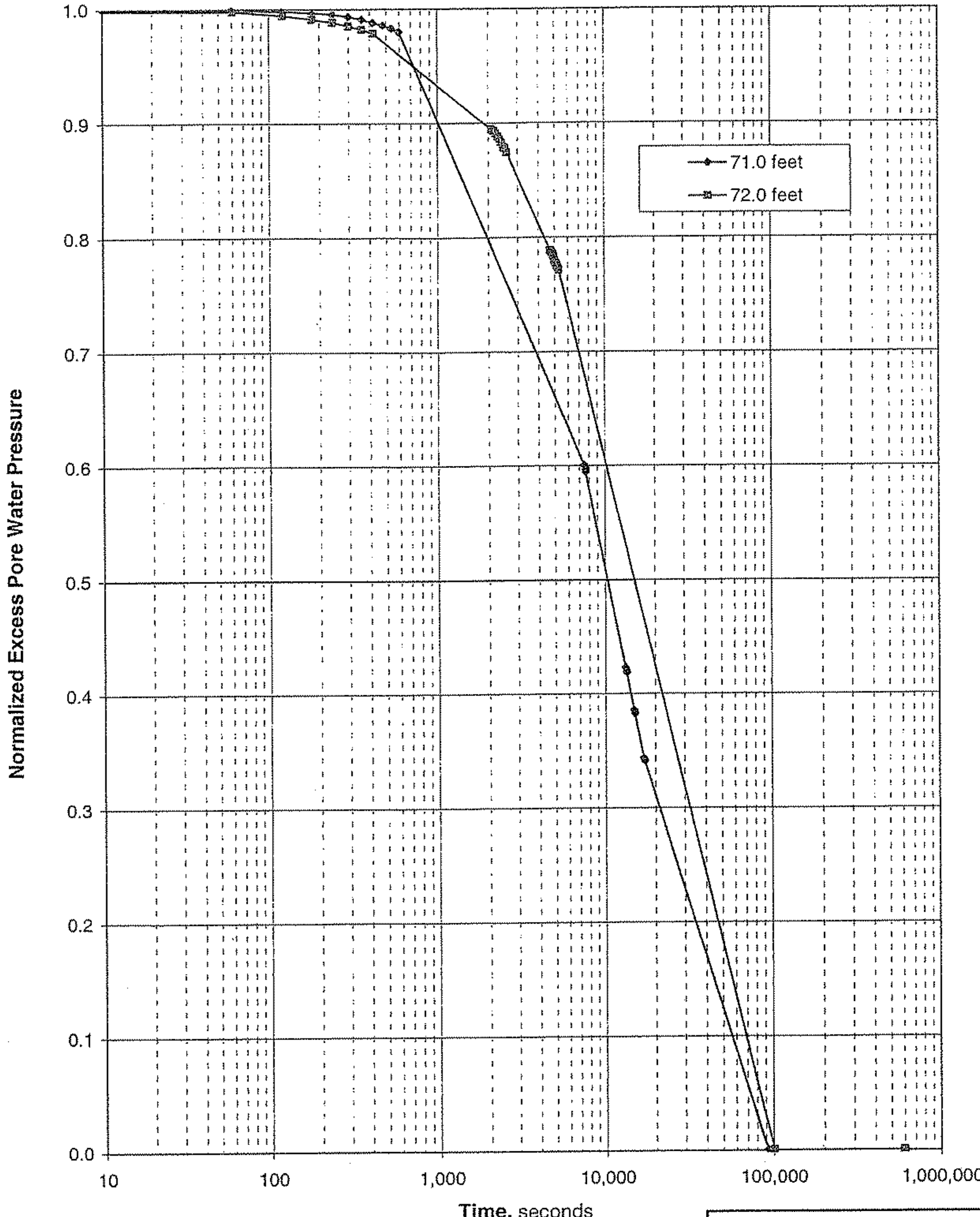


EXCESS PORE WATER PRESSURE DISSIPATION AT PC-10

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

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 9

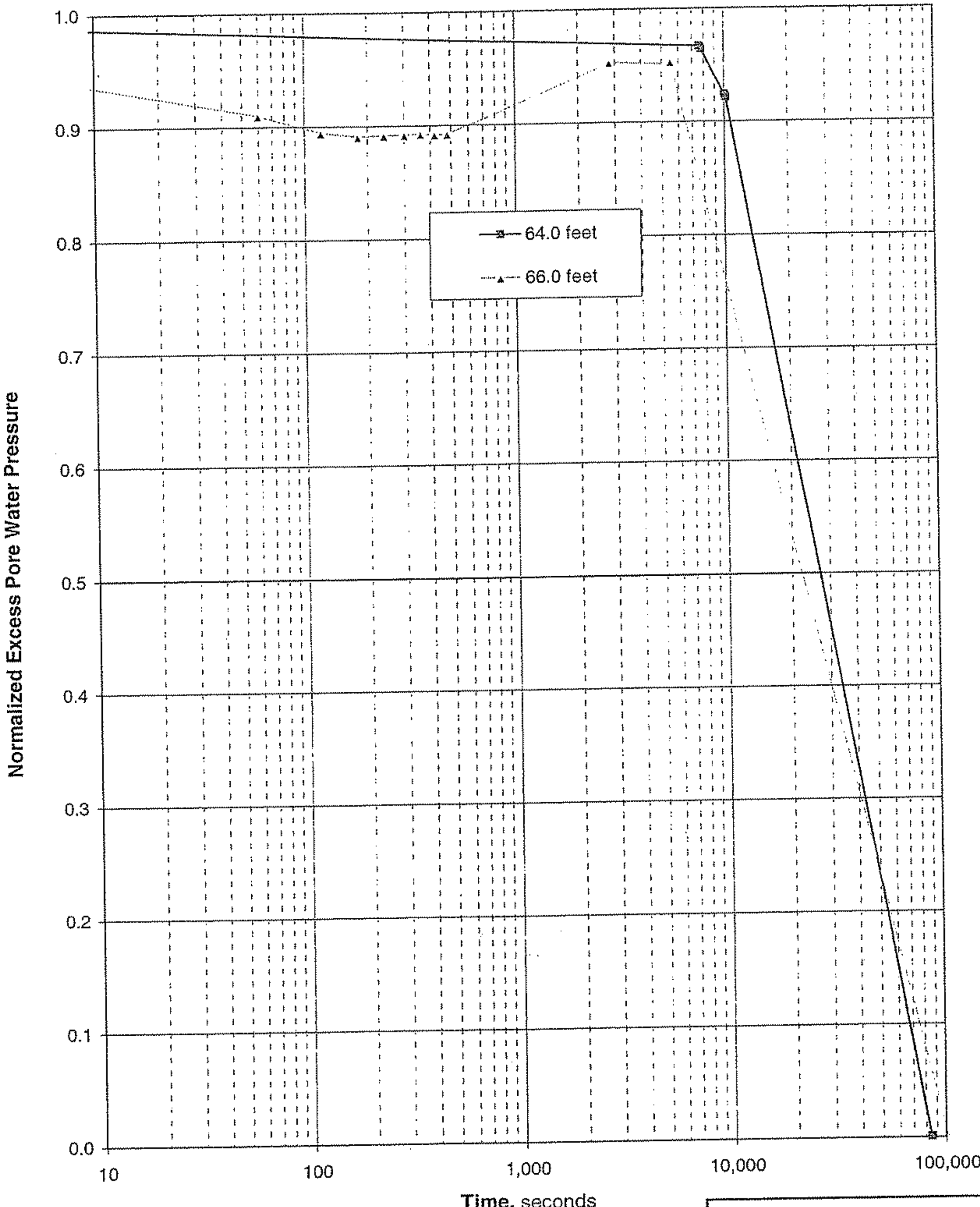


EXCESS PORE WATER PRESSURE DISSIPATION AT PC-1P

Ardaman & Associates, Inc
Geotechnical, Environmental and Material Consultants

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 10

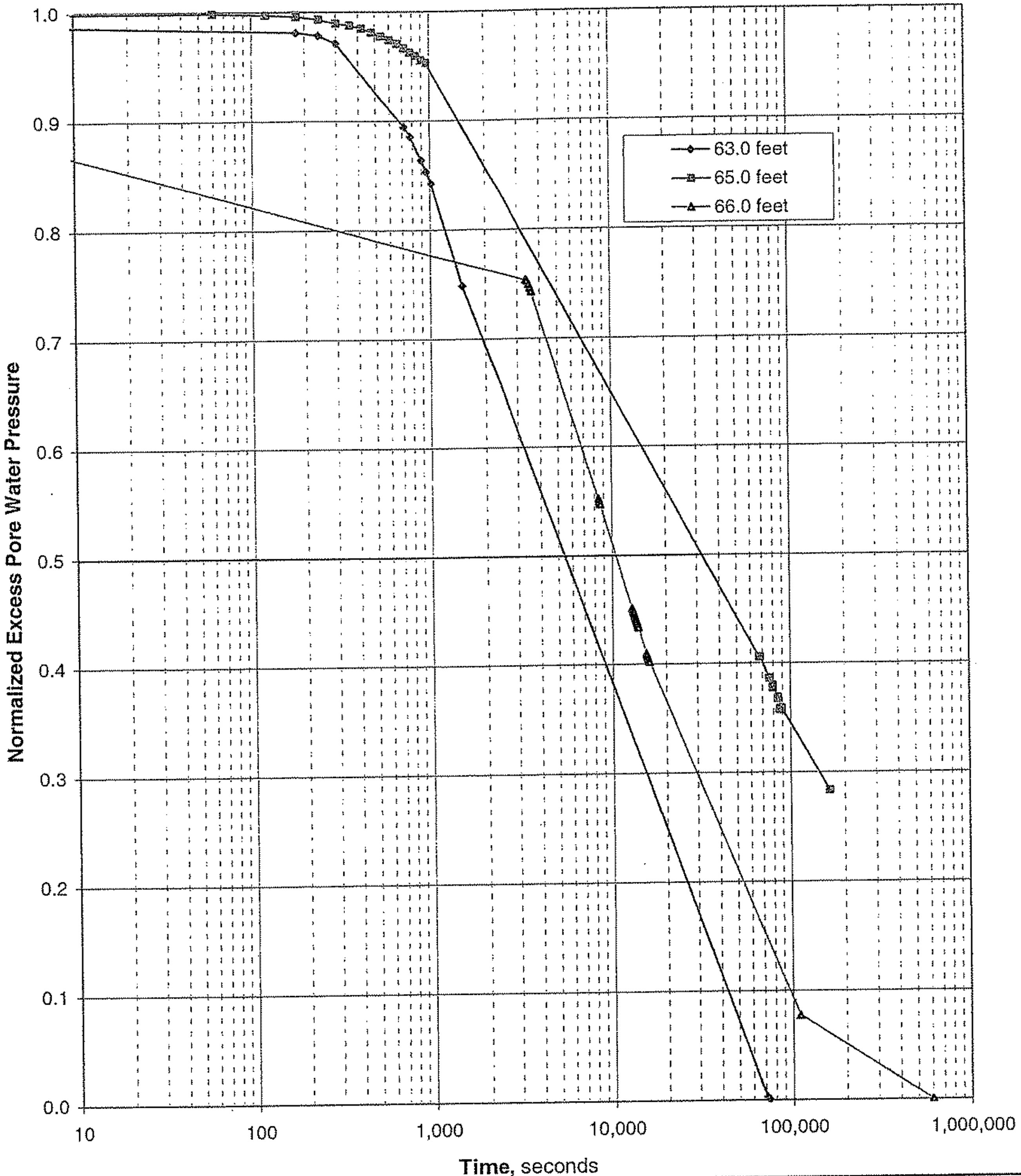


PORE WATER PRESSURE DISSIPATION AT PC-3G

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

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 11



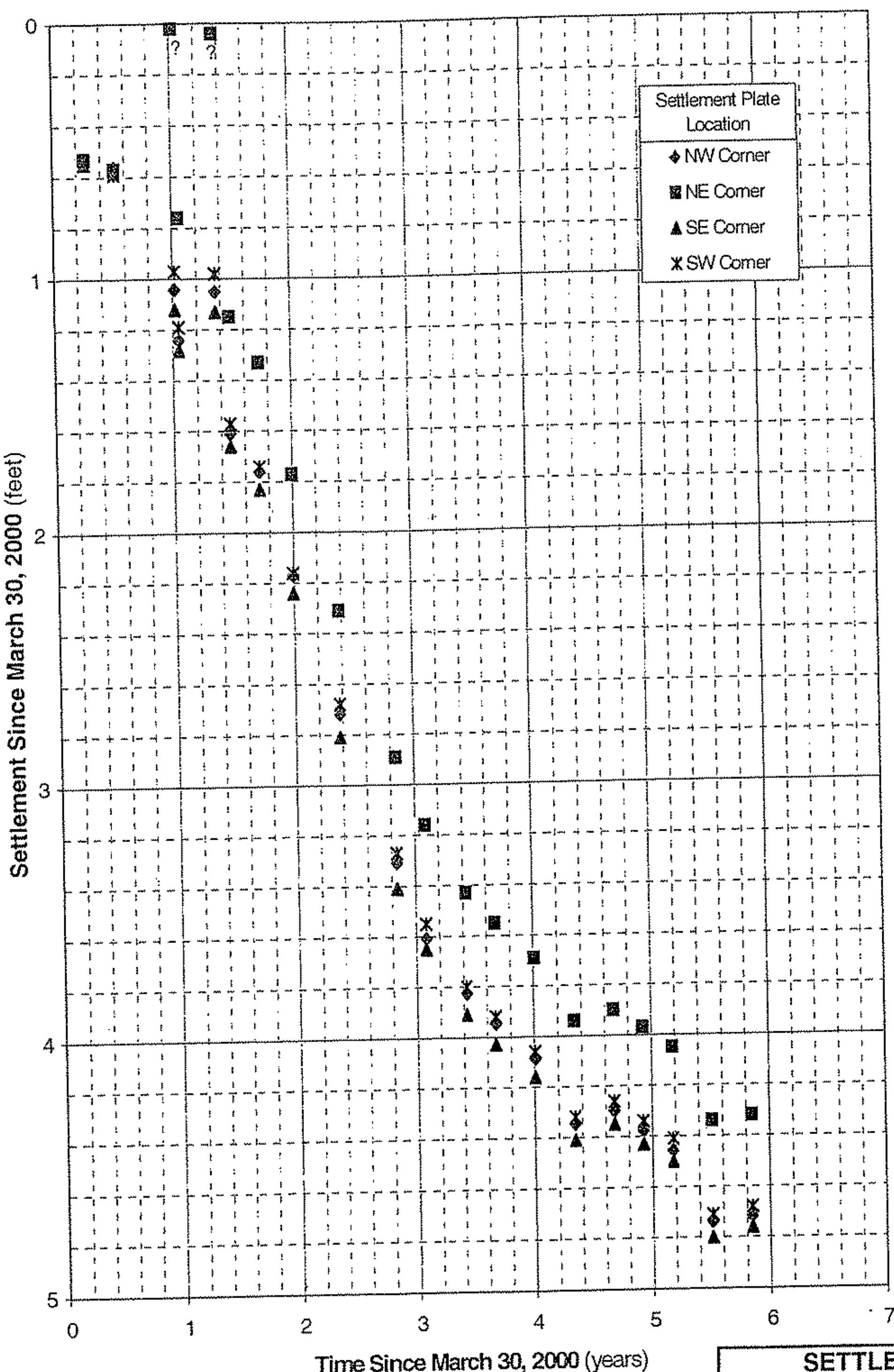
EXCESS PORE WATER PRESSURE DISSIPATION AT PC-4H

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

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

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	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 2007 ANNUAL MONITORING OF PHOSPHATIC CLAY, SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: PAK	CHECKED BY:	DATE: 03/28/2007
FILE NO. 06-212	APPROVED BY:	FIGURE: 13

Top Elevations of
Waste Phosphatic Clay beneath
Southeast County Landfill
Hillsborough County, Florida

October 29, 2007

Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

October 29, 2007
File Number 07-152

Hillsborough County Solid Waste Department
County Center, 24th Floor
601 East Kennedy Boulevard
Tampa, FL 33602

Attention: Ms. Megan J. Miller, P.E.

Subject: Top Elevations of Waste Phosphatic Clay beneath Southeast County Landfill, Hillsborough County, Florida

Gentlemen/Ladies:

As requested and authorized by the Hillsborough County Solid Waste Department (Hillsborough County), Ardaman & Associates, Inc., (Ardaman) has completed a study to estimate the top elevations of the waste phosphatic clay beneath the Southeast County Landfill (Southeast Landfill) in Hillsborough County, Florida. The objective of our study was to develop an elevation contour map for the top surface of the waste phosphatic clay based on the original condition reported by Ardaman in a 1983 study, the expected behavior of the waste phosphatic clay, and the latest condition observed by Ardaman at a limited number of test sites within the landfill area.

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.

Introduction

The Southeast Landfill is constructed directly above a waste phosphatic 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 constructed 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 waste phosphatic clay and to evaluate the technical feasibility of constructing a landfill within the 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 collected data and analyses, Ardaman concluded that a landfill could be constructed directly on top of the waste phosphatic clay. However, to maintain an adequate factor of safety for stability, it was recommended that: (i) the settling area be divided into phases: (ii) each phase be filled with refuse in lifts of no greater than 20 feet, and (iii) a waiting period of seven years be provided between placement of successive refuse lifts. The waiting period is to allow the underlying waste phosphatic clay to consolidate and gain strength under the weight of the latest refuse lift. The 1983 Ardaman report contained the initial elevations and thicknesses as well as the engineering properties of the waste phosphatic clay, and the predicted end-of-consolidation settlements of the waste phosphatic clay at different landfill loads.

The Southeast Landfill was divided into six operational phases designated Phases I through VI, as shown on a site plan in Figure 2. In accordance with Specific Condition No. 16f of the Landfill Operation Permit No. 35435-006-SO issued by the Florida Department of Environmental Protection (FDEP) on June 25, 2002, Ardaman has been performing piezocone soundings and piezoprobe installations at four test sites within the Phases I, III*, and IV areas annually since 2003. The latest field testing program was performed in February and March 2007, and the findings were documented in an Ardaman report titled "Geotechnical Study Associated with 2007 Annual Monitoring of the Phosphatic Clay Liner beneath the Southeast Landfill in Hillsborough County, Florida," dated May 16, 2007.

Ardaman has been performing the annual field tests in two stages. The first stage of work involved drilling through the refuse using an auger rig at four selected test sites, followed by performance of piezocone soundings by a cone penetrometer rig through the drainage sand and phosphatic clay layers, and into the underlying natural soils. The objectives of the first stage of work were to determine the depths and thicknesses of the waste phosphatic clay, and to establish the depths where the piezoprobes should be placed. The second stage of work involved drilling of additional test holes through the refuse adjacent to the piezocone sounding locations, installation of piezoprobes at different depths within the waste phosphatic clay layer, and monitoring of pore water pressure. The objectives of the second stage of work were to estimate the rates of pore water pressure dissipation and any remaining excess pore water pressure from landfill loading.

In 2007, three soil borings (designated TH-1O, TH-1P, and TH-6G) were also performed in the Phase I and VI areas to determine the depths and thicknesses of the waste phosphatic clay, and to obtain undisturbed samples of the waste phosphatic clay for laboratory measurement of undrained shear strengths.

Original Conditions of Waste Phosphatic Clay

The geotechnical study conducted by Ardaman between 1981 and 1983 included drilling of 29 soil borings within the current boundary of Phases I through VI of the Southeast Landfill. Based on the soil boring data, an isopach map that showed the waste phosphatic clay thickness was developed. A digitized version of the isopach map is reproduced in Figure 2. As shown, the waste phosphatic clay thickness varied from 4 feet along the northern, eastern, and southern boundaries to approximately 18 feet within the Phase VI area. The western boundary had a waste phosphatic clay thickness that varied from 4 to 14 feet.

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* In 2006, the test site in the Phase III area was actually performed within the Phase II area because refuse was being placed in the Phase III area at the time of our field testing program.

Based on the soil boring data obtained between 1981 and 1983, the top elevations (referenced in NGVD) of the waste phosphatic clay, prior to operation of the Southeast Landfill, are also displayed adjacent to the soil boring locations on the isopach map in Figure 2. As shown, the top surface of the waste phosphatic clay within the Phase I through VI areas was relatively flat, with elevations typically varied only between +122 and +124 feet (NGVD). The lowest point was located at the northwest corner of the landfill and near the west end of the Phase V area, with an elevation of +121 feet (NGVD). The highest point was located on the east side of the landfill and near the northeast corner of the Phase II area, with an elevation of +127 feet (NGVD).

Predicted Consolidation Settlements of Waste Phosphatic Clay

The waste phosphatic clay layer will compress under the weight of the refuse. The magnitude of compression depends on the thickness and density of the refuse as well as the thickness of the waste phosphatic clay, with greater compression expected to occur in areas where the refuse has a greater height and density, and the waste phosphatic clay has a greater thickness. The rate of compression is a function of the drainage distance (which is related to the thickness of the waste phosphatic clay and the drainage boundary condition) of the waste phosphatic clay layer and the coefficient of consolidation of the waste phosphatic clay deposit.

In the 1983 study, Ardaman computed compression of the waste phosphatic clay under varying load intensity and for different phosphatic clay thicknesses after consolidation settlement has completed (i.e., the excess pore water pressure in the waste phosphatic clay has essentially dissipated). The calculated consolidation settlement curve figure, as documented in the 1983 Ardaman report, is reproduced and included in Appendix I. If the loading condition and original waste phosphatic clay thickness are known, the consolidation settlement or current thickness of waste phosphatic clay can be predicted using these consolidation settlement curves. As shown on the figure, the consolidation settlement of an 18-foot thick phosphatic clay layer under a load of 2 tons per square foot (tsf) could exceed 7 feet. A load of 2 tsf corresponds to a landfill height of approximately 50 to 60 feet.

Landfill Loading

We understand that a mixture of refuse and ash has been disposed of at the Southeast Landfill. The latest landfill heights (i.e., the refuse/ash thicknesses) in the Phase I through VI areas can be estimated from the auger borings that drilled through the refuse/ash in February and March 2007, and also from the topographic survey performed in July 2006.

For the annual field testing program in 2007, the four test sites were designated PC-1O, PC-1P, PC-3G, and PC-4H. PC-1O and PC-1P were located on the western and eastern sides of the Phase I area, respectively, and had a refuse/ash thickness of approximately 70 feet. PC-3G was located near the eastern boundary of the Phase III area, and had a refuse/ash thickness of approximately 60 feet. PC-4H was located near the center of the Phase IV area, and also had a refuse/ash thickness of approximately 60 feet. The two soil borings (i.e., TH-1O and TH-1P) performed in the Phase I area in 2007, which were located adjacent to PC-1O and PC-1P, also confirmed the refuse/ash thickness to be approximately 70 feet. The soil boring (i.e., TH-6G) performed in the Phase VI area in 2007 indicated a refuse/ash thickness of approximately 50 feet.

Based on observations at the test site locations and a review of the latest topographic survey, the Phase I and II areas had a refuse/ash thickness of approximately 70 feet, the Phase III and

IV areas had a refuse/ash thickness of approximately 60 feet, and the Phase V and VI areas had a refuse/ash thickness of approximately 50 feet. According to Jones Edmunds, the refuse/ash mixture at the Southeast Landfill has a unit weight of 74 pounds per cubic foot (pcf). At this unit weight, the corresponding refuse/ash loads in the Phase I/II, III/IV, and V/VI areas beneath the landfill top were computed to be approximately 2.6, 2.2, and 1.9 tsf. Beneath the outside slope of the landfill, the refuse/ash thickness and applied load are less than those at the landfill top.

Based on information provided by Hillsborough County, the latest loading event for each phase is presented below:

Phase	Beginning of Filling of Latest Lift	Completion of Filling of Latest Lift	Waiting Period Prior to Filling of the Latest Lift (years)
I	Mar 2003	Aug 2004	7.9
II	Aug 2004	Aug 2005	7.0
III	Aug 2005	Aug 2006	14.5
IV	Aug 2006	Dec 2006	12.0
V/VI	Dec 2006	Dec 2008	7.6

X

As shown in the table above, the refuse/ash mixture is currently being placed in the combined Phase V/VI area. The filling schedule further indicated that the elapsed time since the completion of filling of the latest lifts was less than seven years.

Observed Conditions of Waste Phosphatic Clay

The thicknesses and elevations of the waste phosphatic clay at the test site locations, which covered the Phase I, II, III, IV, and VI areas, are summarized in Table 1. The data for the Phase I, III, IV and VI areas were obtained in February and March 2007, whereas the data for the Phase II area was obtained in February 2006. No data were available for the Phase V area.

As shown in Table 1, the test sites in the western (i.e., PC-1O and TH-1O) and eastern (i.e., PC-1P and TH-1P) portions of the Phase I area had waste phosphatic clay thicknesses of 7.5 to 8.5 feet and 3.6 to 4.5 feet, respectively. In the Phase II area, where a piezocone sounding (i.e., PC-3F) was performed in February 2006, the waste phosphatic clay was documented to have a thickness of 7.8 feet. The test site near the eastern boundary of the Phase III area (i.e., PC-3G) showed a waste phosphatic clay thickness of approximately 7.2 feet. In the Phase IV area, the waste phosphatic clay had a thickness of approximately 7.6 feet at the location of PC-4H. In the Phase VI area, where the original waste phosphatic clay was thickest (as much as 18 feet), the soil boring (i.e., TH-6G) indicated a waste phosphatic clay thickness of approximately 9.6 feet.

As shown in Table 1, the top and bottom elevations of the waste phosphatic clay ranged from +112.2 to +118.9 feet (NGVD) and from +104.6 to +115.1 feet (NGVD), respectively. The lowest top elevations were recorded at PC-1O/TH-1O and PC-4H, and the lowest bottom elevations were documented at PC-1O/TH-1O, PC-4H, and TH-6G. The waste phosphatic clay thicknesses at the test site locations varied between 3.6 and 9.6 feet. As indicated previously, the study completed by Ardaman in 1983 indicated that the waste phosphatic clay had a typical top elevation of +122 to +124 feet (NGVD) and a typical thickness that ranged from 4 to 18 feet within the settling area.

Foundation Soil Settlement

The change in top elevation of the waste phosphatic clay is a result of compression of the waste phosphatic clay plus any foundation soil settlement below the waste phosphatic clay. Because of the relatively high compressibility of the waste phosphatic clay, it is expected that a majority of the change in top elevation is caused by compression of the waste phosphatic clay. However, during the field testing program associated with permit renewal in 2001, a piezocone sounding and subsequent soil borings located near PC-1O in the western portion of the Phase I area indicated the presence of a peat layer beneath the waste phosphatic clay at this particular test site location, which could contribute to significant foundation soil settlement.

Soil borings performed in 2001 indicated that the peat deposit covered an area with a radius of approximately 200 feet. The approximate boundary of the peat deposit is displayed in Figure 2. As shown, the peat deposit covers portions of the Phase I and VI areas, and encompasses the PC-1O and PC-4H test sites. By comparing the bottom elevations of the waste phosphatic clay in the 1983 Ardaman report to those documented as part of 2007 annual monitoring program at these two test site locations, foundation soil settlement of approximately 3 to 4 feet were noted. No significant foundation soil settlement was observed at other test sites located outside the area of peat deposit.

Comparisons between Original and Observed Conditions

Table 2 presents the thicknesses and elevations of the waste phosphatic clay in 1983 and in 2006/2007 at the latest test site locations. As shown, the top elevations of the waste phosphatic clay at the test site locations varied between +121 and +123 feet (NGVD) in 1983. Near the same locations in 2006/2007, the top surface of the waste phosphatic clay has settled to elevations ranging from +112 to +119 feet (NGVD), i.e., approximately 5 to 10 feet lower than the original top elevations in 1983. As indicated above, the drop in top elevations was a result of consolidation settlements of the waste phosphatic clay and, at the locations of PC-1O and PC-4H, also a result of foundation soil settlement.

At the latest test site locations, the waste phosphatic clay thicknesses ranged from 8.8 to 17.9 feet in 1983, and from 3.6 to 9.6 feet in 2007, i.e., a thickness decrease of approximately 5 to 8 feet.

As shown in Table 2, the bottom elevations of the waste phosphatic clay did not appear to have lowered significantly except at the locations of PC-1O and PC-4H where foundation soil settlement of 3 to 4 feet were noted. As indicated previously, the foundation soil settlement at these two locations could be attributed to the underlying peat deposit.

Comparisons between Predicted and Observed Consolidation Settlements

Table 3 presents the predicted versus observed waste phosphatic clay thicknesses at the latest test site locations. The predicted waste phosphatic clay thicknesses were calculated using estimated landfill loads and the consolidation settlement curves in Appendix I. These consolidation settlement curves assumed that dissipation of excess pore water pressure has essentially completed.

As shown in Table 3, the predicted consolidation settlements of the waste phosphatic clay ranged from approximately 3 to 7 feet, resulting in predicted waste phosphatic clay thicknesses of approximately 6 to 11 feet after consolidation settlement has essentially completed. The observed waste phosphatic clay thicknesses at the same locations during the latest field testing program were documented to be between 4 and 10 feet, and were typically 40 to 60 percent of the original waste phosphatic clay thicknesses in 1983. Accordingly, the observed waste phosphatic clay settlements at the test site locations ranged from approximately 4 to 8 feet. On the average, the latest observed waste phosphatic clay settlement was approximately 25 percent greater than the predicted settlement at the end of consolidation, which suggests that the waste phosphatic clay was more compressible than anticipated and/or the actual unit weight of the refuse/ash mixture was greater than 74 pcf.

It should be noted that the waste phosphatic clay thicknesses observed at the latest monitoring event may not represent the condition at the end of consolidation settlement and, therefore, will continue to decrease as excess pore water pressure continues to dissipate. This will especially be the case in the Phase III and IV areas where filling has only been recently completed in the later part of 2006, and high excess pore water pressures were noted in the early part of 2007.

Top Elevations of Waste Phosphatic Clay

To generate an elevation contour map for the existing top surface of the waste phosphatic clay, Ardaman estimated the settlement at each of the 29 soil boring locations that were drilled between 1981 and 1983 based on the current landfill loads and the predicted consolidation settlements, taking into consideration that the latest observed settlements at the test site locations were approximately 25 percent greater than the predicted settlements. +25%

At each of the 29 boring locations, the latest top elevation of the waste phosphatic clay can be computed using the following equation:

Latest Top Elevation = Original Top Elevation – Projected Clay Settlement – Foundation Soil Settlement

The original top elevation represents the waste phosphatic clay surface elevation documented between 1981 and 1983. The projected clay settlement is based on the predicted consolidation settlement under the current landfill load and a correction factor of 1.25 to reflect the difference between predicted and observed settlements at the test site locations. The foundation soil settlement was considered negligible except at three previous soil boring locations that were in the vicinity of PC-1O and PC-4H. At these three locations, the foundation soil settlement was assumed to be 4 feet. +4 feet for PEAT

Calculations of the latest top elevations and thicknesses of the waste phosphatic clay are presented in Appendix II. As shown, the projected clay settlements at the 29 previous soil boring locations ranged from 0.6 to 8.8 feet. As expected, maximum clay settlement was projected to occur within the Phase VI area where the original waste phosphatic clay was thickest (18 feet). Minimal clay settlement was projected to occur near the toe of the landfill slope where the refuse load was smaller and the original waste phosphatic clay was thinner.

An elevation contour map that shows the projected top surface of the waste phosphatic clay is displayed in Figure 3. As shown, the waste phosphatic clay surface elevations varied between approximately +110 and +125 feet (NGVD). The lowest top elevation was found to occur near the boundary between the Phase I and IV areas where the peat deposit was encountered. As indicated previously, the top elevations of the waste phosphatic clay at PC-1O and PC-4H were established at approximately +113 and +112 feet (NGVD), respectively. For comparison, the top elevation of the waste phosphatic clay at TH-6G in the Phase VI was encountered at approximately +115 feet (NGVD).

Closure

It should be noted that the elevation contours for the top surface of the waste phosphatic clay was generated based on the expected behavior of the waste phosphatic clay and actual observations at a limited number of test site locations. In addition, the top surface of the waste phosphatic clay is expected to change continuously as refuse is placed on top of the landfill and dissipation of excess pore water pressure occurs in the waste phosphatic clay. Based on available data obtained to-date, the low area of the waste phosphatic clay surface appears to occur in the Phase I and IV areas, and extends into the Phase VI area.

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.

Dinh T. Nguyen, E.I.T.
Assistant Project Engineer

Francis K. Cheung, P.E.
Senior Project Engineer
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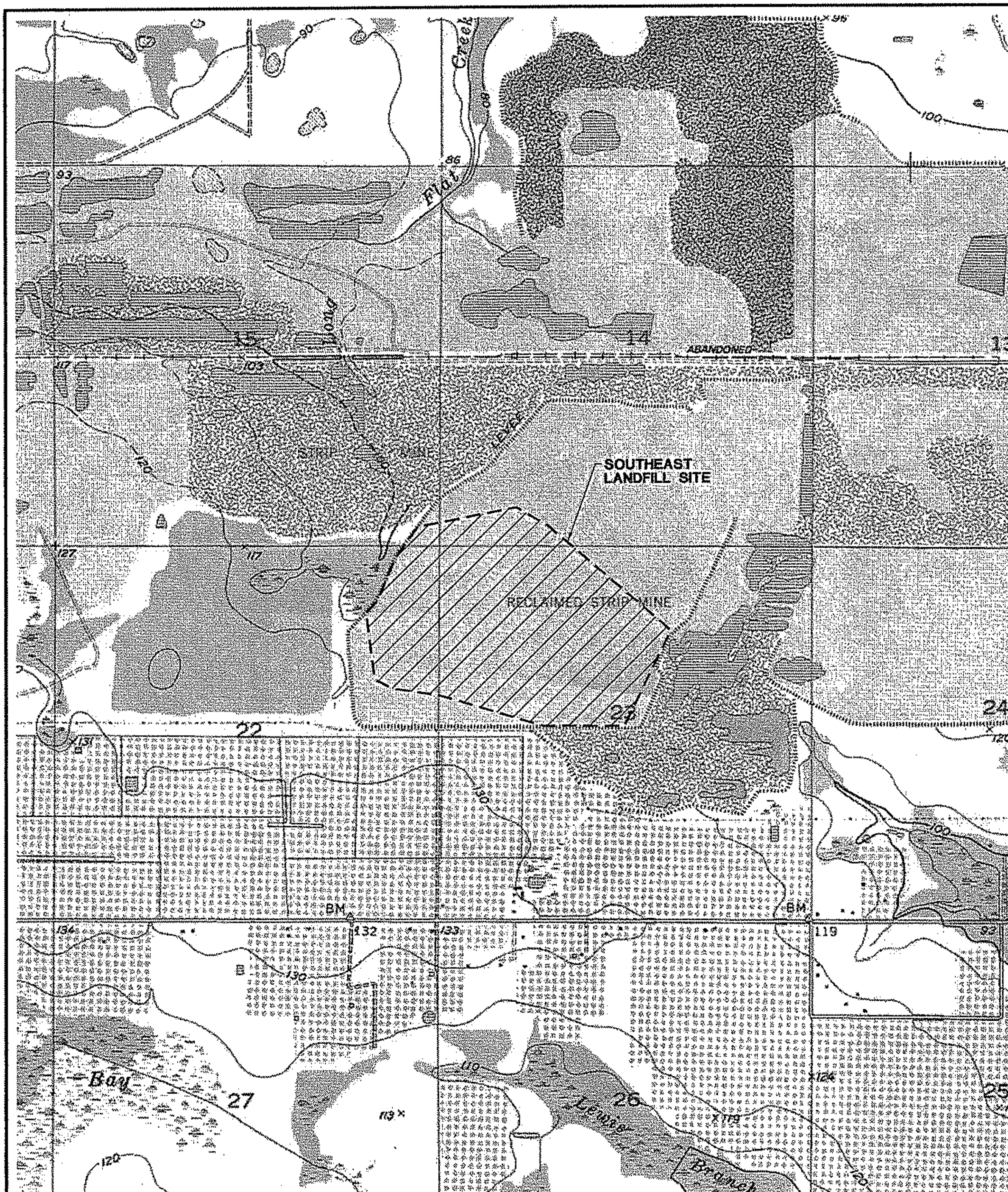
Enclosures



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

PHOSPHATIC CLAY LINER MAP
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: BTW	CHECKED BY:	DATE: 10/09/07
FILE NO. 07-152	APPROVED BY:	FIGURE: 1

T:\Corporate\07\07-152\Site Location Map.dwg 10/29/2007 2:28:14 PM, richard.clemons

Table 1

Latest Elevations and Thicknesses of Waste Phosphatic Clay

Phase	Location	Year	Waste Phosphatic Clay		
			Top Elevation (feet, NGVD)	Bottom Elevation (feet, NGVD)	Thickness (feet)
I	PC-1O	2007	+113.2	+105.7	7.5
	TH-1O	2007	+113.2*	+104.7*	8.5
	PC-1P	2007	+118.7	+115.1	3.6
	TH-1P	2007	+117.6**	+113.1**	4.5
II	PC-3F	2006	+116.3	+111.2	5.1
III	PC-3G	2007	+118.9	+111.7	7.2
IV	PC-4H	2007	+112.2	+104.6	7.6
VI	TH-6G	2007	+114.5***	+104.9***	9.6

* These elevations were based on a land surface elevation of +184.2 feet (NGVD), as surveyed at the nearby PC-1O location.

** These elevations were based on a land surface elevation of +187.6 feet (NGVD), as surveyed at the nearby PC-1P location.

*** These elevations were based on the site topographic map surveyed in July 2006.

Table 2

Comparisons between Original and Latest Waste Phosphatic Clay Elevations and Thicknesses

Phase	Location	Waste Phosphatic Clay in 1983			Waste Phosphatic Clay in 2006/2007		
		Top Elevation (feet, NGVD)	Bottom Elevation (feet, NGVD)	Thickness (feet)	Top Elevation (feet, NGVD)	Bottom Elevation (feet, NGVD)	Thickness (feet)
I	PC-1O	+121.1	+108.7	12.4	+113.2	+105.7	7.5
	TH-1O	+121.1	+108.7	12.4	+113.2*	+104.7*	8.5
	PC-1P	+122.3	+113.5	8.8	+118.7	+115.1	3.6
	TH-1P	+122.3	+113.5	8.8	+117.6**	+113.1**	4.5
II	PC-3F	+123.2	+111.2	12.0	+116.3	+111.2	5.1
III	PC-3G	+123.0	+109.5	13.5	+118.9	+111.7	7.2
IV	PC-4H	+121.0	+107.3	13.7	+112.2	+104.6	7.6
VI	TH-6G	+121.5	+103.6	17.9	+114.5***	+104.9***	9.6

* These elevations were based on a land surface elevation of +184.2 feet (NGVD), as surveyed at the nearby PC-1O location.

** These elevations were based on a land surface elevation of +187.6 feet (NGVD), as surveyed at the nearby PC-1P location.

*** These elevations were based on the site topographic map surveyed in July 2006.

Table II.2
Hillborough County Southeast Landfill
Estimated Clay Liner Thickness and Elevation

Phase	Survey Point	1983 Survey			Completion Date of Last Loading Event	Top of Refuse Elevation (ft)	Refuse Thickness(1) (ft)	Estimated Refuse Load(2) (tsf)	Estimated Clay Settlement(3) (ft)	Estimated Clay Thickness(4) (ft)	Estimated Top Clay Elevation(5) (ft, NGVD)	Bottom Elevation Clay (ft, NGVD)
		Top Elevation (ft, NGVD)	Bottom Elevation (ft, NGVD)	Thickness (ft)								
I	36	122.0	118.5	3.5	Aug-04	135	24	0.89	0.6	2.9	121.4	118.5
	35	123.0	111.0	12.0		204	93	3.44	5.9	6.1	117.1	111.0
	28	122.0	115.5	6.5		177	66	2.44	2.3	4.3	119.8	115.5
	27	122.3	109.3	13.0		197	86	3.18	6.1	6.9	116.2	109.3
	20	122.9	116.9	6.0		146	35	1.30	1.6	4.4	121.3	116.9
	19	122.0	110.0	12.0		190	79	2.92	5.6	6.4	116.4	110.0
	12*	120.6	107.1	13.5		170	59	2.18	5.5	8.0	109.6	101.6
	10*	121.0	109.0	12.0		181	70	2.59	5.4	6.6	110.1	103.5
	5	121.8	114.8	7.0		176	65	2.41	2.5	4.5	119.3	114.8
	2	122.0	116.0	6.0		155	44	1.63	1.8	4.3	120.3	116.0
II	48	124.0	118.0	6.0	Aug-05	138	13	0.48	1.0	5.0	123.0	118.0
	47	124.0	114.0	10.0		150	25	0.93	3.5	6.5	120.5	114.0
	46	124.3	113.3	11.0		177	52	1.92	4.8	6.3	119.6	113.3
	45	126.5	120.5	6.0		135	10	0.37	0.9	5.1	125.6	120.5
	42	122.9	114.9	8.0		182	57	2.11	3.0	5.0	119.9	114.9
	41	123.0	111.0	12.0		196	71	2.63	5.5	6.5	117.5	111.0
	40	124.0	113.5	10.5		174	49	1.81	4.3	6.3	119.8	113.5
	34	123.1	109.6	13.5		189	64	2.37	5.4	8.1	117.7	109.6
III	33	124.1	112.1	12.0	Aug-06	156	31	1.15	4.3	7.8	119.9	112.1
	26	122.0	107.0	15.0		202	77	2.85	7.4	7.6	114.6	107.0
	25	121.9	106.4	15.5		197	72	2.66	7.5	8.0	114.4	106.4
	24	125.5	116.5	9.0		141	16	0.59	2.3	6.8	123.3	116.5
IV	18	121.1	107.1	14.0	Dec-07	176	62	2.29	6.4	7.6	114.7	107.1
	9*	121.0	107.5	13.5		169	55	2.04	5.5	8.0	112.0	104.0
VI	17	121.8	104.3	17.5	Mar-03	159	52	1.92	8.1	9.4	113.7	104.3
	8	122.0	104.0	18.0		159	52	1.92	8.8	9.3	113.3	104.0
V	16	123.0	109.0	14.0	Mar-03	149	40 to 50	1.67	6.0	8.0	117.0	109.0
	7	122.1	108.6	13.5		146		1.67	5.8	7.8	116.4	108.6
	4	121.7	106.7	15.0		140		1.67	6.4	8.6	115.3	106.7

(1) Estimated from the 2007 topographic map unless available from cone data
(2) Base on the estimated waste thickness and a refuse unit weight of 74 pcf
(3) Estimated Settlement = (Settlement from 1983 settlement curve)*1.25
(4) Original thickness minus estimated clay settlement
(5) Bottom Elevations plus the estimated clay thickness
* Locations where organic deposit was encountered - Foundation settlement was included

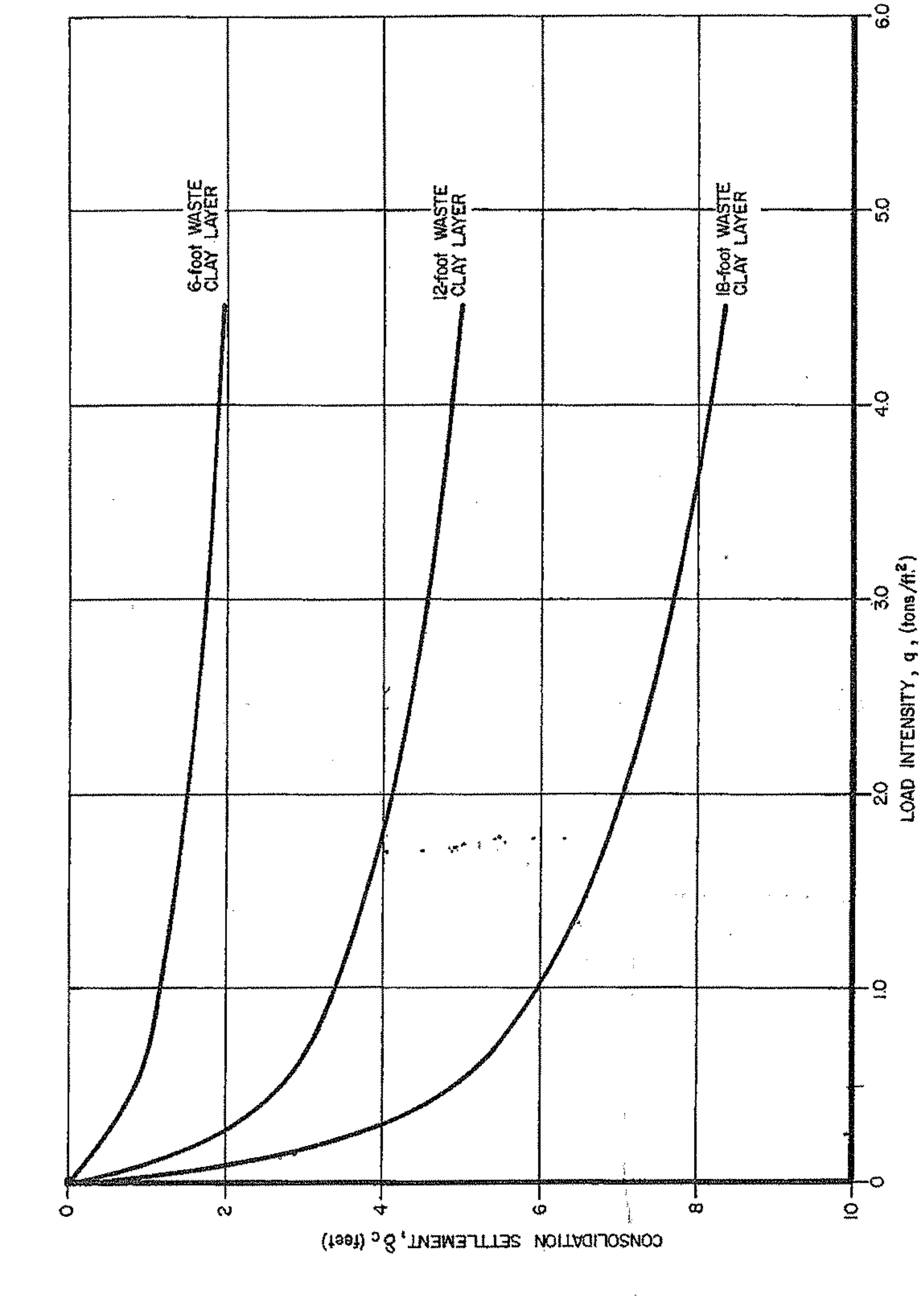
Table 3

Comparisons between Predicted and Observed Waste Phosphatic Clay Settlements and Thicknesses

Phase	Location	Original Clay Thickness in 1983 (feet)	Refuse/Ash Thickness (feet)	Applied Load on Top of Clay (psf)	Predicted Consolidation Settlement of Clay Layer (feet)	Predicted Clay Thickness Upon Completion of Consolidation Settlement (feet)	Observed Clay Thickness (feet)	Observed Clay Settlement (feet)
I	PC-1O	12.4	71	5,326	4.7	7.7	7.5	4.9
	TH-1O	12.4	71	5,326	4.7	7.7	8.5	3.9
	PC-1P	8.8	69	5,178	3.0	5.8	3.6	5.2
	TH-1P	8.8	70	5,252	3.0	5.8	4.5	4.3
II	PC-3F	12.0	55	4,142	4.2	7.8	5.1	6.9
III	PC-3G	13.5	64	4,808	5.2	8.3	7.2	6.3
IV	PC-4H	13.7	62	4,660	5.3	8.4	7.6	6.1
VI	TH-6G	17.9	52	3,920	7.1	10.8	9.6	8.3

Appendix I

Consolidation Settlement Curves from Ardaman 1983 Study



VARIATION OF CONSOLIDATION SETTLEMENT WITH LOAD INTENSITY FOR WASTE PHOSPHATIC CLAY LAYERS OF VARIOUS THICKNESS

Ardaman & Associates, Inc.
Consulting Engineers in Soil Mechanics, Foundations, and Material Testing

HYDROGEOLOGICAL INVESTIGATION
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: T.S.	CHECKED BY: [illegible]	DATE: 1/20/83
FILE NO. 81-159	APPROVED BY: H.G. Stangland	

FIGURE 6.11

Appendix II

Worksheet Calculations for Waste Phosphatic Clay Elevations and Thicknesses

Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

May 16, 2007
File Number 06-212

Jones Edmunds & Associates, Inc.
324 South Hyde Park Avenue
Suite 250
Tampa, FL 33606

Attention: Mr. Joseph O'Neill, P.E.
Solid Waste Department Manager

Subject: Stability Analyses of Proposed Landfill Design Sections in Phase I and V/VI Areas at Southeast Landfill in Hillsborough County, Florida

Gentlemen:

As requested, Ardaman & Associates, Inc. (Ardaman) has completed a field exploration and laboratory testing program, and has performed stability analyses of the proposed landfill design sections in the Phase I and V/VI areas at the Southeast Landfill in Hillsborough County, Florida. The objective of the study was to determine whether the phosphatic clay deposits in the Phase I and V/VI areas have gained sufficient strength and are ready to receive the next lift of refuse/ash.

For this study, Ardaman has reviewed the piezocone sounding and piezoprobe test data recently collected as part of the annual monitoring program required by Specific Condition No. 16f of the Landfill Operation Permit No. 35435-006-SO. In addition, soil borings were performed to obtain undisturbed samples of the underlying phosphatic clay deposit to allow measurements of undrained shear strengths and other soil properties in the laboratory.

Based on the latest topographic survey provided to us by Jones Edmunds, the crest elevations (i.e., the top of the landfill slope) in the Phase I and V/VI areas were at approximately +180 and +140 feet (NGVD), respectively. 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., approximately 3 to 4 years ago), and the prior lift of refuse/ash in the Phase V/VI areas was placed between April 1999 and March 2003 (i.e., approximately 4 to 8 years ago). Presently, the landfill operation has moved or will be moving shortly into the Phase V/VI areas. Filling of the next lift of refuse/ash in the Phase V/VI areas to reach a crest elevation of +160 feet (NGVD) is projected to occur over a period of approximately 2 years. After the Phase V/VI areas are filled, the county would then return to the Phase I area for placement of the next lift of refuse/ash to reach a crest elevation of +200 feet (NGVD). According to Hillsborough County, the crest elevation of the perimeter dike at the toe of the landfill is at approximately +129 feet (NGVD).

Field Exploration Program

Three soil borings, designated TH-1O, TH-1P, and TH-6G, were performed by Ardaman in the Phase I and VI areas. TH-1O was drilled in the western portion at the north side of the Phase I area, adjacent to the piezocone test site designated PC-1O, which was performed as part of the 2007 annual monitoring program. TH-1P was advanced near the eastern end of the Phase I area, adjacent to the piezocone test site designated PC-1P, also performed as part of the 2007 annual monitoring program. TH-6G was drilled by the leachate pumping station near the southeastern corner of the Phase VI area. The borings were advanced through the entire depths of refuse/ash and drainage sand layer to recover samples of the underlying phosphatic clay for laboratory testing. A total of six undisturbed samples were recovered, with two samples from TH-1O, one sample from TH-1P, and three samples from TH-6G.

Depths to and Thicknesses of Phosphatic Clay

In TH-1O, TH-1P, and TH-6G, the top of phosphatic clay was encountered at depths of 71, 70, and 52 feet below landfill surface, and the phosphatic clay thickness was documented to be 8.5, 4.5, and 9.6 feet, respectively. The depth and thickness measurements at TH-1O and TH-1P are generally consistent with those inferred from the piezocone soundings performed at PC-1O and PC-1P in February/March of 2007. Annual piezocone soundings were not required nor performed in the Phase V/VI areas.

TH-6G was drilled at a location where the original phosphatic clay deposit was known to be thickest (14 to 18 feet). The original phosphatic clay deposit was thinnest (4 to 8 feet) along the south side of the Phase I area where TH-1P was drilled. Along the west side of the Phase I area where TH-1O was drilled, the original phosphatic clay thickness could have been as much as 8 to 10 feet.

Based on the latest piezocone soundings and test borings, the existing phosphatic clay thickness along the south side of Phase I area was judged to be less than 4 feet and that along the west side of the Phase I area was determined to be less than 8 feet. In the Phase V/VI areas, the existing phosphatic clay thickness was judged to be approximately 10 feet.

Undrained Shear Strengths of Phosphatic Clay

To measure the undrained shear strengths of the phosphatic clay for stability analyses, Ardaman performed unconfined compression tests, unconsolidated-undrained triaxial tests, and direct simple shear tests on the recovered undisturbed phosphatic clay samples. Results of these undrained shear strength measurements are as follows:

Test Hole	Sample	Depth (feet)	Test Type	Undrained Shear Strength (psf)
TH-1O	US-2	73.5 – 75.5	Unconsolidated-Undrained Triaxial	530
TH-1O	US-4	77.5 – 79.5	Unconsolidated-Undrained Triaxial	640
TH-1P	US-1	70.5 – 72.5	Unconsolidated-Undrained Triaxial	1,350
TH-6G	US-2	55.0 – 57.0	Unconfined Compression	690
TH-6G	US-3	57.0 – 59.0	Direct Simple Shear	630
TH-6G	US-4	60.0 – 61.5	Unconfined Compression	1,300

As shown, the above laboratory measurements indicated that the west side of the Phase I area, with an overburden load of approximately 70 feet and a phosphatic clay thickness of approximately 8 feet, had undrained shear strengths of 530 to 640 psf, characteristic of a medium stiff clay. The south side of the Phase I area, with an overburden load of approximately 70 feet and a phosphatic clay thickness of approximately 4 feet, had a substantially higher undrained shear strength of 1,350 psf, characteristic of a stiff clay. At the location of TH-6G, with an overburden load of approximately 50 feet and a phosphatic clay thickness of approximately 10 feet, the undrained shear strengths were documented to be 630 and 690 psf for the two samples near the middle and 1,300 psf for the sample near the bottom of the phosphatic clay layer. The moisture contents of the undisturbed samples recovered from TH-1O were generally higher than those recovered from TH-1P, which confirmed that the undrained shear strengths of the phosphatic clay were higher at TH-1P than at TH-1O.

The undrained shear strengths of the phosphatic clay could also be inferred from the tip resistance obtained during piezocone soundings. At the location of PC-1O, the tip resistance suggested that the undrained shear strength of the phosphatic clay was on the order of 700 to 800 psf. At the location of PC-1P, the tip resistance indicated that the phosphatic clay had an undrained shear strength on the order of 1,100 to 1,200 psf. Although the piezocone sounding results yielded a slightly higher undrained shear strength compared to laboratory measurements at the locations of PC-1O/TH-1O and a slightly lower undrained shear strength compared to the laboratory measurements at the location of PC-1P/TH-1P, both sets of data indicated that the phosphatic clay near PC-1P/TH-1P had higher undrained shear strengths than the phosphatic clay near PC-1O/TH-1O.

For comparisons, an undisturbed sample obtained at a depth of 42 feet below the landfill surface from the Phase I area in 1994 yielded an undrained shear strength of 460 psf. Previous undisturbed samples obtained from the Phase VI area had measured undrained shear strengths of approximately 120 psf in 1994 (after the surcharge program had begun and prior to placement of any refuse) and 240 psf in 2001 (after the first lift of refuse had been placed). In its original state, the phosphatic clay below the desiccated crust had an undrained shear strength on the order of 70 psf.

As part of the annual monitoring program at the Southeast Landfill, piezoprobes were installed within the underlying phosphatic clay deposit to estimate the excess pore water pressure generated from landfill loadings. The 2007 monitoring program indicated that the excess pore water pressures were less than 2 feet at PC-1O and greater than 6 feet at PC-1P. Although these excess pore water pressures were relatively small in comparison to the 20 to 25-foot excess pore water pressure that can be induced by a 20-foot thick lift of refuse/ash, they seemed to contradict the field readings and laboratory data, which indicated that the phosphatic clay was thinner and stiffer at PC-1P than at PC-1O. With the phosphatic clay being thinner at PC-1P, a lower excess pore water pressure was expected because of the shorter drainage distance for pore water pressure dissipation. Laboratory data from the undisturbed samples indicated that the phosphatic clay at the location of TH-1P has achieved the anticipated strength. With full dissipation of excess pore water pressures at the existing landfill heights, Ardaman estimated that the undrained shear strengths of the phosphatic clay would be on the order of 1,200 psf in the Phase I area and 800 psf in the Phase V/VI areas. Because there are more inherent uncertainties and factors (e.g., instrumentation sensitivity, soil permeability, piezometric heads above and below the phosphatic clay, seepage gradient) associated with estimations of excess pore water pressures using piezoprobes to infer the *in situ* undrained shear strength of the phosphatic clay, laboratory testing on undisturbed samples would provide better indications of strength properties. As indicated previously, no piezoprobe was installed and thus no excess pore water pressure data were available in the Phase V/VI areas. Although excess pore water pressure is theoretically related to the magnitude of strength gain, the undrained shear strength is the parameter of concern for slope stability analyses. Excess pore water pressure is not an input parameter for undrained stability analyses.

Stability Analyses for Phase I and V/VI Areas

As represented by PC-1P/TH-1P, the phosphatic clay deposit along the south side of the Phase I area was considered to have a thickness of 4 feet, a total unit weight of 90 pcf, and an average undrained shear strength of 730 psf beneath the landfill slope. The average undrained shear strength was computed based on a maximum undrained shear strengths of 1,100 psf below the landfill crest and a minimum undrained shear strength of 365 psf near the toe of the landfill slope. An inferred undrained shear strength of 1,100 psf below the crest at +180 feet (NGVD) was calculated based on the measured strengths documented at TH-1P/PC-1P, which are located in an area with the landfill surface elevation slightly above +180 feet (NGVD). An undrained shear strength of 365 psf near the toe of the landfill was computed based on 20 feet of refuse/ash on top of 2 feet of drainage sand and 2 feet of phosphatic clay (i.e., the middle of the 4-foot thick phosphatic clay deposit). The drainage sand layer on top of the phosphatic clay was considered to have a thickness of 2 feet, a total unit weight of 120 pcf, and an internal friction angle of 32 degrees.

As represented by TH-6G, the phosphatic clay deposit along the outside perimeter of the Phase V/VI areas was considered to have a thickness of 10 feet, a total unit weight of 90 pcf, and an average undrained shear strength of 250 psf beneath the landfill slope. The average undrained shear strength was computed based on a maximum undrained shear strengths of 300 psf below the landfill crest and a minimum undrained shear strength of 200 psf near the toe of the landfill slope. An inferred undrained shear strength of 300 psf below the crest at +140 feet (NGVD) was calculated based on the measured strengths at TH-6G (i.e., 630 to 690 psf in the upper portion of the clay layer), which is located in the interior part of Phase V/VI area with a landfill surface elevation at +167 feet (NGVD) and a refuse/ash height of approximately 43 feet. An undrained shear strength of 200 psf near the toe of the landfill was estimated based on previous laboratory measurement by Ardaman in 2001 after the surcharge has been placed in the Phase V/VI areas. The drainage sand layer on top of the phosphatic clay was considered to have a thickness of 9 feet, a total unit weight of 120 pcf, and an internal friction angle of 32 degrees. The drainage sandy layer was thicker in the Phase V/VI areas because fill was placed in these two areas to surcharge the thick phosphatic clay prior to placement of any refuse/ash lift.

The refuse/ash mixture in the landfill was considered to have a total unit weight of 74 pcf. The landfill material was assigned an internal friction angle of 28 degrees with no cohesion. The leachate level was considered to be at the top surface of the sand layer, i.e. at +117 feet (NGVD) in the Phase I area and at +124 feet (NGVD) in the Phase V/VI areas.

Stability of landfill design sections after the next lift of refuse/ash has been placed was analyzed by Ardaman using the computer program SLOPE/W, which is a fully integrated slope stability analysis program developed by Geo-Slope International Ltd. of Calgary, Alberta, Canada. The analyses considered the circular arc failure mode through the refuse as well as the sliding block failure mode through the phosphatic clay foundation. For each failure mode, SLOPE/W performed the search routine and converged on the critical failure surface through an iterative procedure. The stability analyses were based on Spencer's Method.

After placement of the next lift of refuse/ash, the crest elevations will reach +200 feet (NGVD) in the Phase I area, and +160 feet (NGVD) in the Phase V/VI areas. The landfill will continue to be raised on a side slope of 4H:1V. As indicated previously, the crest of the perimeter dike was considered at +129 feet (NGVD). The landfill cross sections in the Phases I and V/VI areas are depicted in Figures 1 and 2, respectively, along with the selected material properties used in the stability analyses.

The factors of safety against the circular arc failure mode through the refuse were found to be higher than the factors of safety against the sliding block failure mode through the phosphatic clay foundation. Results of the sliding block analyses performed by Ardaman and presented in Figures 1 and 2 indicated that the minimum factors of safety against the sliding block failure mode in the Phase I and V/VI areas were 1.5 and 1.6, respectively. A factor of safety of 1.5 is generally considered adequate for these types of analyses.

Based on the collected data and our stability analyses, it is our professional opinion that the south side of Phase I area and the Phase V/VI areas are ready to receive the next lift of refuse/ash. However, Ardaman recommends additional studies be conducted prior to placement of the next lift of refuse/ash along the west side of the Phase I area where the phosphatic clay is relatively thick and the undrained shear strengths, as revealed by both the field piezocone readings and laboratory test data, were not as high as expected. The additional studies will be used to document the continued increase in undrained shear strength of the phosphatic clay.

Ardaman appreciates the opportunity to provide services to Jones Edmunds and Hillsborough County. If you have any questions or need additional information, please contact us.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.

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Assistant Project Engineer

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Senior Project Engineer 05/16/07
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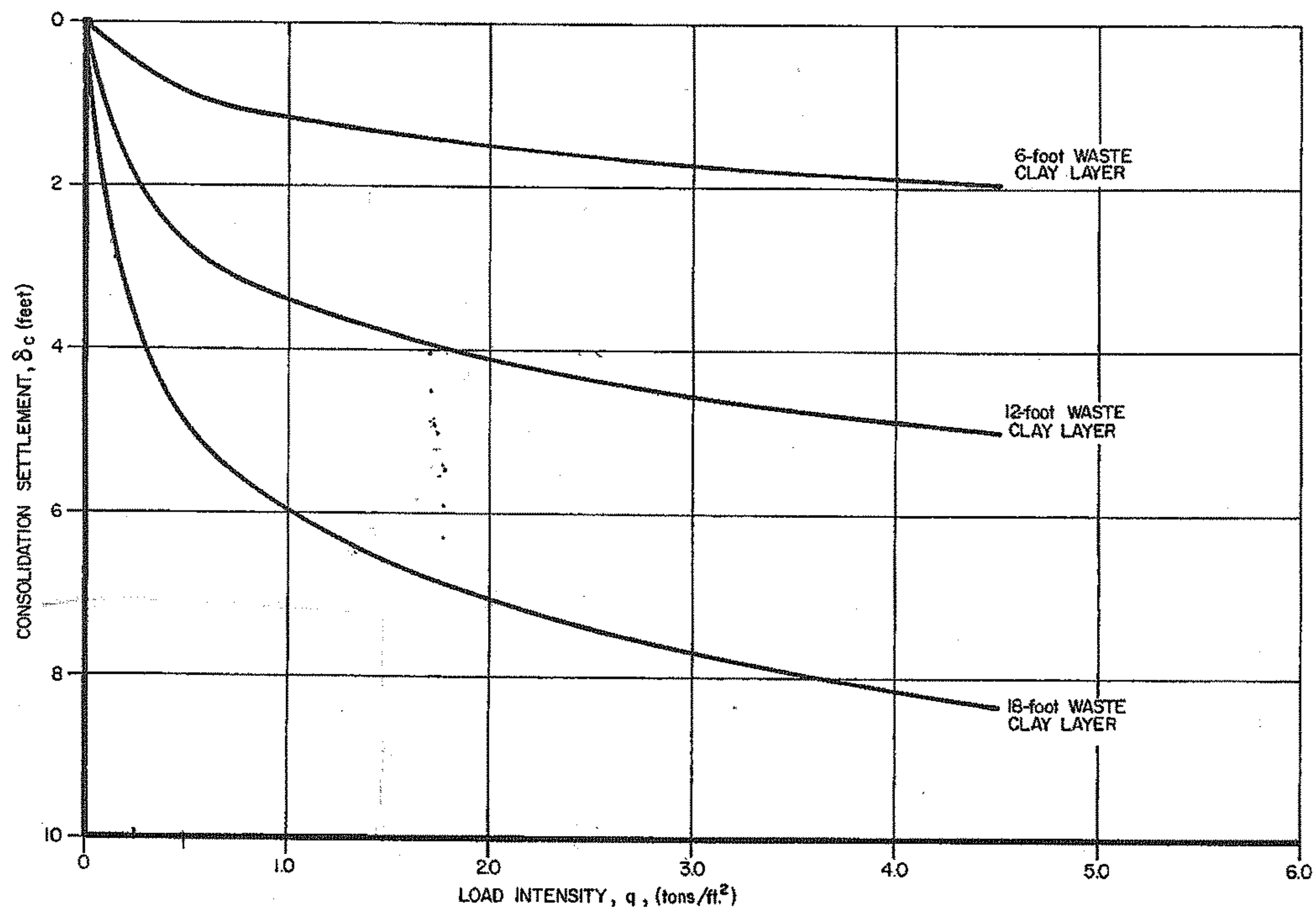
Enclosures

Appendix I

Consolidation Settlement Curves from Ardaman 1983 Study



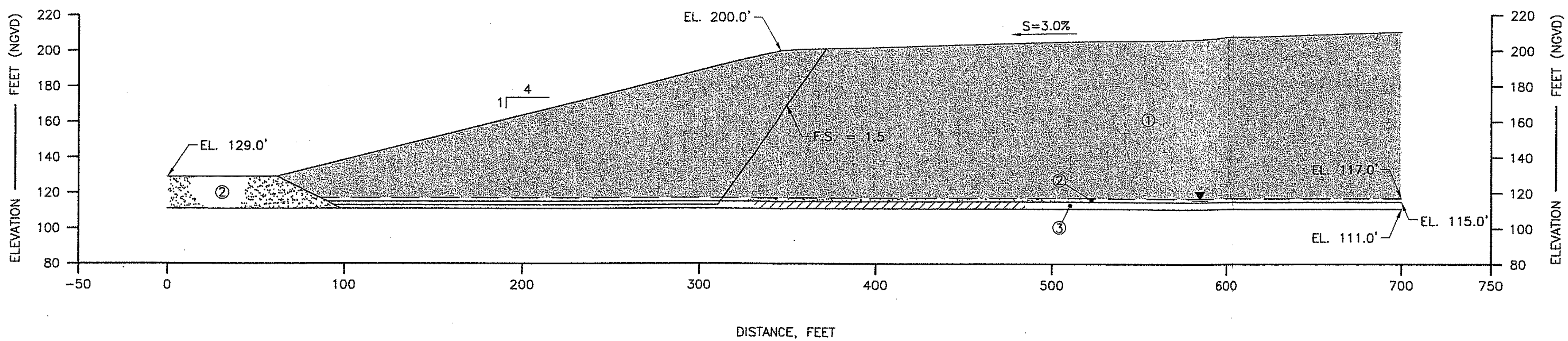
VARIATION OF CONSOLIDATION SETTLEMENT WITH LOAD INTENSITY FOR WASTE PHOSPHATIC CLAY LAYERS OF VARIOUS THICKNESS

Ardaman & Associates, Inc.
Consulting Engineers in Soil Mechanics, Foundations, and Material Testing

HYDROGEOLOGICAL INVESTIGATION
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: T.S.	CHECKED BY: D.J.C.	DATE: 1/20/83
FILE NO. 81-159	APPROVED BY: H.G. Standland	

FIGURE 6.11



LAYER	SOIL DESCRIPTION	TOTAL UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED SHEAR STRENGTH (PSF)
①	REFUSE / ASH	74	28	–
②	SAND	120	32	–
③	PHOSPHATIC CLAY	90	–	730

STABILITY ANALYSES FOR PHASE I AREA

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

STABILITY ANALYSES
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AAD	CHECKED BY: RN	DATE: 04/25/07
FILE NO. 06-212	APPROVED BY: FKC	FIGURE: 1

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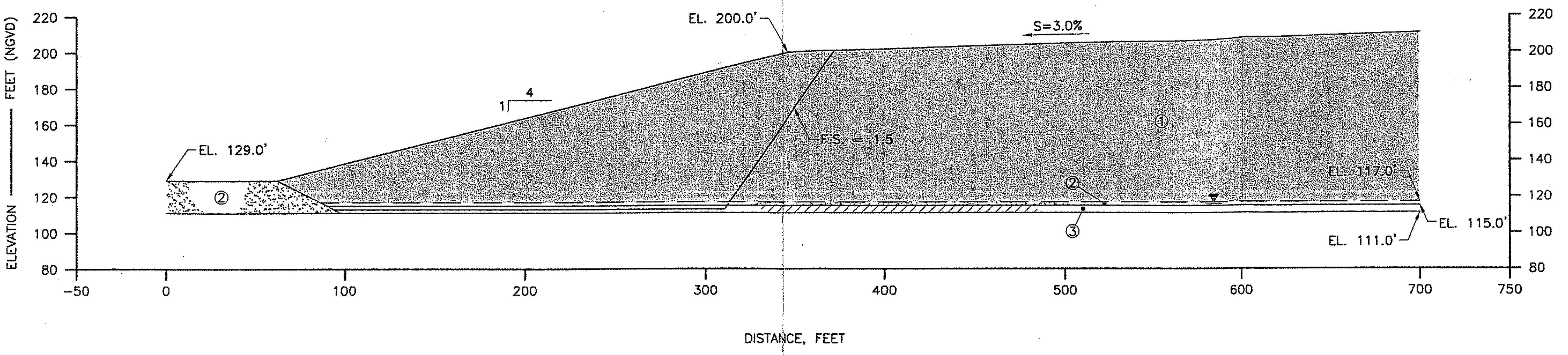

LAYER	SOIL DESCRIPTION	TOTAL UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED SHEAR STRENGTH (PSF)
①	REFUSE / ASH	74	28	–
②	SAND	120	32	–
③	PHOSPHATIC CLAY	90	–	730

STABILITY ANALYSES FOR PHASE I AREA

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

STABILITY ANALYES
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AAD	CHECKED BY: RN	DATE: 04/25/07
FILE NO. 06-212	APPROVED BY: FKC	FIGURE: 1

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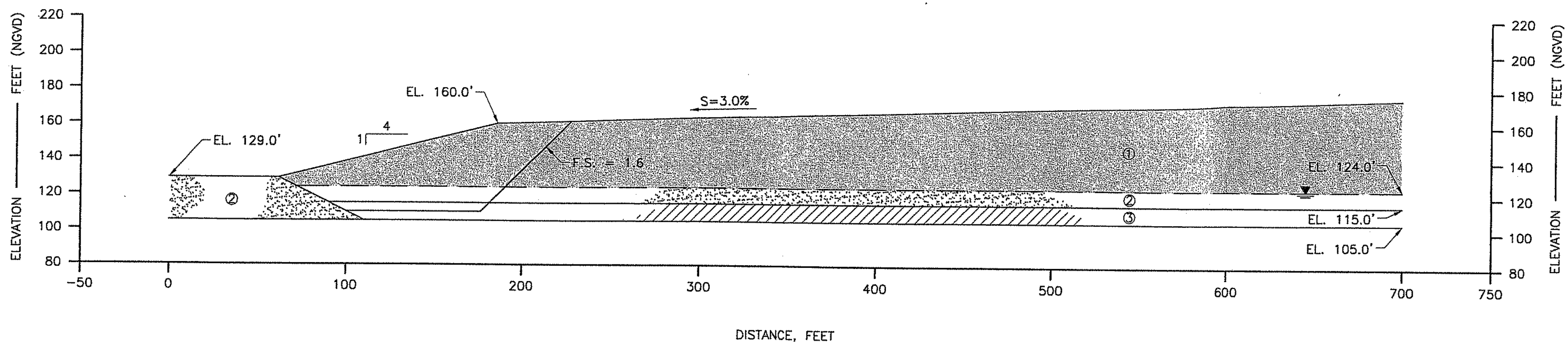

LAYER	SOIL DESCRIPTION	TOTAL UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED SHEAR STRENGTH (PSF)
①	REFUSE / ASH	74	28	–
②	SAND	120	32	–
③	PHOSPHATIC CLAY	90	–	250

STABILITY ANALYSES FOR PHASE V/VI AREAS

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

STABILITY ANALYES
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AAD	CHECKED BY: RN	DATE: 04/25/07
FILE NO. 06-212	APPROVED BY: FKC	FIGURE: 2

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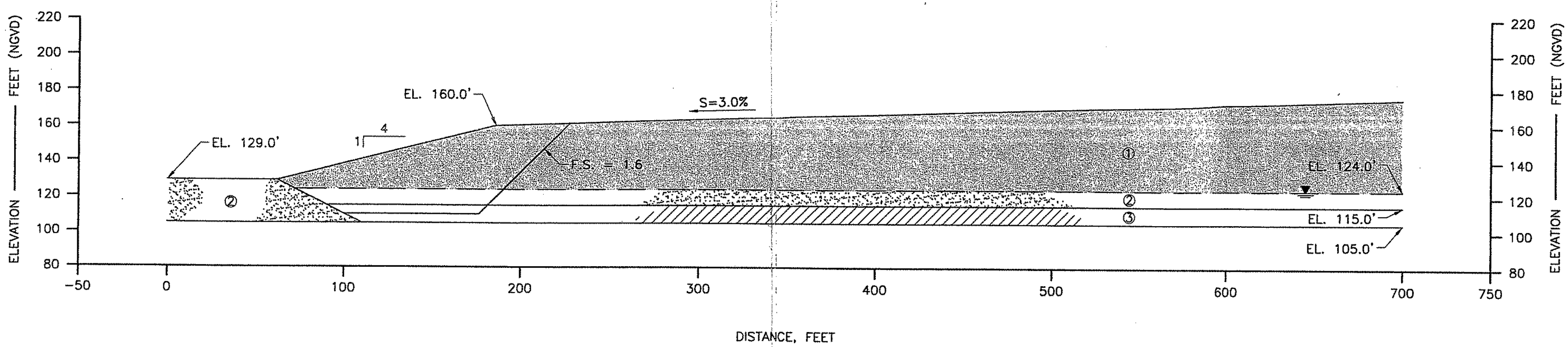

LAYER	SOIL DESCRIPTION	TOTAL UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED SHEAR STRENGTH (PSF)
①	REFUSE / ASH	74	28	–
②	SAND	120	32	–
③	PHOSPHATIC CLAY	90	–	250

STABILITY ANALYSES FOR PHASE V/VI AREAS

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

STABILITY ANALYES
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: AAD	CHECKED BY: RN	DATE: 04/25/07
FILE NO. 06-212	APPROVED BY: FKC	FIGURE: 2

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