FINAL Geotechnical Engineering Report ...

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RAHWAY ARCH PROPERTY

Carteret, New Jersey

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1.0 INTRODUCTION

In accordance with our proposal, Baker has completed the Subsurface Exploration and Geotechnical Evaluation for the proposed Rahway Arch project, located in Carteret, New Jersey (Figure A-1).

The purpose of this study was to determine general subsurface conditions at the project site and to evaluate those conditions with respect to geotechnical engineering considerations for the proposed construction. The specific scope of our services on this project consisted of the following.

- Drilling and Fieldwork, consisting of 29 auger borings and twelve (12) cone penetrometer test with pore pressure measurements (CPTu) borings.
- Laboratory testing consisting of water content, Atterberg limits, and grain-size distribution, USCS textural classification, specific gravity, pH, organic content, permeability, consolidation analyses, unconsolidated-undrained triaxial tests, and consolidated-undrained (CIU) triaxial tests.
- A review and description of the field and laboratory test procedures conducted and their results;
- A review of area and site geologic conditions, including geological hazards at the site, such as soft soils, swelling soils, sensitive soils, liquefaction, etc.;
- A review of subsurface conditions encountered with available physical properties;
- Potential excavation difficulties;
- Results of slope stability and settlement analyses;
- Recommendations for constructing the embankment/cap;
- Recommendations for a geotechnical monitoring program;
- Recommendations for shallow foundations (Net allowable bearing pressure and applied safety factor, recommended bearing depth, resistance to sliding, resistance to uplift, estimated settlement and modulus of subgrade reaction);
- Recommendations for deep foundations, if necessary;
- Subsurface drainage and potential difficulties with groundwater;
- Seismic site classification and recommendations;
- Site preparation, subgrade preparation, and construction and testing compacted fills;
- Other geotechnical concerns that may affect the planned construction; and
- A review and comment on the final remedial plan design for consistency with the geotechnical recommendations.

2.0 SITE AND PROJECT DESCRIPTION

In this section, details of the areas explored are described based on information available at the time of this report. A conceptual design site plan, indicating the location of the buildings, site boundaries, and associated parking; a topographic survey drawing; and past geotechnical and environmental reports were provided for this report by Soil Safe, Inc. (Soil Safe).

2.1. SITE DESCRIPTION

The Rahway Arch site is located on the banks of the Rahway River in the Borough of Carteret and was used from the 1930s through the 1970s for disposal of a mixture of alum sludge and yellow prussiate of soda (YPS) sludge from the American Cyanamide Warner Plant in Linden, New Jersey. The overall 124.7 acre site contains six impoundments, encompassing approximately 85 acres, located on the Rahway River. The impoundments were constructed above existing grade with wooden and earthen dikes. They contain approximately 2,000,000 tons of the cyanide containing alum-YPS sludge. The thickness of the sludge ranges from 5 to 20 feet.

The majority of the site is lightly vegetated with cattails and similar vegetation, with a relatively flat topography. The site is bounded to the northwest, north, northeast, and east by the Rahway River. To the west is marshland and to the south and southeast are tank farms. Site elevations vary between +13 ft above mean sea level (amsl) on the north portion of the property to sea level along the wetlands bordering the site.

2.2. PROJECT DESCRIPTION

The owner has entered into an agreement with Soil Safe to be the reclamation contractor and construct a cap over the impoundments. Soil Safe will be constructing a Class B soil recycling facility on Impoundment 2 to manufacture the engineered fill required for the cap. Impoundment 2 will be capped at the end of the project; after all other impoundments have been capped. The thickness of the soil cap will be determined, in part, from the study contemplated herein. In addition, the Soil Safe process will require the use of large temporary soil stockpiles for both pre-process and post-process soil material. For the purpose of this investigation, Baker will assume soil placement thicknesses in the range of five (5) to thirty (30) feet for the cap and possibly an additional twenty-five (25) feet for the soil stockpiles. Based upon historic data, the subsurface profile consists of surface fill, alum-YPS sludge, peat, organic silts and clays, and alluvial sand and glacial deposits (clay, silt, sand, and gravel) overlying red-brown siltstone and shale.

In order to construct this cap, the shear strength and compressibility of the underlying materials must be determined. This includes the time-dependent stress behavior of the underlying materials in order to formulate a time sequenced soil placement plan to safely construct the soil cap. Specific objectives include:

- characterizing the subsurface conditions in the impoundments, berms and the adjacent wetlands outside the berms;
- determining the stability of the berms for the existing conditions and placement of an engineered fill cap over the impoundments;
- evaluating the geotechnical conditions in the proposed areas for the Class B facility, the scale and the steel bridge that allows access of the site;
- evaluating stability of the impoundments to support the engineered fill cap;
- performing slope stability analyses to ensure that adequate factors of safety can be maintained in the final cap design;
- estimating the settlement that will occur in the impoundments;
- determination of the effective vertical overburden pressure by area and related time rate of consolidation allowable to safely place the soil cap and temporary stockpiles;
- develop construction options for handling excessive settlement and slope stability issues; and
- develop a geotechnical monitoring program to be implemented during construction (along with contingency options for excessive settlement and slope stability issues).

3.0 GEOLOGIC SETTING

3.1. SITE GEOLOGY

Based upon the Surficial Geology Map of the Perth Amboy and Arthur Kill Quadrangles (Stanford, 1999) and the Geologic Map of New Jersey (Drake, etal, 1996), the proposed development will be sited primarily over a relatively thin layer of surficial unconsolidated deposits and bedrock below that. Surficial unconsolidated materials in the Perth Amboy and Arthur Kill quadrangles consist of glacial, stream, wetland, and weathered bedrock sediment. The glacial sediment is not found at the Rahway site. The stream sediment, as much as 40 feet thick, includes sand, gravel, and silt deposited in floodplains, stream terraces, and former river plains. The wetland sediment includes peat and organic silt and clay deposited in freshwater swamps and saltwater marshes and estuaries. It is as much as 100 feet thick. The weathered bedrock consists of silty clay and shale fragments formed by chemical and mechanical decomposition of shale bedrock of Triassic and Jurassic Age. It is generally less than 10 feet thick. In the area of the Rahway Arch property, the primary surficial unconsolidated deposits are pre-glacial weathered bedrock and postglacial artificial fill and estuarine and salt-marsh deposits. The underlying bedrock is comprised of Passaic and Lockatong Formation sedimentary rock. Specific details of the units found at the site are included below:

Artificial Fill overlying Estuarine and Salt-Marsh Deposits (af/Qm). Artificial fill is made up of excavated sand, silt, clay, gravel, rock, and till, and man-made materials (bricks, cinders, ash, slag, glass, construction materials and minor amounts of trash). Color is variable, but generally gray to black. The unit is as much as 50 feet thick, but is generally less than 20 feet thick. At the Rahway site, the artificial fill is primarily YPS-alum sludge.

Estuarine and Salt-Marsh Deposits (Qm). Brown to dark gray, peat and organic clay and silt, with minor sand and shells. Locally, at the base of this unit, alluvial sand and gravel, deposited before marine inundation, may be present. The thickness of the unit may be as thick as 100 feet.

Weathered Shale (Qsw). Poorly-sorted, nonstratified to weakly stratified, reddish-brown to yellowish-red silty clay to clayey silt with some to many angular to subangular chips of red (and minor gray) shale. Derived from mechanical and chemical decomposition of shale of the Passaic Formation of Triassic and Jurassic Age. The unit is generally less than 10 feet thick.

Passaic Formation (JTrp). This Lower Jurassic and Upper Triassic Age formation, previously known as the Brunswick Formation, is comprised of predominantly red beds consisting of argillaceous siltstone; silty mudstone; argillaceous, very fine grained sandstone; and shale; mostly red-brown to brown-purple and gray-red. The red beds occur typically in 10 to 23 feet thick, cyclic playa-lake-mudflat sequences and fining-upward fluvial sequences. Lamination is commonly indistinct due to borrowing, dessication, and paleosol formation. Where layering is preserved, most bedforms are wavy parallel lamination and trough and climbing-ripple cross lamination. Calcite- or dolomite-filled vugs and flattened cavities, mostly 0.02 to 0.08 inch across, occur mostly in the lower half. Sand-filled burrows, 0.08 to 0.2 inch in diameter, are prevalent in the upper two-thirds of the unit. Dessication cracks, intraformational breccias, and curled silt laminae are abundant in the lower half. Lake cycles, mostly 7 to 16 feet thick, have a basal, greenish-gray, argillaceous siltstone; a medial, dark-gray to black, pyritic, carbonaceous, fossiliferous, and, in places, calcerous lake-bottom fissile mudstone or siltstone; and an upper thick-bedded, gray to reddish and purplish-gray argillaceous siltstone with dessication cracks, intraformational breccias, burrows, and mineralized vugs. The thickness of the formation is about 11,500 feet.

Lockatong Formation (Trl). This Upper Triassic formation is comprised of predominantly cyclic lacustrine sequences of silty, dolomitic or analcime-bearing argillite; laminated mudstone; silty to calcareous, argillaceous very fine-grained sandstone and pyritic siltstone; and minor silty limestone, mostly light- to dark-gray, greenish gray, and black. Grayish-red, grayish-purple, and dark brown-red sequences occur in some places, especially in the upper half. Two types of cycles are recognized: freshwater-lake (detrital) and alkaline-lake (chemical) cycles. Freshwater-lake cycles average 17 feet thick and consist of basal, transgressive, fluvial to lake-margin deposits that are argillaceous, very fine-grained sandstone to coarse siltstone with indistinct lamination, planar or cross lamination, or are disrupted by convolute bedding, dessication cracks, root casts, soil-ped casts, and tubes. Medial lake-bottom deposits are laminated siltstones, silty mudstones, or silty limestones that are dark gray to black with calcite laminae and grains and lenses, or streaks of pyrite; fossils are common, including fish scales and articulated fish, conchostracens, plants, spores, and pollen. Upper regressive lake margin, playa lake, and mudflat deposits are light- to dark-gray silty mudstone to argillitic siltstone or very fine-grained sandstone, mostly thick-bedded to massive, with dessication cracks, intraformational breccias, faint wavy laminations, burrows, euhedral pyrite grains, and dolomite or calcite specks. Alkaline-lake cycles are similar to freshwater-lake cycles, but are thinner, averaging 10 feet, have fewer fossils (mainly conchostracens), and commonly have red beds, extensive dessication features, and abundant analcime and dolomite specks in the upper parts of cycles. The thickness of the formation near Byram is about 3,500 feet. The formation thins to the southeast and northeast, with the thickness less than 2,300 feet near Princeton.

The Passaic Formation underlies the majority of the site, whereas the Lockatong Formation underlies Impound 1 and possibly portions of Impound 2. There do not appear to be any major faults close to the site, although any structural features of the basement rock underlying the site are hidden by the overlying unconsolidated deposits.

3.2. SITE HYDROGEOLOGY

The hydrogeology of the site is dominated primarily by the Rahway River and sea level tidal fluctuations, with shallow groundwater flow generally toward the river. Deeper groundwater within the underlying Passaic Formation bedrock flows seaward. The Passaic Formation is a major source of groundwater to the west of the site, with flow occurring primarily in fractured shale. Separating these two aquifers is a continuous layer of red-brown clay. The clay layer identified beneath the shallow unconsolidated material functions as a confining unit for the underlying Passaic Formation (Hydrosystems, 1989). As such, the clay layer will restrict the vertical flow of water between the shallow and bedrock aquifers. Within the impounds, a groundwater mound composed of freshwater roughly five to ten feet above the groundwater elevation of the surrounding areas has developed, creating a horizontal and vertical flow field within the impounds flowing radially outward to the adjacent surface waters of the Rahway River, Cross Creek, and Deep Creek. The natural groundwater in the area is generally brackish, therefore the impound groundwater is less dense and therefore floats above the brackish groundwater table (Hydrosystems, 1989).

Eight paired monitoring well clusters were installed at this site to monitor the shallow unconsolidated and bedrock aquifers. The shallow monitoring wells were screened from depths of 10 to 20 feet in the shallow fill material and tidal marsh deposits. The deep wells were screened in the upper weathered portion of the Passaic Formation at depths ranging from 40 to 60 feet below ground surface (bgs). The water table is encountered approximately 2 feet bgs in shallow monitoring wells. Water table mounding occurs in the shallow aquifer beneath the impoundments, where ground-water elevation was measured as approximately 10 feet above mean sea level (Hydrosystems, 1989).

The fine-grained Passaic Formation typically has low primary porosity. Where coarser-grained rock is present, it is tightly cemented and has a high clay mineral content. Ground water flow occurs primarily in bedding plane fractures or in secondary fractures (joint sets) formed by stresses related to faulting following the deposition and lithification of the beds (USGS, 1968). Regional flow in the Passaic Formation occurs vertically and laterally toward the northeast, with ultimate discharge to surface water bodies which, in the vicinity of the Carteret Impoundments, include the lower Rahway River, Arthur Kilt, and, eventually, the Atlantic Ocean (Blasland, etal., 1995). Disko (1982) completed permeability tests on subsurface samples. Using data presented in Disko (1982), Blasland, etal. (1995) estimated a mean coefficient of permeability (k) for the sludge fill and tidal marsh units. The mean k value derived from these data was 1.10 ft/day.

3.3. SITE SEISMIC HAZARDS

It is unknown whether a seismic hazard assessment has been performed for this site or for any sites nearby. Historical seismicity within the New York/New Jersey area indicates that over the past 300 years, there have been a number of significant seismological events. Earthquakes with a maximum modified Mercalli scale of VII (roughly between 5.5 and 6.0 on the Richter scale) occurred in the New York City area in 1737, 1783, and 1884 (Dombroski, 2005). A number of smaller events have also occurred in the area over the last 300 years. The time spans between events indicate a 100 year return period. Based upon review of a geology map of New Jersey, there are no known faults on or near the site. However, there are many faults in New Jersey including the Ramapo Fault, separating the Piedmont and Highlands Physiographic Provinces, to the northeast of the site.

Utilizing current data developed from earthquake measurements in the region, the peak horizontal ground acceleration with a 7% probability of exceedance in any 75-year period ranges between 88 gals and 98 gals or 0.09g to 0.10g (AASHTO, 2008). The spectral acceleration at 0.2 second period with a 7% probability of exceedance in any 75-year period ranges between 157 gals and 177 gals or 0.16g to 0.18g and the spectral acceleration at 1.0 second period with a 7% probability of exceedance in any 75-year period ranges between 29 gals and 39 gals or 0.03g to 0.04g. Peak acceleration is the acceleration experienced by a particle on the ground. Spectral acceleration is approximately what is experienced by a building, as modeled by a particle on a mass-less vertical rod having the same natural period of vibration as the building.

Most methods for determining seismic soil response are based upon the assumption that upward propagation of horizontally polarized shear waves from the underlying rock formation governs the response of the soil deposit. Two independent design response spectra are typically developed, one to define the horizontal component of ground motion, and the second to define the vertical component. The vertical component of ground motion usually contains much higher frequency content than the horizontal component; therefore the spectral shape is different than that of the horizontal component. The peak ground acceleration (PGA) associated with the vertical component will also be different than the PGA of the horizontal component. Both values of PGA are dependent on the distance from the source.

The type of soil affects the response to dynamic loading. The most significant factors include grain size distribution, clay fraction, and degree of saturation. For sensitive cohesive soils, such as those that exist at the site, liquefaction and seismic response may be important. Embankment slope materials that are vulnerable to earthquake loadings include very steep, weak, fractured, and brittle rocks or unsaturated loess; loose saturated sand; sensitive cohesive soils with natural moisture exceeding the liquid limit; and dry cohesionless material on slopes at the angle of repose.

4.0 HISTORICAL DATA

4.1. PREVIOUS INVESTIGATIONS

There have been a number of geotechnical and environmental-related studies conducted at the Rahway Arch property. M. Disko Associates (Disko, 1981a) conducted the earliest study on the impounded sludge in June 1981, drilling twelve (12) borings inside the impounds (one at the edge and one at the center of each impound) to the bottom of the sludge layer. Twelve (12) in-situ density tests on the sludge were made using the Sand Cone Method (ASTM D1556). The resulting sludge densities ranged from 36.4 pcf to 81.9 pcf with an average in-situ density of 54.7 pcf. The moisture content was not tested. Falling head permeability tests were also run on remolded samples of sludge, with permeability ranging from $6.57(10)^{-6}$ cm/sec to $1.19(10)^{-4}$ cm/sec, averaging $5.33(10)^{-5}$. Field conditions within the impounds were also noted. Pond #1 was the only pond that was covered by vegetation. The sludge also showed signs of stratification and coloring within all of the impounds, being most pronounced within Impounds 2, 3, and 4.

In September 1981, Disko (1981b) again conducted a sludge investigation, drilling twelve (12) borings and conducting laboratory permeability tests. The intent was to evaluate permeability closer to the base of the sludge layer. The borings were again drilled at the edge and center of each impound to depths ranging from 10 feet at the edge of Impound 1 to 29 feet at the center of Impound 6. Sludge depths ranged from 5 feet at the edge of Impound 2 to 20 feet at the edge of Impound 5. The borings were terminated in the organic silt layer underlying a layer of peat. Falling head permeability tests were run on remolded samples of sludge 1 to 2 feet above the bottom of the sludge layer and on remolded samples of the underlying silt collected 3 to 13 feet below the sludge/soil interface. Permeability values of the bottom of the sludge ranged from $5.90(10)^{-6}$ cm/sec to $8.00(10)^{-5}$ cm/sec, averaging $2.46(10)^{-5}$ cm/sec. The permeability values of the silt layer ranged from $2.20(10)^{-7}$ cm/sec to $6.34(10)^{-6}$ cm/sec, averaging $2.46(10)^{-6}$ cm/sec.

A third investigation was conducted by Disko (1982) in January 1982 to evaluate the condition and soil material of the earth berms surrounding the impounds and to evaluate what type of material they were constructed upon. The borings were drilled through the berms and underlying silt material and to the underlying rock formation (for three of the borings). Eleven (11) borings were drilled at the site, B-1 to B-10 and B-6A. Three (3) borings were drilled to the top of rock at depths ranging from 29 feet to 38.5 feet. Sludge was found below the earth berms in eight (8) of the eleven (11) borings. The berm was found to be lying directly on peat in three (3) borings. Falling head permeability tests were also run on remolded samples of sludge, berm material, peat, organic silt, sand and gravel, and shale fragments. The permeability of the berm material ranged from $3.00(10)^{-2}$ cm/sec to $5.00(10)^{-6}$ cm/sec. The permeability of the materials were similar to past test results.

In 1997, Blasland, Bouck, and Lee (BBL, 1997) conducted a geotechnical exploration at the site, performing SPT drilling, Shelby tube sampling, field vane shear testing, and laboratory analysis. Seven (7) soil borings (SB1 to SB4, SB6 to SB8) were drilled within the Impounds 2, 3, 4, and 5 to collect Shelby tube samples and to determine the depth of the sludge. A total of 56 vane shear tests were also performed within all of the sludge impoundments conducted using a Geonor H-60 at 13 locations and at depth of 1, 2, 5 and 9 feet below the ground surface. At locations further into the impounds, the tested undrained shear strength ranged from 115 psf to 940 psf with an average strength of 482 psf. At test locations near the berms, the undrained shear strength varied from 668 psf to 1337 psf with an average of 981 psf. The results from locations in Impound 6 have the lowest undrained shear strength. Remolded vane shear tests were also performed with results ranging from 0 to 574 psf, averaging 47.6 psf.

The “undisturbed” Shelby tube sludge samples were tested in a geotechnical laboratory. A total of 13 samples from Borings SB1 to SB4 and SB6 to SB8 were tested for index properties, moist and dry density, specific gravity, shear strength testing, and consolidation testing. The sludge samples were characterized as elastic silts and silts with moisture contents ranging from 69.3% to 128.9% with an average of 93.8%. The tested moist density ranged from 60.3 to 102.7 pcf with an average moisture density of 92.5 pcf. Specific gravity values ranged from 2.89 to 3.27 with an average value of 3.11. The initial void ratio ranged from 2.295 to 3.948 with an average of 3.073.

A total of seven (7) undisturbed sludge samples were laboratory tested for unconsolidated undrained (UU) shear strength in 1997 by BBL (1997). The applied confining pressure in the triaxial chamber for the UU testing is approximately equivalent to the total overburden pressure at the sample depth. The tested undrained shear strength ranged from 215 to 1085 psf depending on the sample location. Three (3) undisturbed sludge samples were tested using the direct shear test method under drained conditions in 1997 by BBL. The samples were tested under 1, 3 and 10 ksf vertical pressures. The tested peak drained shear strength was 35º internal friction angle and 300 psf cohesion. If the cohesion is neglected, the equivalent internal friction angle under each vertical loading varies from 36º to 48º. Four (4) consolidation tests were also performed on undisturbed samples of the on-site sludge. Compression and recompression indices ranged from 0.767 to 1.152, averaging 0.965 and 0.02 to 0.022, averaging 0.021, respectively. Estimated preconsolidation stresses ranged from 0.9 to 3.1 tons per square foot (tsf) with the highest stress reported from a sample collected within two (2) feet of ground surface, indicating that drying of the material over time has potentially created an overconsolidated material.

In 2006, Edwards and Kelcey (2006) conducted a geotechnical investigation for the Tremley Point

Connector Road, Interchange 12 Improvements Project, for the New Jersey Turnpike Authority (NJTA). The exploration program consisted of drilling and sampling 32 SPT soil borings, installing two (2) observation wells, and performing laboratory testing on select soil and rock samples. The investigation covered wetland areas and three alternate alignments adjacent to the Rahway Property (then owned by Cytec) as well as a portion of the Rahway Property within Impound 3. All borings were drilled to the top of bedrock, which was encountered between 20 and 50 feet below ground surface. Eight (8) unconfined compression tests and ten (10) consolidation tests were performed on undisturbed samples of organic clayey silt to peat with varying amounts of sand. Undrained shear strength values ranged from 60 psf to 290 psf, averaging 181 psf. Compression and recompression indices ranged from 0.062 to 5.127, averaging 1.285 and 0.006 to 0.959, averaging 0.159, respectively. Estimated preconsolidation stresses ranged from 0.2 to 4.7 tons per square foot (tsf).

5.0 *FIELD AND LABORATORY WORK*

5.1. FIELD EXPLORATION

The field exploration consisted of drilling thirty-one (31) standard penetration test (SPT) borings and ten (10) CPTu borings. Warren George, Inc. (Warren George) completed the borings using a swamp buggy-mounted drill rig and a truck-mounted drill rig from July 10th to August 3rd, 2012 using flush joint casing with mud rotary drilling (casing advancer system) methods to drill the borings. "Tiger" mud and powdered sodium-bentonite were circulated within the hole to remove cuttings. Rock coring was not conducted. All CPT borings were performed from the swamp buggy using a drill rig to advance the CPT rods and probe. Baker personnel logged the borings.

The boring locations were staked by Eaststar Environmental (Eaststar) personnel. Final surveyed locations and elevations at the as-drilled boring locations were provided by Kernan Consulting Engineers (Kernan). Current and historical boring locations are shown in Figure A-2, located in Appendix A. All SPT and CPT borings were drilled and sampled to the depths shown in Table 5-1.

Table 5-1. Boring Locations and Depths

Boring	Depth (ft)	Location	Boring	Depth (ft)	Location
BD-01R	41.5	Impound 4 Berm	IS-02	27.0	Impound 2
BD-02	32.0	Impound 3 Berm	W-01	41.0	Wetland adjacent to Impound 4
BD-03	47.0	Impound 4 Berm	W-02	26.5	Wetland adjacent to Impound 4
BD-04R	39.0	Impound 2 Berm	W-03	30.0	Wetland adjacent to Impound 3
BD-05	37.0	Impound 1 Berm	W-04	32.0	Wetland adjacent to Impound 1
BS-01	32.0	Impound 6 Berm	W-05	42.0	Wetland adjacent to Impound 5
BS-02	32.0	Impound 3 Berm	W-07R	27.0	Wetland adjacent to Impound 2
BS-03	26.0	Impound 4 Berm	W-08	32.0	Wetland adjacent to Impound 3
BS-04	27.0	Impound 4 Berm	CP-01	28.9	Impound 4
BS-05	27.0	Impound 1 Berm	CP-02	29.5	Impound 4
BS-06	24.0	Impound 1 Berm	CP-03	19.8	Wetland adjacent to Impound 2
BS-07R	27.0	Impound 2 Berm	CP-04	26.6	Impound 6
BS-08	25.0	Impound 3 Berm	CP-05	39.5	Impound 5
BS-09	26.0	Access Bridge	CP-06A	26.9	Impound 2
ID-01	49.0	Impound 4	CP-08	24.1	Impound 6

Table 5-1. Boring Locations and Depths					
Boring	**Depth (ft)**	**Location**	**Boring**	**Depth (ft)**	**Location**
ID-02	47.0	Impound 5	CP-08A	20.8	Impound 6
ID-03R	34.5	Impound 6	CP-09D	18.9	Impound 3
ID-04	36.0	Impound 3	CP-10	21.0	Wetland adjacent to Impound 6
ID-05	47.0	Impound 2	CP-11A	21.2	Impound 2
ID-06	37.0	Impound 1	CP-12	20.8	Impound 1
IS-01	27.0	Impound 2/5			

5.1.1. Standard Penetration Testing

The field exploration consisted of SPT soil samples obtained at a continuous 2-feet interval for the top 10-feet of drilling, and at 5-feet intervals below 10-feet, except for within the impounds, where continuous sampling was conducted through the entire depth of the sludge. In general, the SPT consisted of advancing a sampling spoon (2-inch outside diameter) 2-feet by driving it with a 140-pound hammer falling 30-inches. Typically, an 18 inch spoon is driven, however, a 24-inch sampling spoon was used for this project. The values reported on the boring logs are the blows required to advance four successive increments. The first 6-inch increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value. The fourth value is not used, but is recorded on the logs. The soils were classified in general accordance with the Unified Soil Classification System.

5.1.2. Cone Penetrometer Testing with Pore Pressure Measurements

The CPTu boring program took place on July 10th, 12th, 13th and 16th, 2012. A total of eighteen (18) soundings were completed at eleven (11) different sounding locations. The CPT program was performed to evaluate in situ geotechnical criteria relative to the soils. In addition to the CPT soundings, shear wave velocity tests were performed at eight (8) of the locations with testing at five-foot depth intervals and dissipation tests were performed at nine (9) locations at various depths. The cone penetrometer tests were carried out using an integrated electronic piezocone. The piezocone used was a compression model cone penetrometer with a 15 cm^2 tip and a 225 cm^2 friction sleeve. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80. The piezocone dimensions and the operating procedure were in accordance with ASTM standard D-5778-07.

Pore pressure filter elements, made of porous plastic, were saturated under a vacuum using silicone fluid as the saturating medium. The pore pressure element was six millimeters thick and was located immediately behind the tip (the u_2 location) for all soundings. The cone was advanced using a skid drill rig, mounted on a Kori rig operated by Warren George. The following data were recorded onto magnetic media every five centimeters (approximately every two inches) as the cone was advanced into the ground: Tip Resistance (qc), Sleeve Friction (fs), and Dynamic Pore Pressure (u).

During seismic testing, the seismic signals were recorded using a geophone mounted in the cone and an up-hole digital oscilloscope. A sledge hammer, struck against a steel wedge was used as the seismic source. While stopped, pore water pressures were automatically recorded at five-second intervals and the readings stored in a dissipation file for estimation of C_h, the coefficient of consolidation that can in turn be used to calculate K_h, the horizontal hydraulic conductivity.

5.2. LABORATORY ANALYSES

The laboratory testing consisted of performing classification and index testing; including natural moisture content, grain-size distribution, Atterberg limits, and hydrometer analysis; specific gravity; pH; organic content; consolidation analyses; unconsolidated-undrained triaxial tests; and consolidated-undrained (CIU) triaxial tests as shown in Table 5-2 below.

Table 5-2. Laboratory Testing

Laboratory Analysis	ASTM Standard	Purpose
Natural Moisture Content	D2216	Determine soil moisture content
Atterberg Limits	D4318	Determine soil plasticity
Sieve Analysis	D422	Determine soil grain size distribution
Hydrometer Analysis	D422	Determine clay and silt fraction
Specific Gravity	D854	Determine specific gravity
pH	D4972	Determine acidity
Organic Content	D2974	Determine organic content
Permeability	D5084	Determine soil permeability
Consolidation	D2435	Determine soil compressibility
unconsolidated-undrained (UU) triaxial	D2850	Determine undrained shear strength
consolidated-undrained (CIU) triaxial	D4767	Determine drained shear strength

5.2.1. Classification Testing

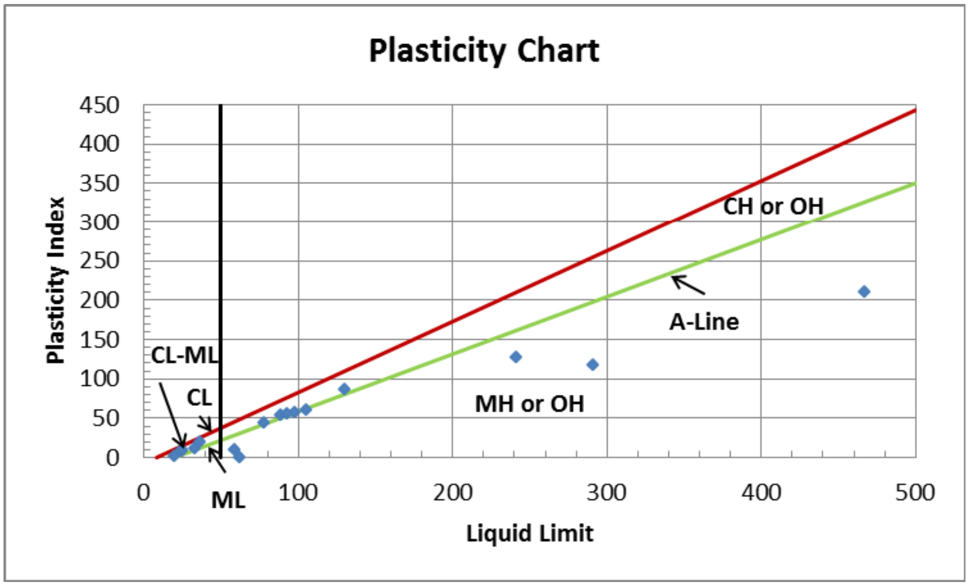
Results of classification testing are summarized in Table 5-3. Natural Moisture Content results are shown on Test Boring Logs in Appendix B and grain-size distribution graphs, specific gravity, pH, organic content, consolidation, UU triaxial, CIU triaxial test results in Appendix C.

Table 5-3. Laboratory Classification Results

Sample	Depth (ft)	Description	LL%	PL%	NMC	% Clay/Fines	USCS
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (Peat)	291	173	72.9	16.5/79.1	MH
BD-03/T-1	23.0-25.0	Sandy Elastic SILT (Peat)	105	45	65.1	18.3/66.5	MH
BD-03/S-12	25.0-27.0	Sandy Fat CLAY (Peat)	89	35	73.8	13.8/56.6	CH
BD-04R/T-1	21.0-23.0	Fat CLAY (Peat)	98	41	75.0	17.9/93.2	CH
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (Peat)	78	34	70.9	16.2/82.3	CH
BS-01/S-9	16.0-18.0	Sandy Lean CLAY	25	17	20.4	17.4/64.6	CL
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (Peat)	467	256	375.7	21.5/62.0	MH
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	-	-	87.3-124.3	18.0/93.0	MH
BS-04/T-1	23.0-25.0	Fat CLAY (Peat)	93	38	71.9	21.2/87.3	SM
BS-04/S-12	25.0-27.0	Silty SAND	-	-	45.0	-/32.0	SM
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	-	-	73.6-84.9	21.0/96.0	MH
BS-08/T-1	15.0-17.0	SILT with Sand	20	18	20.1	7.0/71.1	ML
BS-08/S-10	21.0-23.0	SILT with Sand	-	-	16.0	-/80.0	ML
BS-09/T-1	12.0-14.0	Silty SAND (Peat)	241	113	58.6	7.7/43.1	SM
ID-01/S-3	4.0-6.0	Elastic SILT (Sludge)	59	49	134.3	5.0/96.6	MH
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	-	-	114.5-170.1	26.0/86.7	MH
ID-06/S-2	2.0-4.0	Elastic SILT (Sludge)	62	61	109.2	-/-	MH
ID-06/U-2	14.0-16.0	Fat CLAY (Peat)	130	43	120.9	-/-	CH
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (Peat)	33	22	99.3	-/-	CL
W-05/T-1	15.0-17.0	Sandy Lean CLAY (Peat)	25	15	26.1	-/-	CL
W-08/U-1	14.0-16.0	Lean CLAY (Peat)	36	16	30.4	-/-	CL
COMPOSITE	-	Elastic SILT	NP	64	94.3-115.8	-/-	MH

USCS: Unified Soil Classification System **PL:** Plastic Limit **LL:** Liquid Limit **NMC:** Natural Moisture Content

In general, the liquid and plastic limits of the on-site organic peat material are very high, which is characteristic of these materials. The sludge material generally has lower liquid and plastic limits. There are some peat soils that have lower limits, which may indicate some mixing of the peat and overlying sludge material. The accompanying figure below presents classification results on a plasticity chart.



5.2.2. Permeability Testing

One sample was tested to determine the coefficient of permeability of the sludge using the falling head constant volume method within a triaxial cell. The test was performed on a composite sludge sample taken from several Shelby tubes to determine the flow characteristics of the sludge in a remolded condition. Numerous attempts were made to collect Shelby tube samples within the sludge, however many were unsuccessful due to the low shear strength of the sludge. Many of the Shelby tube samples collected tended to be within the upper portions of the sludge, which tended to be drier and stronger and not necessarily representative of the entire sludge profile. The sample was remolded to a specified density and molding water contents (similar to the undisturbed samples collected), consolidated to 700 psf and then tested to estimate permeability. The pre-test moisture content and dry density were 71.89% and 103.0 pcf, respectively. Table 5-4 presents the results of the permeability testing.

Table 5-4. Permeability Testing Result

Sample	Depth (ft)	Description	Consolidation Pressure (psf)	Permeability (cm/sec)
COMP	-	Elastic SILT (Sludge)	700	$6.43(10)^{-6}$

Note: composite compacted from BS-03/T-1, BS-06/U-1, and ID-03R/T-1

The remolded permeability of the sludge is in the lower range of permeability values measured by previous investigators. The data sheets are presented within Appendix C.

5.2.3. Shear Strength Testing

Shear strength testing was conducted on a total of eleven (11) undisturbed Shelby tube samples and two (2) composited, remolded samples; nine (9) unconsolidated-undrained (UU) compression tests and four (4) consolidated isotropically undrained compression (CIUC) tests. UU testing was conducted on undisturbed soils from borings BD-01R/T-1 (24.0’-26.0’), BD-04R/T-1 (21.0’-23.0’), BS-02/T-1 (7.0’-9.0’), BS-04/T-1 (23.0’-25.0’), BS-08/T-1 (15.0’-17.0’), IS-02/U-1 (19.0-21.0’), W-05/T-1 (15.0’-17.0’), and W-08/U-1 (14.0-16.0’. UU testing was also conducted on a composited, remolded sample of sludge. The CIUC tests were conducted on undisturbed soil from borings BD-03/T-1 (23.0’-25.0’), BS-09/T-1 (12.0’-14.0’), and BS-03R/T-1 (8.0’-10.0’) and a composited, remolded sample of sludge. Appendix E contains photographs of these samples before, during, and after testing.

UU compression tests are performed within a triaxial cell with the drainage lines closed. A minimal cell pressure is applied followed by axial loading, therefore, only a small amount of consolidation is allowed to occur. Measured soil parameters depend heavily upon degree of saturation. The confining pressure is atmospheric (or zero gauge pressure). CIUC compression tests involve the initial consolidation of the sample under some designated confining cell pressure with the drainage lines open. Under isotropic conditions, the confining cell pressure is equal in all directions. After consolidation is complete, the drainage lines are closed and the sample is compressed.

UU and CIUC samples contained in shelby tubes were extruded, trimmed and set-up at the specified confining pressures of 2,000 psf for BD-01R/T-1 (24.0’-26.0’), 2,500 psf for BD-04R/T-1 (21.0’-23.0’), 900 psf for BS-02/T-1 (7.0’-9.0’), 2,800 psf for BS-04/T-1 (23.0’-25.0’), 1,700 psf for BS-08/T-1 (15.0’-17.0’), 2,300 psf for IS-02/U-1 (19.0-21.0’), 1,600 psf for W-05/T-1 (15.0’-17.0’), and 1,700 psf for W-08/U-1 (14.0-16.0’. The confining pressures were chosen to simulate the existing loading conditions of the samples when they were collected. The confining pressure for the composited, remolded sludge was 1,000 psf. The CIUC samples were also contained within Shelby tubes and were extruded, trimmed and set-up within triaxial cells for testing. The samples were back-pressured up to 100 psi to improve the degree of saturation. They were then consolidated to specified effective consolidation pressures of 500, 2,000 and 5,000 psf for BD-03/T-1 (23.0’-25.0’) and BS-09/T-1 (12.0’-14.0’), and 200, 2,000 and 5,000 psf for BS-03R/T-1 (8.0’-10.0’) and then sheared under undrained conditions. The effective consolidation pressures for the composited, remolded sludge sample were also 200, 2,000 and 5,000 psf. Table 5-5 presents the results.

Table 5-5. Triaxial Compression Test Results					
Boring/ Sample	**Depth (ft)**	**Description**	**Undrained Shear Strength**	**Effective Stress Parameters**	
			S_u (psf)	**ϕ'-angle**	**c' (psf)**
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	665	-	-
BD-03/T-1	23.0-25.0	Sandy Elastic SILT (MH)	-	27.4	450
BD-04R/T-1	21.0-23.0	Fat CLAY (CH)	390	-	-
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	360	-	-
BS-03/T-1	8.0-10.0	Elastic SILT (MH)	-	63.4	0
BS-04/T-1	23.0-25.0	Fat CLAY (CH)	550	-	-
BS-08/T-1	15.0-17.0	SILT with Sand (MH)	305	-	-
BS-09/T-1	12.0-14.0	Silty SAND (SM)	-	33.6	550
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (CL)	155	-	-
W-05/T-1	15.0-17.0	Sandy Lean CLAY (CL)	175	-	-
W-08/U-1	14.0-16.0	Lean CLAY (CL)	305	-	-
Composite	-	Elastic SILT (MH)	30	44.0	0
S_u: Undrained Shear Strength; **ϕ, ϕ':** Angle of Internal Friction (total and effective); **c, c':** cohesion (total and effective)					

After testing of the samples, the failure planes were analyzed and photographed (see Appendix E). In overconsolidated clays, the clays tend to expand or dilate when sheared, creating negative pore pressures, while normally consolidated clays contract when sheared, creating positive pore pressures as pore water is squeezed out. As shown on the CIUC data sheets within appendix B, excess pore pressures are generally positive for samples BD-03 and BS-09. For sludge sample BS-03, the excess pore pressures are initially positive but become slightly negative as strain continues. Since this sample was collected relatively close to ground surface, this may indicated that the sludge has dried out to a degree, therefore appearing to be slightly overconsolidated.

While in the field, the Baker field engineer conducted pocket penetrometer testing of the SPT samples. The pocket penetrometer is a hand-held device with a calibrated spring. The device is pushed into the sample until the clay is penetrated a certain distance. The resulting value of unconfined compressive strength is approximately equal to twice the undrained shear strength, as illustrated below in Table 5-6.

Table 5-6. Pocket Penetrometer Results

Boring	Depth (ft)	Description	Unconfined Compressive Strength (tsf)	Undrained Shear Strength (psf)
BD-01R	32	Clayey SAND	1.25	1.250
	42	Clayey SAND	4.0	4,000
BD-03	19	Sandy Elastic SILT (peat)	0	0
	21	Sandy Elastic SILT (peat)	0	0
BD-04R	0	Clayey SILT (fill)	4.5	4,500
	2	Elastic SILT (sludge)	1	1,000
	23	Fat CLAY (peat)	0	0
BD-05	9	Fat CLAY with Sand (peat)	0	0
	12	Fat CLAY with Sand (peat)	0	0
	36	Silty SAND	0.5	500
BS-01	10	Sandy Lean CLAY	0.5	500
	12	Sandy Lean CLAY	0.5	500
	14	Sandy Lean CLAY	1.0	1,000
	16	Sandy Lean CLAY	2.0	2,000
	18	Sandy Lean CLAY	1.0	1,000
	23	Sandy Lean CLAY	0.25	250
	24	Sandy Lean CLAY	1.0	1,000
	25	Sandy Lean CLAY	1.5	1,500
BS-02	4	Elastic SILT (sludge)	0	0
BS-05	16	Silty SAND	<0.01	<10
BS-06	10	Fat CLAY (peat)	0	0
	16	Fat CLAY (peat)	0	0
	18	Fat CLAY (peat)	<0.05	<50
BS-08	6	Organic CLAY (peat)	0.25	250
	8	Organic CLAY (peat)	0	0
	10	Organic CLAY (peat)	0	0
	12	Organic CLAY (peat)	0	0
	13	SILT with Sand	0	0
ID-01	10	Elastic SILT (sludge)	<0.1	<100
	12	Elastic SILT (sludge)	<0.1	<100
	18	Fat CLAY (peat)	<0.1	<100
	24	Fat CLAY (peat)	<0.1	<100
ID-01	35	Clayey SILT	1.0	1,000
ID-02	14	Elastic SILT (sludge)	0	0
	16	Sandy Silty CLAY (peat)	0	0
	18	Sandy Silty CLAY (peat)	0	0

Table 5-6. Pocket Penetrometer Results				
Boring	**Depth (ft)**	**Description**	**Unconfined Compressive Strength (tsf)**	**Undrained Shear Strength (psf)**
	23	Sandy Silty CLAY (peat)	0	0
	25	Sandy Silty CLAY (peat)	0	0
ID-06	16	Fat CLAY (peat)	0	0
	18	Fat CLAY (peat)	0.15	150
	19.3	Silty SAND	2.5	2,500
IS-01	12	Lean CLAY (peat)	0	0
	17	Lean CLAY (peat)	0	0
	21	Lean CLAY (peat)	0	0
	23	Lean CLAY (peat)	0	0
IS-02	14	Sandy Lean CLAY (peat)	0	0
	16	Sandy Lean CLAY (peat)	0	0
W-01	8	Fat CLAY (peat)	0	0
	14	Fat CLAY (peat)	0	0
	27	Silty GRAVEL with Sand	2.0	2,000
W-03	6	Fat CLAY (peat)	0	0
	8	Fat CLAY (peat)	0	0
	10	Fat CLAY (peat)	0	0
	17.8	Silty GRAVEL	2	2,000
W-04	8	Elastic SILT (peat)	0	0
	16	Elastic SILT (peat)	0	0
W-05	4	Sandy Lean CLAY (peat)	0	0
	6	Sandy Lean CLAY (peat)	0	0
	8	Sandy Lean CLAY (peat)	0	0
	10	Sandy Lean CLAY (peat)	0	0
	12	Sandy Lean CLAY (peat)	0	0
W-07R	15	Sandy SILT	1.25	1,250
	20	Sandy SILT	1.5	1,500
W-08	8	Lean CLAY (peat)	0	0
	10	Lean CLAY (peat)	0	0
	16	SILT	2.0	2,000

5.2.4. Consolidation Testing

Eight (8) one-dimensional consolidation tests were performed on undisturbed Shelby tube samples from borings BD-04R/T-1 (21.0'-23.0'), BS-02/T-1 (7.0'-9.0'), BS-03/T-1 (8.0'-10.0'), BS-06/U-1 (2.0'-4.0'), BS-09/T-1 (12.0'-14.0'), ID-03R/T-1 (0.0-2.0'), ID-06/U-2 (14.0'-16.0'), and IS-02/U-1 (19.0'-21.0'). One (1) consolidation test was performed on a composted, remolded sample of sludge. Consolidation tests are performed on saturated samples placed within a confining metal fixed-ring or floating ring apparatus. As load is applied to the sample, water flows from the sample and the sample volume subsequently reduces. The samples were extruded, trimmed and set-up within Antius® consolidometers and were then loaded up to 1, 2, 4, or 6 tons per square foot (tsf)) before initial unloading and then reloaded up to 2, 8, 12, or 16 tsf to better define the virgin slope and pre-consolidation characteristics. Table 5-7 presents the results.

Table 5-7. Consolidation Test Results

Boring/ Sample	Depth (ft)	Description	C_c	C_r	e_o	P_c (psf)	P_o (psf)	C_v (ft^2/day) @ 2 tsf	OCR
BD-04R/T-1	21.0-23.0	Fat CLAY	0.83	0.13	2.000	1,440	1,450	0.07	1.0
BS-02/T-1	7.0-9.0	Sandy Elastic SILT	5.64	0.64	8.150	920	380	0.01	2.4
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	1.29	0.03	3.866	4,800	538	4.93	8.9
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	0.77	0.03	2.790	6,400	336	1.37	19.1
BS-09/T-1	12.0-14.0	Silty SAND	0.48	0.06	1.279	1,660	870	2.07	1.9
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	1.10	0.05	4.289	800	104	0.61	7.7
ID-06/U-2	14.0-16.0	Fat CLAY	0.79	0.13	2.466	980	600	0.11	1.6
IS-02/U-1	19.0-21.0	Sandy Lean CLAY	0.41	0.04	0.904	2,160	1,060	0.02	2.0
Composite	-	SILT (Sludge)	0.60	0.06	3.704	-	-	3.00	-

C_c: Compression Index **C_r**: Recompression Index **C_s**: Swell Index **P_c**: Preconsolidation Pressure
P_o: Current Overburden Pressure **C_v**: Coefficient of Consolidation **OCR**: Overconsolidation Ratio

5.2.5. Miscellaneous Testing

Specific Gravity (ASTM D854). Nine (9) samples were analyzed to determine specific gravity, including BD-03R/T-1 (24.0'-26.0'), BD-03/T-1 (23.0'-25.0'), BS-02/T-1 (7.0'-9.0'), BS-03/T-1 (8.0'-10.0'), BS-06/U-1 (2.0'-4.0'), BS-09/T-1 (12.0'-14.0'), ID-03R/T-1 (0.0-2.0'), ID-06/U-2 (14.0'-16.0'), and IS-02/U-1 (19.0'-21.0'). Specific gravity is useful in determining the void ratio of a soil and to determine the density of a soil. The results are tabulated below in Table 5-8.

Table 5-8. Specific Gravity Testing Results			
Boring/Sample	**Depth (m)**	**Description**	**Specific Gravity**
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand	2.36
BD-03/T-1	23.0-25.0	Sandy Elastic SILT	2.53
BS-02/T-1	7.0-9.0	Sandy Elastic SILT	2.14
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	3.17
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	3.10
BS-09/T-1	12.0-14.0	Silty SAND (SM)	2.52
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	2.76
ID-06/U-2	14.0-16.0	Fat CLAY (CH)	2.58
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (CL)	2.63

The specific gravity of the sludges on site varies between 2.76 and 3.17, substantially higher than typically encountered soils.

Organic Content (ASTM D2974). Four (4) samples were analyzed to determine organic content, including BD-01R/T-1 (24.0'-26.0'), BD-05/S-5 (14.0'-16.0'), BS-02/T-1 (7.0'-9.0'), and BS-09/T-1 (12.0'-14.0'). The purpose of this testing was to determine the potential for secondary long-term creep settlement due to compression of organic materials. The results were fairly low as shown in Table 5-9.

Table 5-9. Organic Content Testing Results			
Boring/Sample	**Depth (m)**	**Description**	**Organic Content (%)**
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	14.8
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (CH)	5.7
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	19.3
BS-09/T-1	12.0-14.0	Silty SAND (SM)	2.9

Based upon the organic content of these samples, BD-01R/T-1 and BS-02/T-1 would not necessarily be classified as peat, but do have moderately high organic contents. The other two samples are not organic soils.

pH (ASTM D4972). Four (4) samples were analyzed to determine pH, including BD-01R/T-1 (24.0'-26.0'), BD-05/S-5 (14.0'-16.0'), BS-02/T-1 (7.0'-9.0'), and BS-09/T-1 (12.0'-14.0'). The purpose of this testing was to determine whether these samples are organic materials. The results are illustrated in Table 5-10.

Table 5-10. pH Testing Results

Boring/Sample	Depth (m)	Description	pH
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	6.9
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (CH)	6.6
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	6.7
BS-09/T-1	12.0-14.0	Silty SAND (SM)	6.5

Based upon the pH of these samples, they do not appear to be peat.

6.0 SUBSURFACE CONDITIONS

6.1. SUBSURFACE CONDITIONS

The soil profiles and the test boring logs in Appendix B, depict details related to the subsurface conditions encountered in the various borings. The stratification lines shown on the soil profiles and the test boring logs represent approximate transitions between material types. In situ, strata changes could occur gradually or at slightly different levels. Also, the borings depict conditions at particular locations and at the particular times indicated. Some conditions, particularly groundwater conditions between borings could vary from the conditions encountered at the particular boring locations. The borings encountered the following seven (7) distinct strata.

Stratum I: FILL (Existing Berm): This stratum was encountered at ground surface to a depths ranging from 0.5 feet to 14.9 feet below ground surface within all berm borings, except BS-03 and BS-06 where sludge was encountered at ground surface and BD-05 where peat was encountered at ground surface. Fill was also encountered at ground surface in several impoundment borings including ID-02, ID-04, ID-05, IS-01, and IS-02. The stratum generally consisted of black to brown to gray, loose to very dense, Silty SAND (SM) with Gravel and red-brown, Clayey GRAVEL (GC) to stiff to very stiff, yellow to light gray, Clayey SILT (ML) and red-brown, Sandy Lean CLAY (CL), with varying amounts of debris (bricks, rail ties, and similar debris). The SPT N-values ranged from 5 to 200 blows per foot (bpf), averaging 43 bpf.

Stratum II: FILL (Sludge): This stratum was encountered in all berm and impoundment borings and one wetland boring (W-08), except for BD-05, BS-01, BS-05, BS-08, and BS-09. It was found at ground surface or just below berm FILL soils to depths ranging from 2.3 feet to 19.8 feet. Soils within this layer can be generally classified as gray to dark gray, very loose to dense, Elastic SILT (MH) with varying amounts of peat and sand. The SPT-N values varied from 0 to 38 bpf, averaging 9 blows bpf, indicating a wide range of density for this layer. The sludge had liquid limits ranging from 59 to 62 percent, averaging 61 percent, and plasticity indices ranging from 1 to 10 percent, averaging 5 percent. The moisture content ranged from 73.6 to 269.9 percent, averaging 122 percent.

Stratum III: Peat: This stratum was encountered in all borings, except BS-01, BS-03, and BS-07R, generally below Stratum I and/or II to depths ranging from 8.7 feet to 29.9 feet below ground surface. In wetland borings, it was encountered at ground surface. Soils within this layer can be generally classified as gray, very loose to very dense, Elastic SILT (MH) with varying amounts of Sand and brown, very loose, Silty SAND (SM) to gray to dark gray, very soft to very stiff, Organic Fat CLAY (CH-OH); very soft to stiff, Organic Lean CLAY (CL) with varying amounts of Sand, and very soft to soft, Sandy Silty CLAY (CL-ML), with varying amounts of plant fiber. The SPT-N values varied from 0 to over 35 bpf, averaging 3 bpf. The higher blow counts were generally found near interfaces with harder soils. The soil was highly plastic with liquid limits ranging from 25.0 to 467.0 percent, averaging 141 percent and plasticity indices ranging from 15 to 256 percent, averaging 69 percent. The moisture content ranged from 26.1 to 375.7 percent, averaging 101 percent.

Stratum IV: Soft Organic Clay/Silt: This stratum was encountered in all borings, except BS-01 and BS-07R, generally below Stratum III and above Stratum IV or V. Soils within this layer can be generally classified as very loose to very dense, Elastic SILT (MH) with varying amounts of Sand to gray to dark gray, very soft to very stiff, Organic Fat CLAY (CH-OH); very soft to stiff and Organic Lean CLAY (CL) with varying amounts of Sand. The SPT-N values varied from 0 to over 7 bpf, averaging 5 bpf.

Stratum V: Stiff Clay/Silt: This stratum was encountered in borings BD-02, BS-01, BS-02, BS-03, BS-07R, BS-08, ID-03R, ID-05, ID-06, and W-05 generally below Stratum III or IV. Soils within this layer can be generally classified as brown to gray, medium dense to very dense Clayey SILT (ML) to medium stiff to very stiff Silty CLAY (CL), Fat CLAY (CH) and Lean CLAY (CL) with varying amounts of Sand. The SPT-N values varied from 4 to over 60 bpf, averaging 24 bpf.

Stratum VI: Alluvial Soil: This stratum was encountered in all borings except BD-01R, BD-02, BS-02, BS-03, BS-07R, BS-08, ID-01, ID-04, W-03, and W-08 generally below Stratum III or IV. Soils within this layer can be generally classified as grey to brown, loose to very dense, Silty GRAVEL (SM) with Sand; Poorly-Graded GRAVEL (GP) with Sand; Clayey SAND (SC); Silty SAND (SM); and Sandy SILT (ML). The SPT-N values varied from 0 to over 200 bpf, averaging 55 bpf.

Stratum VII: Residual Soil: This stratum was encountered in borings BD-01R, BD-02, ID-04, ID-06, W-01, W-02, W-03, W-07R, and W-08 generally below Stratum V or VI to the boring termination depth. Soils within this layer can be generally classified as red-brown, medium dense to very dense, Clayey SAND (SC) and Clayey SILT (ML) to very stiff to hard, Sandy Lean CLAY (CL); commonly with Shale fragments. The SPT-N values varied from 10 to over 200 bpf, averaging 59 bpf.

6.2. GROUNDWATER

Groundwater was encountered in all most borings from ground surface to as deep as 6 feet below ground surface, averaging 3.5 feet, during the field exploration. Within the berms, groundwater depths ranged from 1 foot to 6 feet below ground surface, averaging 3.8 feet below ground surface. Within the impoundments, groundwater depths ranged from 0 foot to 6 feet below ground surface, averaging 3.2 feet below ground surface. Groundwater within the wetlands was approximately at ground surface. A more accurate determination of the hydrostatic water table would require the installation of monitoring wells or piezometers. It should be noted that the actual level of the hydrostatic water table and the amount and level of perched water should be anticipated to fluctuate throughout the year, depending upon variations in precipitation, surface run-off, infiltration, site topography, and drainage.

7.0 EVALUATIONS AND RECOMMENDATIONS

7.1. RECOMMENDED SOFT SOIL PARAMETERS FOR DESIGN

7.1.1. Geotechnical Properties of Alum Sludge

Index Properties

Based upon available data on the alum sludge, the in-situ density is between 36.4 pcf to 81.9 pcf, with an average of 54.7 pcf. Laboratory test moist density ranged from 60.3 to 102.7 pcf with an average moisture density of 92.5 pcf. The moisture content ranged from 69.3% to 130.8% with an average of 93.8%. Currently, Baker tested 6 Shelby Tube samples from borings BR-3R, BS-06 and ID-03R for moist and dry density values (Table 7-1). The tested moist density values are between 83.6 and 96.4 pcf with an average of 91.9 pcf. It is noted that the moisture density in both 1997 and the current investigation is significantly greater than the in-situ sand cone test results performed in 1981.

A total of 8 specific gravity values were obtained from the 1997 and current investigations, ranging from 2.76 to 3.27 with an average value of 3.07. The specific gravity of typical natural soil ranges from 2.50 to 2.80 (US Department of the Interior, Bureau of Reclamation, 1998). The grain size distribution curves are summarized in Figure 7-1. The grain size of the alum sludge is predominantly silt size. As can be seen from Table 7-1, about half of the samples were found to be non-plastic (primarily from the 1997 investigation). The remaining samples were classified as "elastic silt". The calculated void ratio ranged from 2.361 to 4.292 with an average of 3.221. The void ratio as well as the moisture content indicates that the alum sludge falls within the typical soft soil classification. Comparing with the moisture density measured from 1981 and 1997, it appears that the moisture density of the on-site sludge has slightly increased due to long time exposure.

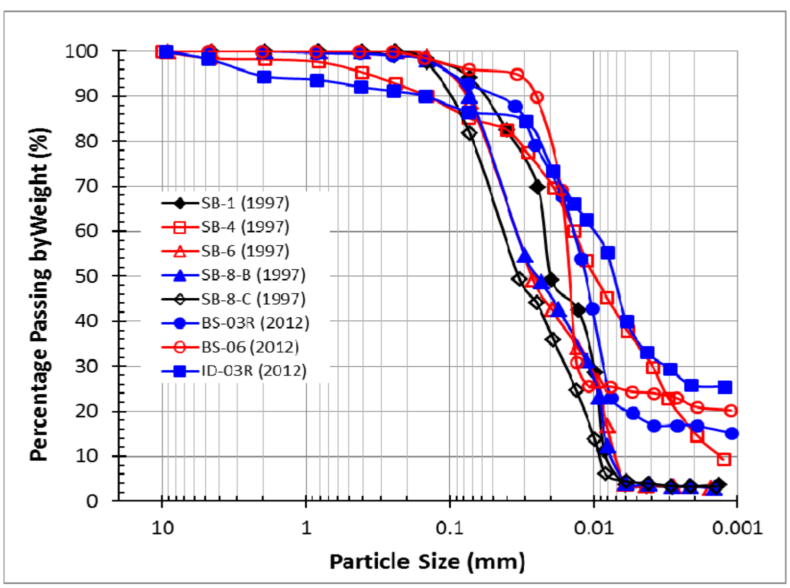
Table 7-1. Index Properties of Alum Sludge

Boring No.	Sample No	Sample Depth (ft)	Moisture Content (%)	Liquid Limit	Plastic Limit	Sieve Minus No. 200 (%)	Hydro. % Minus 2 μm (%)	Moist Density (pcf)	Dry Density (pcf)	Specific Gravity
BS-03R	T-1A	8.35	90.9	59	49			96.4	50.5	
BS-03R	T-1B	8.9	108.1					91.7	44.0	
BS-03R	T-1C	9.45	106.7					94.2	45.6	
BS-03R	T-1D	10.0	123.9			93.0	18	91.3	40.8	3.173
BS-06	U-1C	3.55	84.9	NP	64	96.0	21	94.4	51.1	3.100
ID-03R	T-1C	1.25	157.1			86.7	26	83.6	32.5	2.756
SB-1	B	2.9	112.4	78	59			86.7	40.8	
SB-1	C	7.6	105.4	NP	NP	94.4	4	92.3	44.9	3.141

Table 7-1. Index Properties of Alum Sludge										
Boring No.	**Sample No**	**Sample Depth (ft)**	**Moisture Content (%)**	**Liquid Limit**	**Plastic Limit**	**Sieve Minus No. 200 (%)**	**Hydro. % Minus 2 µm (%)**	**Moist Density (pcf)**	**Dry Density (pcf)**	**Specific Gravity**
SB-1	A	6.45	98					91.9	46.4	
SB-2	C	2.6	85.5					95.6	51.5	
SB-3	C	3.6	130.8	NP	NP			88.3	38.3	
SB-4	B	4.95	128.9	67	55	85.2	15	95.7	41.8	2.969
SB-6	B	2.95	90.7	NP	NP	89	4	93.3	48.9	2.890
SB-6	C	11.6	75.5	NP	NP			60.3	34.4	
SB-7	B	2.5	90.5					96.5	50.7	
SB-8	C	6.95	87.9					99	52.7	
SB-8	C	7.25	87.7	NP	NP	81.9	4	96.7	51.5	3.267
SB-8	C	7.7	69.3					102.7	60.7	
SB-8	B	10.5	71.7					97.9	57.0	
SB-8	B	11	79.5	NP	NP	90	4	98.6	54.9	3.271

The index properties of the on-site alum sludge are comparable with the properties of alum sludge from water treatment processes (Wu et. al., 2007; O'Kelly, 2008). Wu et al (2007) reported a case history of constructing a highway embankment over approximately 20 feet of alum sludge from a water treatment operation. The water treatment process involved the use of alum as a coagulant and lime and soda ash for softening. The index properties of the sludge were liquid limit = 125-135%, plastic limits = 40-43% and water content between 200% and 300%. O'Kelly (2008) also studied the geotechnical properties of different alum sludges from water treatment processes. The sludges had the following index properties: Liquid limit: 490%, plastic limit: 240%, specific gravity: 1.86, moisture density: 65 pcf and dry density: 9 pcf, void ratio: 12.2.

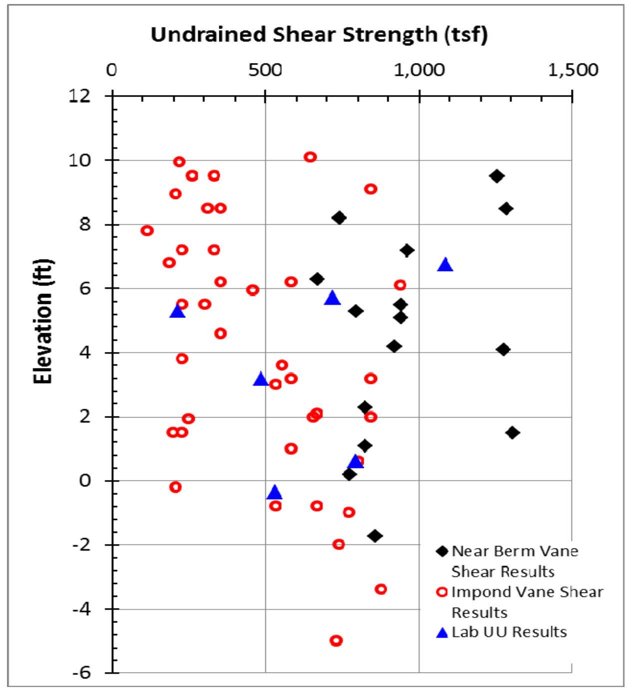
Figure 7-1 Grain Size Distribution of On-Site Alum Sludge



<u>Undrained Shear Strength</u>

When saturated soft cohesive soil is covered with overburden soils, the pore pressure within the soil will increase and the strength will decrease (in the short term). Before dissipation of the excess pore pressure, the stability problem should be evaluated by using the undrained shear strength of the soft soils (in this case, the alum sludge). The laboratory UU test results and vane shear test results (BBL, 1997) are summarized in Figure 7-2. From the field vane shear tests performed at different locations and the laboratory UU test results, it is clear that the undrained shear strength results from locations further into the ponds are generally lower than those near the berms. In addition, the results from locations within Impound 6 have the lowest undrained shear strength.

Figure 7-2. Tested Undrained Shear Strength of the On-Site Alum Sludge



The undrained shear strength can be estimated from the tip resistance from Cone Penetrometer Testing with Pore Pressure Measurement (CPTu) by using the following empirical relationship:

$$S_u = \frac{Q_c - \sigma_{v0}}{N_k}$$

where

Q_c = cone tip resistance;

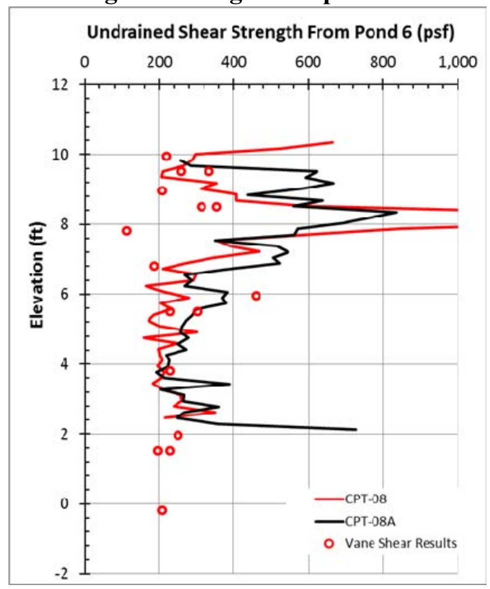
σ_{v0} = total vertical overburden pressure;

N_k = empirical cone factor.

For soft soil, N_k ranges from 8 to 15 with an average of 12.5 (Lunne et al., 1997). Due to the variation of the N_k factor, the undrained shear strength obtained from the CPTu should be calibrated with laboratory or in-situ test results.

A total of twelve (12) CPTu borings were conducted during the current investigation phase. Eight (8) CPTu borings were performed within the sludge impoundments. If an N_k factor of 15 is used in the analysis, the undrained shear strength values from the CPTu borings within Impoundment 6 (CPT-08 & CPT-08A) are similar to those from vane shear test results from the same pond (Figure 7-3). Based on this observation, it was decided that an N_k factor of 15 should be utilized to determine the undrained shear strength of the sludge.

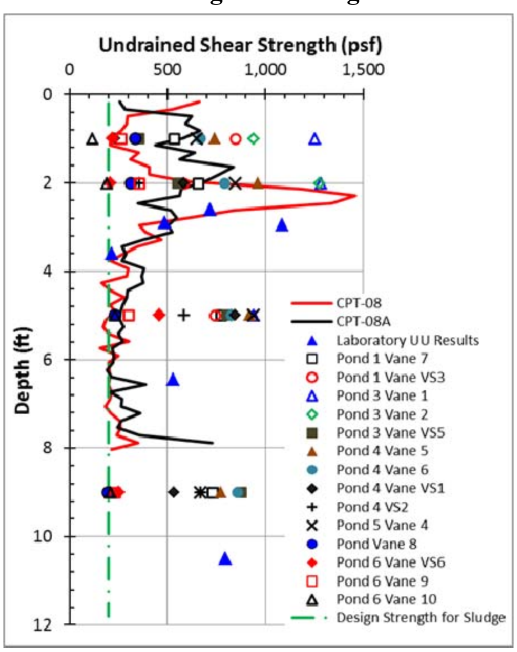
Figure 7-3 Tested Undrained Shear Strength of Sludge in Impoundment 6



O'Kelly (2005) indicated that the shear strength of the alum sludge increases with a decrease in moisture content. It is also noted that the strength of the on-site sludge varies significantly due to the desiccation (loss of moisture content), inundation, and compression by the overburden soils. The variation of shear strength was observed in the field investigation. Higher SPT N values were encountered in the sludge layers underneath the soil berms (i.e., Boring BD-01R) or above groundwater elevations (i.e., BS-03).

Figure 7-4 summarizes the undrained shear strength values from laboratory UU tests, field vane shear tests, and field CPTU tests. The undrained strength values are very scattered but a value of 200 psf represents the lowest tested undrained shear strength value of sludge. Therefore, a minimum design undrained shear strength of 200 psf is recommended for the stability analysis in this project.

Figure 7-4. Design Undrained Shear Strength for Sludge



Drained Effective Shear Strength

The long-term stability of the site under the proposed fill loads should be calculated utilizing drained (effective) strength parameters. Undisturbed sludge samples were collected and tested using the direct shear test under drained conditions in 1997 (BBL, 1997). The tested peak strength was 300 psf cohesion and 35º internal friction angle. If the cohesion is neglected, the equivalent internal friction angle under each vertical loading varies from 36º to 48º.

During the current investigation, both undisturbed as well as remolded sludge samples were tested for effective strength parameters by using the Consolidated Undrained Triaxial Shear Test with Pore Water Pore Water Pressure Measurement (CIU) method. The resulting effective shear strength parameters for the undisturbed samples were zero cohesion and an internal friction angle of 63º.

The test results for the remolded samples with the same density and moisture content as the undisturbed samples indicated an internal friction angle of 44º with zero cohesion. These tests demonstrate an unconventionally high internal friction angle.

Baker further performed an extensive literature review to detemine if these high effective strength values were reasonable. Wu et al. (2007) determined that the alum sludge at an Ohio embankment construction site had a range of internal friction angles from 38º to 41º, depending on the triaxial test method. The sample tested had an initial moisture content of 250%. O'Kelly (2008) reported a CIU test results of 39º on a sludge sample with 250% of initial moisture content. In the current study, the moisture content of the sludge samples are around 100%. It is reasonable that the strength of the on-site sludge has a higher strength. For design purposes, an internal friction angle of 36º was conservatively used in the analysis due to the small amount of sludge samples tested.

Compressibility

A total of seven (7) consolidation test results on the on-site sludge are available; three (3) tested at the current stage and four (4) samples tested in 1997. Curves showing the volumetric strain vs. consolidation pressure are presented in Figure 7-5 and the test results are summarized in Table 7-2. Only six (6) tests are shown in Table 7-2 since the results from Boring SB-8 (1997) were unreasonable and were discarded. From Table 7-2 and Figure 7-6, it can be seen that the compressibility of the different sludge samples is similar to each other in terms of the compression index as well as the recompression index. However, the "pre-consolidation pressure" is significantly different. Generally, higher pre-consolidation pressure value is associated with lower moisture content of the sample. It is worth noting that the term "pre-consolidation pressure" here is not necessarily associated with the past overburden pressure in classical soil mechanics. Besides the overburden soil pressure, the desiccation or chemical reactions within the sludge will likely increase the tested "pre-consolidation pressure".

Table 7-2. Summary of Consolidation Tests on Sludge

Sample	Depth (ft)	Water Content (%)	Effective Overburden Pressure (tsf)	Initial Void Ratio	Compression Index, C_c	Recompression Index, C_r	P'c (tsf)	OCR
BS-03R/T-1D	10	123.9	0.269	3.875	1.287	0.029	2.4	8.9
BS-06/U-1C	3.55	84.9	0.168	2.790	0.731	0.030	3.2	19.1
ID-03R/T-1C	1.25	157.1	0.052	4.289	0.968	0.053	0.4	7.7
SB-1	7.6	105.4	0.207	3.362	1.152	0.022	2.7	13.0
SB-4	4.95	128.9	0.151	3.948	0.975	0.020	0.9	5.9
SB-6	2.95	90.7	0.138	2.686	0.767	0.022	3.1	22.5
Average		115.2	NA	3.492	0.980	0.029	2.12	12.9
Standard Deviation		27.0	N/A	0.656	0.215	0.012	1.18	6.7

At Ohio embankment over sludge site (Wu et al., 2007), the tested pre-consolidation pressure of 0.3 tsf for sludge with 250% moisture content seems to be independent with the depth. At current site, a minimum pre-consolidation pressure of 0.4 tsf is assumed for the settlement analysis.

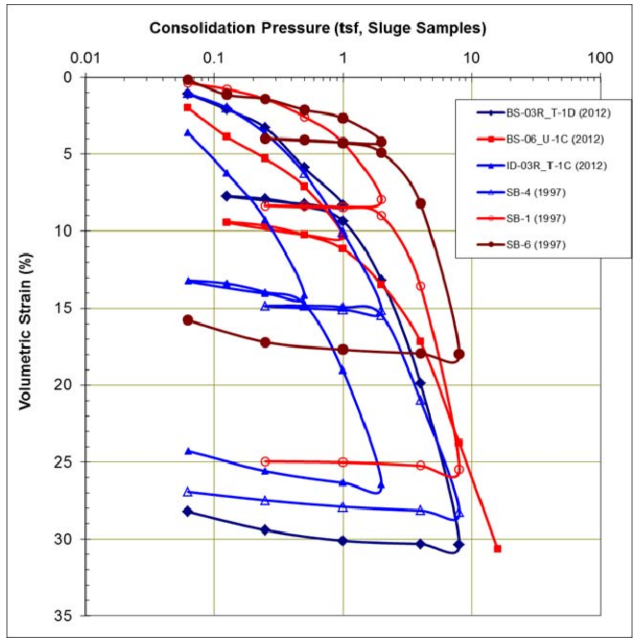


Figure 7-5. Summary of Consolidation Test Results on Sludge

Settlement Rate

The settlement rate of the soft sludge is related to the rate of consolidation. The rate of consolidation in the vertical direction is evaluated through laboratory consolidation tests. The rate of consolidation in the horizontal consolidation can be evaluated using CPTu dissipation test result. The laboratory tested coefficients of consolidation values in the vertical direction are summarized in Table 7-3. The average coefficient of consolidation is several square feet per day and the value does not decrease with an increase in consolidation pressure. The test results are very similar to the value of average laboratory tested coefficient of consolidation of 3.72 ft^2/day determined by Wu et al. (2007). The coefficient of consolidation of a typical soft cohesive soil is 0.05 square feet per day.

Table 7-3. Summary of Laboratory Tested Coefficient of Consolidation

Boring/Sample I.D.	Effective Overburden Pressure (tsf)	Coefficient of Consolidation (ft^2/day) at Consolidation Pressure (tsf)							
		0.0625	0.125	0.25	0.5	1	2	4	8
BS-03R/T-1D	0.269	6.85	6.43	6.16	6.00	5.37	5.06	4.65	3.80
BS-06/U-1C	0.168	7.18	6.41	5.53	5.67	1.18	1.77	0.83	0.18
ID-03R/T-1C	0.052	0.24	0.22	0.13	0.27	0.59	0.72		
SB-1	0.207		10.65	19.27	7.42	6.69	9.65	7.17	6.89
SB-4	0.151	1.85	4.79	3.32	2.16	2.17	2.65	3.25	3.36
SB-6	0.138	6.12	4.56	4.49	5.93	0.81	17.47	16.54	7.47
Average		3.72	4.74	5.59	3.99	2.54	5.62	6.07	4.95

The coefficient of consolidation in the horizontal direction, C_h, can be obtained through the use of CPTu dissipation test results and the following equation (Houlsby and Teh, 1988):

$$C_h = \frac{T^*_{50} R^2 \sqrt{I_r}}{t_{50}}$$

where:

T^*_{50} = modified dimensionless time factor for 50% of pore pressure dissipation, as recommended by Houlsby and Teh (1988), T^*_{50} = 0.245 for pore pressure filter at the U_2 location.

R = Radius of the CPTu probe (1.83 cm or 0.72 inch).

I_r = Soil rigidity index. A rigidity index of 100 was used for the sludge.

t_{50} = Time duration at 50% dissipation of excess pore pressures.

Robertson *et al.* (1992) reviewed dissipation data from CPTU tests to predict the coefficient of consolidation using Houlsby and Teh's (1988) solutions with reference values from laboratory tests and field observations. The review showed that the Teh and Houlsby solution provided reasonable estimates of C_h. Results were evaluated for pore pressure data from different filter locations and the least scatter was obtained with the pore pressure element location immediately above the cone *(u_2)*. The coefficients of consolidation in the horizontal direction estimated from the CPTU dissipation test results are summarized in Table 7-4.

Table 7-4. Summary of CPTu Dissipation Results

Boring	Dissipation Depth (ft)	Effective Overburden Pressure (tsf)	t_{50} (min)	t_{50} (second)	c_h (ft²/day), Houslby and Teh (1988)
CPT-04	11.3	0.410	0.89	53.3	22.36
CPT-05	10.8	0.375	0.75	45.2	28.48
CPT-08	10.5	0.375	0.16	9.6	113.32
CPT-09	7.05	0.268	0.24	14.2	87.35
CPT-12	7.05	0.268	1.17	70.4	17.96
				Average Value:	**53.89**

The average in-situ CPTu dissipation tested coefficient of consolidation in the horizontal direction is approximately one magnitude higher than the laboratory tested value. This result is consistent with CPTu dissipation results in other stratified soils. A coefficient of consolidation in the vertical direction of 4 square feet per day was used in estimating the settlement rate of the on-site sludge.

Tested Permeability of the Alum Sludge

The permeability of the impound sludge has historically been extensively evaluated. In June 1981 (M. Disko Associates), the remolded sludge samples from all six impoundments were laboratory tested for permeability (Table 7-5). M. Disko Associates (September 1981) also tested the permeability of sludge samples at depths 2 ft above the underlying soil. The average coefficient of permeability is $4.4(10)^{-5}$ cm/sec for remolded soils and $3.67(10)^{-5}$ cm/sec for in-situ sludge. It was concluded that the permeability of the sludge is little different between remolded and undisturbed samples.

Table 7-5. Laboratory Tested Permeability Results on Sludge (June & Sept., 1981)

Sample	Unit Weight (pcf)	Permeability (cm/sec, Remolded Sample, Jun.)	Permeability (cm/sec, Undisturbed Sample, Sept.)
Impound 1 (Edge)	59.09	1.19E-04	1.49E-05
Impound 1 (Center)	51.51	6.57E-06	3.65E-05
Impound 2 (Edge)	36.42	5.61E-05	1.67E-05
Impound 2 (Center)	46.75	6.05E-05	1.05E-05
Impound 3 (Edge)	81.88	4.02E-05	2.58E-05
Impound 3 (Center)	81.32	2.63E-05	5.90E-06
Impound 4 (Edge)	51.96	1.03E-04	8.07E-06
Impound 4 (Center)	48.92	5.73E-05	3.06E-06
Impound 5 (Edge)	39.7	8.06E-06	8.00E-05
Impound 5 (Center)	50.6	2.28E-05	3.09E-05
Impound 6 (Edge)	66.47	1.34E-05	9.15E-05
Impound 6 (Center)	42.31	1.62E-05	2.67E-05

The permeability can also be indirectly evaluated from the consolidation test results (Table 7-6). The samples below were from the 1997 and current (2012) investigations. The permeability values range from $7.94(10)^{-6}$ cm/sec to $5.70(10)^{-7}$ cm/sec. The calculated permeability generally reduces with an increase in consolidation pressure.

Table 7-6. Permeability of Sludge From Laboratory Consolidation Tests

Boring/Sample I.D.	Coefficient of Permeability (cm/sec) at Consolidation Pressure (tsf)							
	0.0625	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**
BS-03R/T-1D	5.16E-06	9.74E-06	5.33E-06	3.39E-06	2.77E-06	8.32E-07	1.20E-06	8.32E-07

Table 7-6. Permeability of Sludge From Laboratory Consolidation Tests								
Boring/Sample I.D.	**Coefficient of Permeability (cm/sec) at Consolidation Pressure (tsf)**							
	0.0625	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**
BS-06/U-1C	1.23E-05	1.85E-05	7.29E-06	3.56E-06	5.36E-07	1.53E-07	1.46E-07	2.68E-08
ID-03R/T-1C	1.40E-06	7.64E-07	3.38E-07	4.97E-07	4.25E-07	4.64E-07		
SB-1	7.96E-06	8.81E-06	7.68E-06	2.46E-06	1.92E-06	2.44E-06	6.45E-07	1.51E-07
SB-4	2.54E-06	7.15E-06	3.78E-06	2.11E-06	1.63E-06	1.35E-06	5.86E-07	5.49E-07
SB-6	1.24E-06	2.67E-06	2.78E-06	1.04E-06	1.18E-07	8.15E-07	1.12E-06	1.29E-06
Average	5.10E-06	7.94E-06	4.53E-06	2.18E-06	1.23E-06	1.01E-06	7.39E-07	5.70E-07

In the current investigation, one composited, remolded sludge sample was laboratory tested using using the constant head permeability method, resulting in a permeability value of $6.43(10)^{-6}$ cm/sec, which is very similar to in-situ test results and laboratory test results on undisturbed samples.

7.1.2. Organic Clays/Silts and Peat

Based on the subsurface investigations, the soil underlying the sludge (and earthen berms) consists of a peat layer and a soft organic clay/silt layer followed by a stiff clay or alluvial layer before hard residual soil is encountered. The peat layer however is commonly intermingled with the underlying soft clay/silt so that a clear interface between the two often cannot be defined. Beneath the berms and impounds, the two layers are roughly 10 to 11 feet thick, while within the wetlands, the two layers are closer to 16 feet thick. The lower thickness of the peat and clay/silt layer on the Rahway Arch property may be due to settlement caused by the overlying sludge and berm materials.

According to Leroueil and Rowe (2001), peat is defined as a soil material with ash content less than 20% or organic content greater than 80%. Typically, this material also has very high moisture contents, low dry density, very low shear strength, and very high compressibility (Samson and Rochelle; 1972, Rowe et al, 1984; Duncan et al, 1989; Schober et al, 1993; Volk et al, 1994; Rowe and Mylleville, 1996; Mesri et al, 1997, Leroueil and Rowe, 2001). Samson and Rochelle (1972) reported a case history involving the use of surcharge loading at a highway project in Canada where construction of an embankment over peat deposits was conducted. The original thickness of the peat material varied from 11 to 19 feet with an average organic content of 91%, void ratio of 14 and moisture content of 890%. Under an approximate embankment and surcharge height of 10 feet, the observed total settlement during the two year construction period was 5 to 11 ft.

Rowe et al (1984) reported that a roadway embankment 19.5 ft high was constructed over a highly compressible peat deposit using geotextile reinforcement. The thickness of the peat in this deposit varied between 15.8 ft and 18.7 ft. The average moisture content of peat was 445% and 785% at the two investigated Sections (A and B). The average unit density was 66.2 pcf with an organic content of 50%. The average in situ vane shear strength was 355 psf with a "remolded shear strength" of 135 psf. The observed maximum settlement was 15.4 ft.

Volk et al (1994) reported on the construction of an embankment over peat/organic soil in the coastal plain of New Jersey. The peat/organic soil thickness varied between 20 to 23 ft; the moisture content ranged from 97% to 240%; liquid limits were between 90% and 200%, and plasticity indices ranged from 30% to 80%. The measured total unit weight ranged from 70 to 90 pcf. The laboratory tested unconsolidated undrained shear strength was 100 psf for the root mat at the top and varied from 30 psf to 200 psf in the organic soil layer. The design embankment height varied from 9 feet to 15 feet high. The final embankment elevation was achieved through a staged construction process with four embankment loading stages in order to maintain the stability of the embankment. The observed vertical settlement was approximately 6 feet with 2 feet of lateral deflection.

The organic content of the soft clays/silts and peat at this site were generally less than 20%, therefore they don't meet this definition, however, they have other properties in common with peat, such as high liquid limits and plasticity indices, and elevated moisture content. As indicated from the above summary, the index and mechanical properties of the soil samples are very important to the design and construction of the embankment.

<u>Index Properties</u>

The index properties of the organic clays/silts and peat at the site are summarized in Table 7-7. As can be seen from this table, the organic content ranged from 24% to 39%. According to Leroueil and Rowe (2001), peat is a soil with an organic content greater than 80%, therefore the soft soil at the site might not be classified as a "peat". And according to the Unified Soil Classification System (USCS), the soft soils at the embankment site can be classified as Elastic Silt with Sand (MH) or Silt with Sand (ML).

The void ratio of the soil was determined using the one-dimensional consolidation test. This value ranged from 5.758 to 11.385, indicating that the soil might be very compressible. The unit weight varied from 63.2 pcf to 73.9 pcf. Considering the potential moisture lost during sample extraction and transportation, an average unit weight of 70 pcf is recommended for the analysis. Comparing the unit weight of the soils at the project site with recent case histories of highway embankment construction over organic soil within the coastal plain of New Jersey (Volk et al, 1994), it is clear that the unit weight at the current project site is even smaller.

Table 7-7. Index Properties of Organic Clays/Silts and Peat

Boring No.	Sample No	Sample Depth (ft)	Moisture Content (%)	Liquid Limit	Plastic Limit	Sieve Minus No. 200 (%)	Hydro. % Minus 2 μm (%)	Moist Density (pcf)	Dry Density (pcf)	Specific Gravity
BD-01R	T-1	25.0	72.9	291	173	79.0	16.0	100.6	58.2	2.36
BD-03	T-1	24.0	65.1-80.9	105	45	66.0	18.0	90.2	51.4	2.53
BD-03	S-12	26.0	73.8	89	35	57.0	14.0			
BD-04R	T-1	22.0	75.0-81.9	98	41	93.0	18.0	93.1	51.2	
BD-05	S--5	15.0	70.9	78	34	82.0	16.0			
BS-02	T-1	8.0	375.7-384.1	467	256	62	21	78.3	13.6	2.14
BS-04	T-1	24.0	71.9-110.5	93	58	87	21	96.0	45.6	
BS-09	T-1	13.0	58.6-170.9	241	113	43	8		24.3-92.6	2.52
ID-06	U-2	15.0	96.4-120.9	130	43	94	32	91.3	46.5	2.58
IS-02	U-1	20.0	37.5-223.1	33	22	69	14		22.7-86.2	2.63
W-05	T-1	16.0	26.1-27.0	25	15	53	15	123.6	97.3	
W-08	U-1	15.0	30.4	36	16	87	33	119.8	91.9	
A-5-3	TS-1	5.0	63.6-93.6	52	40	98.3	17	89.2-98.6	46.1-59.5	
A-5-4	TS-1	3.0	64.1-77.1	38	32	98.4	3	94.3-97.5	55.5-56.1	
A-7-1	A	11.0	18.4-454.1	258	105	63.6	21	80.2-107.4	31.5-67.7	
A-7-3	B	9.0	47.3-145.9	106	37	86.2	32	86.7-91.7	37.8-45.8	
CR-10	B	9.0	21.6-22.4	28	20	88.6	10	117.5-127.9	105.2	
CR-24	D	7.0	89.8-557.8	398	144	80.2	27	65.6-74.0	10.0-11.5	
CR-36	ST-1	6.0	92.2-142.4	122	42	98.4	38	83.2-97.9	34.3-51.0	
CR-41	C	8.0	50.3-91.6	59	41	96.4	13	91.4-97.7	47.7-56.1	
CR-66	TS-2	7.0	15.3-22.0	24	16	64.6	10	126.2-132.5	108.6	
CR-74	TS-1	9.0	51.4-60.8	44	39	89.6	11	99.0-102.4	63.0-67.7	

Undrained Shear Strength

For geotechnical analyses, the undrained shear strength, S_u, of the underlying peat and clay/silt soils was evaluated using several methods and data sources, including derivation of field CPT derived strengths using an N_k factor of 15, using shear strengths derived from laboratory unconsolidated-undrained (UU) triaxial tests, and using the SHANSEP (Stress History and Normalized Soil Engineering Parameter) method. SHANSEP is an empirical method used to estimate the undrained shear strength of cohesive soils (Ladd et. al, 1993). It is a common approach recommended in the FHWA Soil and Rock Properties manual (FHWA-IF-02-034). The equation is given as: $S_u=0.23\sigma'_v OCR^{0.8}$.

The soil strengths were evaluated for different locations, namely impound soils, soils underneath or close to the existing berms, and wetland soils. Results are presented in Figure 7-6 to 7-8. Two major findings are presented herein:

Figure 7-6. CPTs Near Berms – Initial Undrained Shear Strength (psf)

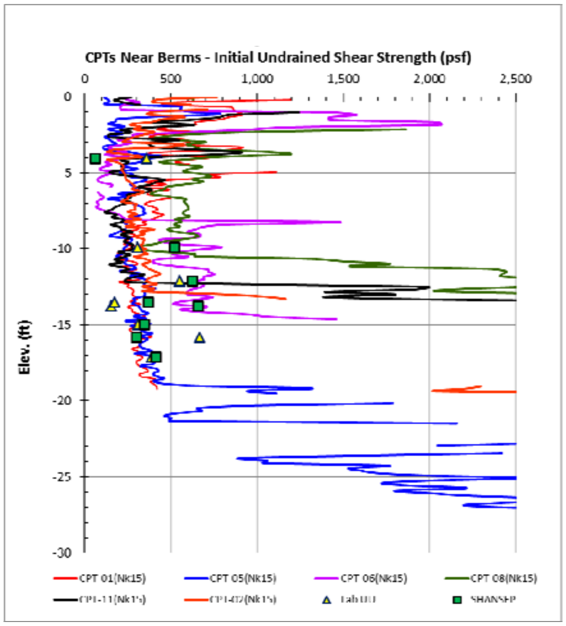


Figure 7-7. Impound CPTs – Initial Undrained Shear Strength (psf)

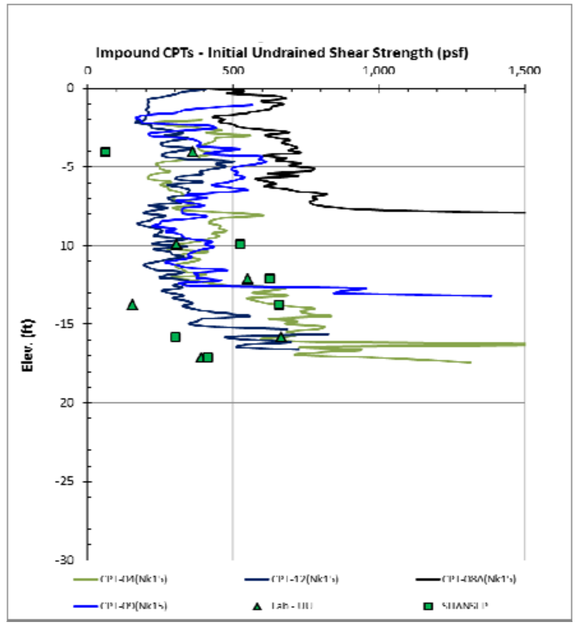
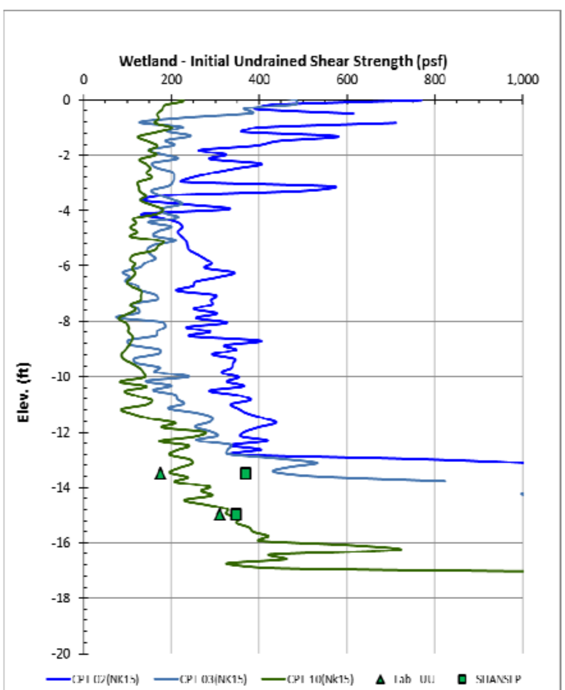


Figure 7-8. Wetland CPTs– Initial Undrained Shear Strength (psf)



- The organic peat and clay/silt soils within the impounds and underneath the berms have higher strengths than those at the wetland areas. CPT curves for soils underneath the berms are more scattered at the top perhaps due to mixing of the overlying berm materials with the native clays.
- The results from the laboratory UU tests, field CPT data interpretation, and the SHANSEP method correlate fairly well with each other.

From these results, the initial S_u (prior to loading by the proposed cap) was determined to be 240 psf for impound peat and organic clays, 260 psf for soils underneath the existing berms, and 190 psf for wetland soft clays. Upon consolidation, the soil strengths are expected to increase. Depending on specific locations, a shear strength value (S_u) of 400 psf to 450 psf (and 220 psf for wetland soils) was used for geotechnical analyses for subsequent construction stages. These values were determined from using the upper bound of the CPT curves. Overall, these estimated soil shear strengths are conservative.

Compressibility

A total of fifteen (15) consolidation test results on the on-site sludge are available; five (5) tested during the current investigation and ten (10) samples tested in 2006. Test results are summarized in Table 7-8. In order to reasonably estimate the settlement amount, one of the key parameters is the soil pre-consolidation pressure, P_c, as it is used to determine whether the in-situ soil is normally consolidated or over-consolidated.

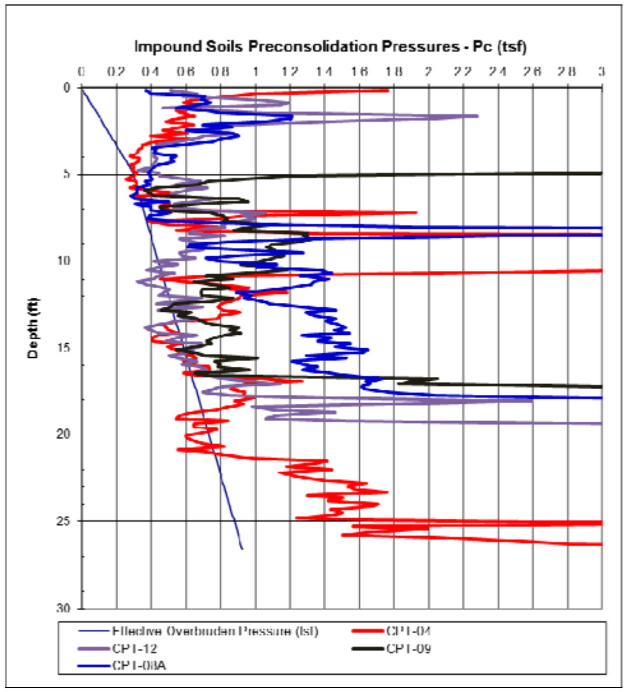
The laboratory consolidation tests included an estimation of P_c using the Cassagrande method, typically involving a search for the sharpest curvature (a turning point) of the consolidation e-log(p) curve. However, the judgment of the curvature is highly arbitrary and empirical. Current laboratory test results generally indicate higher than expected P_c. Due to the highly sensitive and soft nature of the in-situ soils and thus the high tendency of sample disturbance during transportations, these estimated pre-consolidated pressures may not be reliable.

Table 7-8. Summary of Consolidation Tests on Peat and Clay/Silts

Sample	Depth (ft)	Water Content (%)	Effective Overburden Pressure (psf)	Initial Void Ratio	Compression Index, C_c	Recompression Index, C_r	P'c (psf)	OCR
BD-04R/T-1	22.0	75.0	1,450	2.000	0.83	0.13	1,440	1.0
BS-02/T-1	8.0	375.7	380	8.150	5.64	0.64	920	2.4
BS-09/T-1	13.0	58.6	870	1.279	0.48	0.06	1,660	1.9
ID-06/U-2	15.0	120.9	600	2.466	0.79	0.13	980	1.6
IS-02/U-1	20.0	99.3	1,060	0.904	0.41	0.04	2,160	2.0
A-5-3/TS-1	5.0	93.6	188	2.643	0.743	0.026	2,000	10.6
A-5-4/TS-1	3.0	75.8	113	2.106	0.547	0.037	4,200	37.2
A-7-1/B	11.0	154.2	414	3.650	1.841	0.237	600	1.5
A-7-3/B	9.0	129.4	338	3.620	1.705	0.162	600	1.8
CR-10/B	9.0	21.6	338	0.591	0.062	0.006	9,400	27.8
CR-24/D	7.0	557.8	263	10.147	5.127	0.959	400	1.5
CR-36/ST-1	6.0	142.4	226	3.802	1.623	0.120	400	1.8
CR-41/C	8.0	91.6	301	2.405	0.678	0.027	2,400	8.0
CR-66/TS-2	7.0	22.0	263	0.604	0.008	0.101	6,400	24.3
CR-74/TS-1	9.0	60.8	338	1.578	0.428	0.008	9,000	26.6
Average		138.6	476	3.063	1.394	0.179	2837	10.0
Standard Deviation		143.0	371	2.706	1.713	0.267	3039	12.4

Both Paul Mayne (2007) and Dermers (2002) provided an alternative approach to estimate P_c based on in-situ CPT data. The formula is given as $P_c = (1/3.4) \times (q_t - \sigma'_v)$ or similarly, $0.33P_c \times (q_t - \sigma'_v)$, where q_t is the cone tip resistance and σ'_v is the effective overburden stress of the soil. These approaches involve the use of in-situ data so that the sample disturbance issue can be avoided. For the current settlement analyses, the P_c was determined using these approaches. Results are provided in Figure 7-9. The preconsolidation pressure for peat and the soft organic clay/silts was estimated to be 0.6 tsf and 0.65 tsf, respectively. As can be seen from Figure 7-9, the soils close to ground surface have higher values of preconsolidation pressure possibly due to drying.

Figure 7-9. Impound Soils Preconsolidation Pressures – Pc (tsf)



7.1.3. Stiff Clay, Alluvial Soils, and Residual Soils

As stated earlier, these soils consist of relatively dense sands and gravels (Stratum VI and some Stratum VII soils) and stiff to very stiff clays and silts (Stratum V and some Stratum VI soils). There is no available shear strength or consolidation testing data available for these soils, primarily due to the difficulty in obtaining undisturbed samples in dense cohesionless soils and very stiff cohesive soils. Stratum V (stiff clay/silt) soils within this layer can be generally classified as Clayey SILT (CL) to Silty CLAY (CL), Fat CLAY (CH) and Lean CLAY (CL) with varying amounts of Sand. The SPT-N values varied from 4 to over 60 bpf, averaging 24 bpf, with the bulk of the N-values lying between 10 and 40. Stratum VI soils (alluvial soils) can be generally classified as Silty GRAVEL (SM) with Sand; Poorly-Graded GRAVEL (GP) with Sand; Clayey SAND (SC); Silty SAND (SM); and Sandy SILT (ML). The SPT-N values varied from 0 to over 200 bpf, averaging 55 bpf, with the bulk of the N-values lying between 15 and 50. Stratum VII soils (residual soils) can be generally classified as Clayey SAND (SC) and Clayey SILT (ML) to Sandy Lean CLAY (CL). The SPT-N values varied from 10 to over 200 bpf, averaging 59 bpf, with the bulk of the N-values lying between 25 and 65.

7.2. GEOTECHNICAL ANALYSIS

The design and construction of an embankment over very soft organic soil/peat materials has been widely discussed and case histories have been reported (Weber, 1969; Samson and Rochelle, 1972; Lefebvre et al., 1984; Rowe et al., 1984; Bonaparte and Christopher, 1987; Duncan et al., 1989; Schober et al., 1993, Volk et al., 1994; Holtz et al., 1998; Christopher et al., 2000; Kerr et al., 2001; Leroueil and Rowe, 2001; and Hinchberg and Rowe, 2003). Different design procedures have been developed to address the very low shear strength and very high compressibility of the organic soil/peat material (Holtz et al., 1998; Christopher et al., 2000; and Leroueil and Rowe, 2001).

The Federal Highway Administration (FHWA) provided a guideline for the design and construction of an embankment over soft soils (FHWA HI-95-038). Most findings from aforementioned studies are incorporated in the manual. For the current remediation project, this guideline is followed step by step to perform analyses and design of the fill cap.

7.2.1. Define Fill Cap Dimensions and Loading Conditions

As shown in the cross sections provided by Eaststar, the cap is to be constructed to a maximum elevation of EL ±36 with side slopes being no steeper than 8H:1V. The only exception is for impound 6, which has a lower elevation compared to the other impounds (EL ±26). It should also be noted that toes of the proposed fill caps are always retained by berms. Berms will be proposed if there is no existing one to retain the cap. According to the proposed cross sections, the top of retaining berms are at E.L. 10 with a minimum width of 20 ft. For geotechnical analyses, an 11-ft wide berm is conservatively used based on the measured minimum width of existing berms surrounding the site. Generally, the existing soil berms have a slope of greater than 3H:1V towards the lower wetlands at E.L. 2.

Within the project limits, one section representing an unfavorable subsurface condition was selected for various analysis purposes. The section was cut approximately from the southwest edge of Impound No. 6 where boring BS-01 was located, to the east edge of Impound No. 2. The proposed cap fill along the section spans approximately 1,100 ft. This section was accurately modeled for settlement studies. In the final global stability models, however, the fill cap width was cut in half to 700 ft to simplify the analysis.

7.2.2. Establish Soil Profile and Determine Engineering Properties of Foundation Soil

In all analytical models, the impound surface was considered to be at elevation EL 10, which is level with the top of the existing berm. The in-situ soils were characterized, in descending order, as a 13 feet thick sludge layer, 5 feet of peat meadow mat, 7 feet of organic clay/silt, and a 2 feet thick stiff clay layer. Lying underneath these compressible soils is the dense to very dense alluvial and residual soil which extends to elevation EL -40, approximately the top of the shale bedrock. The residual soil stratum is hard enough to be considered incompressible. Detailed discussions regarding the strengths and compressibility of the each soil layer are provided in the previous sections.

It should be noted that the field investigation indicated that the actual impound sludge layer has a maximum thickness of roughly 20 feet. However, a portion of the sludge has SPT-N values greater than 15, indicating a very stiff material. This portion was excluded in the sludge analyses. The modeled sludge layer in the analysis represents the soft part of the actual material. A thickness of 13 feet is reasonably conservative.

7.2.3. Obtain Engineering Properties of Fill Materials

The cap material used by Soil Safe for previous projects has a wide grain size range. It has been classified as silty gravel (GM), silty sand (SM) and clayey sand with silt (SC-SM) under the USCS classification system or as A-2-4 under the AASHTO classification system. The bulk density of the material varies from 102 pcf to 127 pcf with natural moisture contents in the range of 7%-10%. Upon compaction at the optimum water content, the maximum dry density of the material can be 127.7 pcf. According to the Remedial Action Workplan, the same cap material will be used for the current project. Permeability of the cap fills is no greater than 2×10^{-6} cm/sec. In the analyses, this material was conservatively modeled as granular sand with a moisture density of 120 pcf and an internal friction angle of 33˚, although the actual material is believed to have some cohesion due to its low permeability.

7.2.4. Suggested Minimum Factors of Safety

The following factors of safety are typically recommended:

- Overall Bearing Capacity: 1.5
- Global (Rotational) Shear Stability at the End of Construction: 1.2*
- Internal Shear Stability, Long Term: 1.5
- Lateral Spreading (Sliding): 1.5

*: FHWA suggests an F.S of 1.3 for the global stability of embankments in the short term, utilizing rotational critical failure circle methods.

For this project, considering the highly sensitive and soft nature of the in-situ soils, a random failure plan search method was used instead. The random method searches the most critical curve along the weakest zone within soil layers and such a curve does not necessarily need to be circular in shape. Therefore, this method should yield more realistic results. Past experience on other projects related to highway embankments on soft soils indicate that the F.S. using the random search method is 0.3 to 0.6 lower than those derived from the conventional circle methods. Since this method typically yields more conservative results, it was decided to use a F.S of 1.2 for short term stability. The F.S. for long term global stability analysis, however, was maintained as 1.5, regardless of the analysis methods used.

7.2.5. Check Bearing Capacity

Due to the large width of the proposed cap fill (> 600 ft), the bearing capacity of the foundation soil estimated from conventional methods is not a concern. However, both Holtz et al (1998) and Leroueil & Rowe (2001) point out that classical analyses are not appropriate if the thickness of the underlying soft deposit is small compared to the width of the embankment. In this case, high lateral stresses in the confined soft stratum beneath the embankment could lead to a lateral squeeze type failure. Failures due to lateral squeeze can be evaluated based on FHWA's Soils and foundation Workshop Manual (FHWA-NHI-00-045, 2000). The threshold condition for lateral squeeze is given as:

$$\gamma_{fill} H < 3S_u$$

where γ_{fill} is the unit weight of the fill, H is the height of the embankment, s_u is the undrained shear strength of the foundation soil.

Assuming that the initial S_u of the sludge deposit is 200 psf and the initial fill density is 120 pcf, the maximum height of the cap is calculated as 5 feet, indicating that staged construction is required for cap installation. Each lift shall not be greater than 5 feet. After the completion of the first lift, the undrained shear strength of the sludge shall increase upon its consolidation. Since the consolidation coefficient (C_v) of the sludge is high, the waiting period between each construction stage is not expected to be long.

According to the Workshop, the equation for Safety Factor against Lateral Squeeze is given as:

$$FS = \left[\frac{2S_u}{\gamma \cdot D_s \cdot \tan\theta}\right] + \left[\frac{4.14S_u}{\gamma \cdot H}\right] \quad \text{(Silvestri, 1983)}$$

where:

θ = angle of slope ($\tan(\theta)$ = 1V : 8H = 0.125)
γ = unit weight of the fill (120 pcf)
D_S = depth of soft soil beneath the toe of the end slope or side slope of the fill (13-ft sludge + 5-ft peat + 7-ft organic clay + 2-ft stiff clay = 27 ft total)
H = height of the fill (first lift = 5 ft)
S_u = undrained shear strength of soft soil beneath the fill (200 psf)

Solving this equation using these parameter values, an F.S. of 2.0 can be achieved. Overall, based on the bearing capacity evaluation, the cap should be installed in stages with a maximum of 5 feet fill for each lift. A waiting period is required for each construction stage.

In a review of 40 embankment case histories (Humphrey and Holtz, 1986; Humphery, 1987), the risk of embankment failure due to bearing, even with high strength geotextile reinforcement, increases significantly (4 out of 9 cases) when the height is more than 6 to 7 feet. Wu (2005) reported using 10 feet high embankments for each construction stage with a total of 3 stages. However, the embankment was constructed using 3 layers of high strength geotextile. Considering that the current project will not apply high strength geotextile for cap reinforcement (indicated to be adequate from the stability analysis), a maximum lift height of 5 feet is reasonable.

7.2.6. Short-Term and Long-Term Global Stability

The stability of the fill cap was evaluated based on limit equilibrium theory. The random search method from the commercial software, STEDWIN (Version 2.4), was used to find the critical plane and calculate its factor of safety. The cap was assumed to be initially supported by a layer of drainage blanket lying above the sludge deposit. A full height fill cap model (36 feet) was initially analyzed as if the cap was built without staged construction. If the result from such a model can satisfy the minimum F.S of 1.2 and 1.5 for short term and long term, respectively, the stability of the actual staged cap construction would not be a concern. This height included additional amount of fills to compensate for the anticipated settlement in order to reach a final design elevation of EL ±36.

The drainage blanket is necessary for areas where the impound sludge is exposed at the ground surface, facilitating and speeding the consolidation process and strengthening the sludge. It also can serve as a protective crust for further cap installation as well as a platform for construction activities. Detailed discussions are provided in the settlement evaluation section. Soil parameters for stability analyses are summarized in the following tables:

The soil strengths at the initial stage were estimated from the in-situ CPT data calibrated with both current and existing field and laboratory test data. For peat and organic clay/silt soils, the amount of strength increase at the end stage of construction was estimated by using the SHANSEP method, which estimates shear strength using $S_u=0.23\sigma'_v\, OCR^{0.8}$. Upon consolidation, the undrained shear strength of the peat/clay soil shall increase proportionally to the effective overburden pressures induced by continuous cap installation. In the model, using an undrained shear strength of 400 psf for impound peat and organic clay at the end stage of construction is conservative. For the wetland area, however, the shear strength is not expected to increase significantly because no considerable consolidation is anticipated for soils within

Table 7-9. Initial Soil Strength (proposed for initial construction of the 3-ft drainage blanket–short term)												
Soil Layer	**Soils underneath/close to existing berms**				**Impound Soils**				**Soils at wetlands (Outside)**			
	γ_m (pcf)	**φ (˚)**	**C (psf)**	**S_u (psf)**	**γ_m (pcf)**	**φ (˚)**	**C (psf)**	**S_u (psf)**	**γ_m (pcf)**	**φ (˚)**	**C (psf)**	**S_u (psf)**
Drainage Layer (3 ft)	-	-	-	-	120	34	0	0	-	-	-	-
Existing Berm	125	34	100	0	-	-	-	-	-	-	-	-
Sludge (13 ft)	92	0	-	220	92	0	-	200	-	-	-	-
Peat (5 ft)	82	0	-	260	82	0	-	240	80	0	-	190
Soft Organic Clay (7 ft)	95	0	-	260	95		-	240	85	-	-	190
Stiff Clay (2 ft)	120	0	-	600	120	0	-	600	110	0	-	400
Residual Soil	125	35	500	-	125	35	500	-	125	35	500	-

Note: $\gamma_{m:}$ soil density; φ: soil internal friction angle; C: cohesion; Su: undrained shear strength

Table 7-10. Soil Strength (end of full height cap construction & before completion of consolidation–short term)												
Soil Layer	**Soils underneath/close to existing berms**				**Impound Soils**				**Soils at wetlands (Outside)**			
	γ_m (pcf)	**φ (˚)**	**C (psf)**	**S_u (psf)**	**γ_m (pcf)**	**φ (˚)**	**C (psf)**	**S_u (psf)**	**γ_m (pcf)**	**φ (˚)**	**C (psf)**	**S_u (psf)**
Cap Fill (Max. 32 ft)	-	-	-	-	120	33	0	0	-	-	-	-
Existing Berm	125	34	100	0	-	-	-	-	-	-	-	-
Sludge (13 ft)	100	0	-	250	100	0	-	220	-	-	-	-
Peat (5 ft)	82	0	-	450	82	0	-	400	80	0	0	220
Soft Organic Clay (7 ft)	105		-	450	105		-	400	95	0	-	220
Stiff Clay (2 ft)	120	0	-	800	120	0	-	800	110	0	0	600
Residual Soil	125	35	500	-	125	35	500	-	125	35	500	-

Table 7-11. Final Soil Strength (full Cap fill after consolidation – long term)												
Soil Layer	Soils underneath/close to existing berms				Impound Soils				Soils at wetlands (Outside)			
	γ_m (pcf)	φ (˚)	C (psf)	S_u (psf)	γ_m (pcf)	φ (˚)	C (psf)	S_u (psf)	γ_m (pcf)	φ (˚)	C (psf)	S_u (psf)
Cap Fill (Max. 32 ft)	-	-	-	-	125	33	0	0	-	-	-	-
Existing Berm	125	34	100	0	-	-	-	-	-	-	-	-
Sludge (13 ft)	115	36	0	-	115	36	0	-	-	-	-	-
Peat (5 ft)	85	0	-	450	85	0	0	400	80	0	0	220
Soft Organic Clay (7 ft)	105	0	-	450	105	0	-	400	80	0	-	220
Stiff Clay (2 ft)	123	0	-	800	123	0	-	800	112	0	0	600
Residual Soil	125	35	500	-	125	35	500	-	125	35	500	-

that area. Results of the stability analyses are presented in the following table.

Table 7-12. Stability Analysis Results		
Construction Stage	**Construction Height (ft)**	**Factor of Safety**
Initial Drainage Blanket (Short Term)	3	1.21
End of Fill Cap Construction (Before Consolidation - Short Term	32	1.20
End of Fill Cap Construction (Completion of Consolidation - Long Term)	32	1.50

With reasonably conservative soil parameters assigned, analyses results indicate that a minimum factor of safety (F.S.) for both short term and long term stabilities can be achieved. It is anticipated that the actual staged construction would have higher F.S. than the modeled case.

The global stability of the existing berm with the proposed cap fill was also evaluated. For the analysis, a failure plane was defined directly underneath the berm in the global stability model. For deep-seated shear failures, the search for failure circles was limited to within 30 feet of the berm toe. Results indicated that the defined critical planes would have an F.S of 2.25 and 1.43 in the short term, both of which are greater than the required minimum values. The sliding and overturning potential of the existing berm was also evaluated by using conventional methods (Appendix D). Based on the resulting factors of safety being greater than 3.0 for overturning and 1.3 for sliding, the existing berm appears to be stable. It should be emphasized that the analyses are based on the assumption that all berms retaining the fill cap are topped at elevation EL 10. Also, additional searches were conducted along the proposed cap slope. Results suggest that the most critical failure curve would not develop on the proposed slope. An 8H:1V slope ratio is therefore reasonable.

Overall, extensive global stability analyses indicate that the proposed cap would be stable in both the short and long term without high strength geotextile reinforcement. The most critical circle is more likely to occur near the edge of the proposed cap top where the slope starts. Therefore, it is recommended that no heavy construction live load (other than lightweight dozers) shall be applied within 50 feet of the cap edge during construction. Small temporary stockpiles should also be placed at least 50 feet from the cap edge with a lightweight dozer spreading the material from that point forward. Compaction of the first lift should only be performed by the tracking of the lightweight dozer.

7.2.7. Check Lateral Spreading (Sliding) of the Proposed Cap

According to the FHWA manual, lateral spreading for an un-reinforced embankment is evaluated via the following equation:

$$FS = \left[\frac{b \cdot \tan \phi'_{sg}}{K_a H} \right]$$

where b is the width of the proposed slope (assuming 5 ft lift and 8H:1V slope ratio, b = 40 ft), ϕ'_{sg} is friction angle between the cap material and the foundation soil (cap will be firstly constructed on a crust, $\phi'_{sg} = 30^\circ$, therefore, tan ($\phi'_{sg} = 0.58$), H is the fill height (5 ft for each lift), and K_a is the coefficient of lateral pressure (assuming an internal friction angle of 33° for the fill, $K_a = \tan^2(45^\circ - 33/2^\circ) = 0.283$). Following the equation and inputting all the known values, the calculated F.S. is greater than 16. Overall, due to the proposed gentle slope, lateral spreading of the proposed cap is not likely.

7.2.8. Settlement Evaluation

Due to the high compressibility of the site soils, the majority of the settlement is expected to be caused by the consolidation upon loading. The amount of consolidation can be estimated based on the conventional Terzaghi's one dimensional theory. Settlement parameters were obtained from current and existing laboratory tests and are summarized in the table below. The FHWA funded computer program Fossa (Version 2.0) was used to calculate soil settlement. The software is the improved and commercial version of FHWA recommended software, EMBANK. It can solve two or three dimensional embankment settlement problems with complicated boundary conditions.

Table 7-13. Primary Consolidation Parameters for Cohesive Soils

Compressive Soil Layers	γ_m (pcf)	WC (%)	Initial Void Ratio, e_0	Compression Index, Cc	Compression Ratio, CR	Recompression Index, Cr	Recompression Ratio, RR	RR/CR	P_c (tsf)
Soft Sludge (13 ft)	92	91.3	2.42	1.155	0.273	0.028	0.007	0.046	0.4
Peat (5 ft)	82	228	4.73	2.750	0.485	0.386	0.07	0.13	0.6
Soft Organic Clay (7 ft)	105	228	4.73	2.750	0.485	0.386	0.07	0.13	0.65
Stiff Clay (2 ft)	120	61	1.62	0.788	0.277	0.086	0.030	0.082	1.6

P_c: Preconsolidation Pressure
*: Depending on the significance of the fibrous organic content

Two consolidation models were developed for analyses. The first model was established to simulate the soil consolidation in exposed sludge areas upon the initial installation of a 3-ft thick drainage layer. The other model was created to evaluate the settlement due to the construction of a full height fill cap (32 ft).

Table 7-14. Settlement Rate									
Soil Layer	**Coefficient of Consolidation (ft^2/day) at Consolidation Pressure (tsf)**								**Used for analyses**
	0.0625	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**	
Soft Sludge (13 ft)	5.95	5.53	7.10	5.46	3.84	5.67	6.90	3.70	4
Peat (5 ft)	1.63	1.03	1.04	1.17	1.19	1.46	1.34	1.28	1.0-1.2
Soft Organic Clay (7 ft)	0.32	0.08	0.05	0.02	0.02	0.02	0.03	0.16	0.02-0.04*
Stiff Clay (2 ft)	0.13	0.05	0.04	0.02	0.02	0.02	0.03	0.02	0.026

*: Depends on the significance of the fibrous organic content.

An 1,100-ft wide cap was used in the model. The model considered the drainage blanket as a "new ground" after its settlement. This mean that the deformed soil layers from the initial drainage stage were inputted into the latter construction model. New void ratios and soil densities as a result of the initial consolidation were also used in the fill cap model. Results are presented in the following table:

Table 7-15. Primary Consolidation Results						
Construction Stage	**Construction Height (ft)**	**Total Primary Consolidation (ft)**	**Time to Reach 90% of The Sludge Consolidation (days)**	**Time to Reach 90% of The Peat Consolidation (days)**	**Time to Reach 75% Organic Clay Consolidation (days)**	**Time to Reach 90% of Total Primary Consolidation (days)**
Initial Drainage Blanket (3 ft)	3	1.0-1.3	38	20	110	-
Full Height Fill Cap (32 ft)	32	5.5	36	20	111	515~1010*

It should be noted that the selected section for analyses represented the most unfavorable condition at the project site. Under such a condition, the sludge is directly exposed at the ground surface without natural soil covers. Moreover, a conservatively large width of the cap was used in the analyses and only the maximum settlement value, typically occurring in at the center of the embankment, was reported. Additionally, the model did not consider the staged cap construction effects and waiting period among each stage. Therefore, the estimated result is potentially conservative.

Results indicated that a waiting period of 38 days is required after the construction of the drainage blanket. When the fill cap is to be constructed, the waiting period between each construction stage is three and a half months. After the finish of fill cap construction, it would take 1.5 to 3 years to complete 90% of the total consolidation. Due to the highly variable subsurface conditions of the project site, the actual completion of settlement and its rate should be monitored through a field instrumentation program.

7.2.9. Immediate and Secondary Settlement

In theory, the total settlement of a cohesive organic soil layer also includes an immediate settlement due to the elasticity of the soil and a secondary settlement due to gradual decay of the organic matter within the soil matrix. The immediate settlement can be estimated using Hoek's Law based on known loading conditions and the Young's Modulus (E) of the material. Young's modulus can be further converted from the CPT derived shear moduli by using the equation: E=G/2(1+v), where v is the Poisson's Ratio of the material. The immediate settlement for the drainage stage and cap construction stage is estimated as 1 inch and 5 inches, respectively. Considering the rapidity of immediate settlement and the much lower magnitude compared to consolidation settlement, immediate settlement was not taken into account for this project.

The secondary settlement can be calculated using the AASHTO recommended formula expressed as:

$$S" = C_\alpha H \log \frac{t}{t_p}$$

However, for this project, the secondary settlement was not analyzed since the major focus of the project is the stability of the proposed cap. Settlement is not a significant concern. Moreover, as compared to the primary settlement, the secondary consolidation is always minor. Past experiences on similar embankments over peat/soft clay soil indicate that secondary consolidation is typically several inches and lasts as long as 30 years if there is no preloading applied.

7.3. EXISTING BRIDGE FOUNDATION EVALUATION

The existing bridge foundation was analyzed to determine the long-term stability (bearing capacity, overturning, and sliding) of the structure. Assuming an internal angle of friction of 30 degrees based upon the SPT N-values from boring BS-09, the bridge foundation appears to be stable. Supporting calculations are contained within Appendix D.

7.4. SEISMIC CONSIDERATIONS

Liquefaction is a phenomenon related to saturated, loose and cohesionless soil deposits that are subjected to repeated shaking during an earthquake event. The liquefaction potential for a given cohesionless soil deposit is a function of various factors, including its relative density, earthquake intensity (ground acceleration), duration and the effective number of cycles of shaking (earthquake magnitude). Relatively loose, clean to moderately silty sands at shallow depths below the groundwater table are most susceptible to total or partial liquefaction.

Soil liquefaction resistance can be evaluated with different in-situ test methods. A standard procedure termed the "simplified procedure," which is largely empirical, has been evolved for evaluating the liquefaction resistance of soils. This procedure was recommended by the National Center of Earthquake Engineering (NCEER) workshop (Youd et al., 2001). The liquefaction potential of this project was evaluated using SPT blow counts and CPT normalized cone resistance and friction values. The SPT evaluation methods were based on the NCEER "simplified procedure" (Youd et al., 2001) and more up to date materials (Andrus and Stokoe, 2000; Robertson and Wride, 1998; Youd, 1998). The CPT analysis methods were based on procedures contained within Idriss and Boulanger (2008). Based on the current field testing information and laboratory test results, the underlying sludge, clays and sand and gravel soils overlying rock at the site are not likely to have a significant liquefaction potential under an earthquake with a magnitude of M~5.

Baker also evaluated slope stability during a seismic event utilizing the pseudo-static method. This method involves the inclusion of horizontal and vertical static seismic forces that are used to simulate the potential inertial forces due to ground accelerations in an earthquake. These forces are assumed to be proportional to the weight of the potential sliding mass multiplied by a seismic coefficient. Typically, the vertical forces are ignored in these analyses. The horizontal earthquake coefficient was estimated by using ½ of the peak horizontal ground acceleration (PGA) or 0.045g. The resulting factor of safety was found to exceed 1.0, the minimum recommended factor of safety for seismic events. Supporting calculations are contained within Appendix D.

7.5. INSTRUMENTATION

It is highly recommended that instrumentation be installed prior to construction and monitored during filling operations. Recommended instrumentation includes settlement plates, vibrating wire piezometers, and inclinometers. The recommended locations are shown in Figure A-13 and in the tables below.

<u>Settlement Plates:</u>
Settlement plates should be installed after 2 ft to 3 ft of the drainage blanket has been placed at the site at the following locations shown in Table 7-16:

Table 7-16. Settlement Plate Locations

Settlement Plate Number	Northing	Easting
SP-1	570681.69	642532.11
SP-2	570133.86	642776.74
SP-3	570145.24	643236.96
SP-4	570706.47	643159.61
SP-5	571229.91	643780.89
SP-6	571335.68	644248.12
SP-7	571714.86	644186.74
SP-10	571953.06	642164.95
SP-11	571830.79	641874.42

Vibrating Wire Piezometer

The vibrating wire piezometers should be installed at the following locations shown in Table 7-17.

Table 7-17. Vibrating Wire Piezometer Locations

Piezometer Number	Northing	Easting	Probe Elevation (ft)
VW-1	570676.04	642502.77	-12, -20
VW-3	570084.62	643228.39	-12, -20
VW-4	570734.91	643146.35	-12, -20
VW-6	571314.74	644279.51	-12, -20
VW-7	571689.62	644223.79	-12, -20
VW-10	571930.74	642187.58	-12, -20
VW-11	571850.29	641850.75	-12, -20

Slope Inclinometer

The slope inclinometers should be installed at the following locations shown in Table 7-18.

Table 7-18. Slope Inclinometer Locations

Inclinometer Number	Northing	Easting	Bottom Elevation of Probe (ft)
IN-1	570305.94	642055.82	-35
IN-2	569795.94	642770.90	-35
IN-3	570114.93	643235.70	-35
IN-4	570657.78	643607.96	-35
IN-5	571018.34	644186.03	-35

Table 7-18. Slope Inclinometer Locations			
Inclinometer Number	**Northing**	**Easting**	**Bottom Elevation of Probe (ft)**
IN-6	571336.17	644790.87	-35
IN-7	571726.89	644241.95	-35
IN-8	571721.79	643483.05	-35
IN-9	571419.25	642671.14	-35
IN-10	571974.44	642141.70	-35

8.0 GEOTECHNICAL RECOMMENDATIONS

Both the settlement and the stability analysis indicate that the proposed grading plan can be safely achieved as long as construction is performed in stages with waiting periods between stages. It is further understood that the completion of the fill will take approximately 5 years; therefore this should not be an issue. Based on the current available data, 90% consolidation of the cohesive soil underneath the sludge will take two to three years after the placement of fill. For a sludge layer 13 feet thick, the estimated time required to reach 90% of sludge consolidation is approximately 38 days.

The following construction sequence is recommended:

1. Prepare the Subgrade

    - Only cut trees and stumps flush with ground surface.
    - Do not remove or disturb root mat or meadow mat.
    - Leave small vegetative cover, such as grass and reeds, in place. The vegetation will help to carry the construction equipment load during the site preparation and placing the first several feet of the fill.

    Prior to receiving materials without permeability requirements, a minimum of three (3) feet of "fair drainage material" shall be placed on top of the current impoundments. The "fair drainage material" is defined as soil materials with laboratory tested permeability greater than 5.0E-05 cm/s. The laboratory permeability test procedures should conform to ASTM D2434 or ASTM D5856. The sample for the permeability test should have a minimum of 90% maximum dry density as determined by ASTM D698 (Standard Compaction Effort). The following procedure should be followed:

    a. End-dump the fill materials into the impoundment from the adjacent access road. The first lift of the fill should consist of "fair drainage material".

        - Use appropriate trucks and equipment compatible with constructability design.
        - End-dump on the previously placed fill;
        - Limit height of dumped piles, e.g., to less than four (4) ft above the adjacent sludge or five (5) feet above the previous fill, to avoid a local bearing failure. Spread piles immediately to avoid local depressions.
        - Use lightweight dozers and/or front-end loaders to spread the fill.

    b. Traffic on the first lift should be limited.

- Construction vehicles should be limited in size and weight to limit initial lift rutting to 3 inches. If rut depths exceed 3 inches, decrease the construction vehicle size and/or weight.

c. The first lift should be compacted only by tracking in place with low-ground-pressure bulldozers or end-loaders.

d. Once the cap is at least 2 ft above the original ground, subsequent lifts can be compacted with a smooth drum vibratory roller or other suitable compactor. If localized liquefied conditions occur, the vibrator should be turned off and the weight of the drum alone should be used for compaction.

e. Generally, the above procedure applies to Impoundments 1, 4, 5 and 6. Minor adjustment of the thickness of the first lift and the construction sequence might be required during construction.

2. After the "fair drainage material" (first lift) is placed, the geotechnical instrumentation needs to be installed including settlement plates, vibrating wire piezometers and slope inclinometers. Special Provisions for instrumentation will be developed during final design when a draft fill plan is available. The following are the brief summary:

   a. The settlement plates should be placed at the top of the first lift. The objective of the settlement plate installation is to control the construction sequence.
   b. For the piezometers located near the river, filters shall be installed in both organic cohesive soil as well as the sludge. At other locations, the piezometer filters are only required to be installed within the sludge. The objective of installing piezometers is to monitor the dissipation of excess pore pressures generated during construction. The shear strength of the soil will increase after dissipation.
   c. The slope inclinometers should be installed along the edge of the proposed fill to ensure that slope stability is maintained.
   d. Additional geotechnical in-situ tests such as CPTu and field vane shear testing will be required during construction.

3. End of Stage One construction. The starting time for the next stage of construction will be based on the instrumentation monitoring results. Generally, the criterion for the next construction stage is when

the soft material reaches a minimum of 90% of primary consolidation. The estimated waiting period is approximately 50 days.

4. At areas near the river (Impoundments 4 and 6), the fill placement might also be controlled by dissipation of excess pore pressure within the in-situ cohesive soils underlying the sludge. A longer waiting period time might be required. For construction activities in these areas, more frequent instrumentation monitoring will be required to ensure the rate of fill placement is slow enough to allow the dissipation of excess pore pressure.

5. Place the embankment and surcharge fills for each construction stage. The thickness of each construction should not exceed 5 ft. The fill material needs to be roller compacted. No heavy construction live loads (other than lightweight dozers) shall be applied within 50 feet of the cap edge during construction. Small temporary stockpiles should also be placed at least 50 feet from the cap edge with a lightweight dozer spreading the material from that point forward. Compaction of the first lift should only be performed by the tracking of the lightweight dozer.

6. To facilitate the dissipation of the excess pore water pressure in the in-situ cohesive soil underlying the sludge, place all fill material in a pattern with relatively the same elevation.

7. Due to the relatively high shear strength of surface material at Impoundments 2 and 3, fill material should be no more than ten feet in height above the adjacent ground. At all other areas, piles should be no more than six (6) feet in height. If higher stockpile heights are desired, it is recommended that the stockpiles be placed in a staged manner, thereby allowing the underlying soils to consolidate. Once the stockpile has been reached the desired height, further staged placement should not be required. The consolidation of the underlying materials should also be monitored with field shear strength testing completed prior to building the stockpiles to greater heights.

8. All staged cap fills shall be properly graded to divert storm water runoff away from the fill area. No concave grading is allowed that will create ponding during construction.

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10.0 BASIS OF RECOMMENDATIONS

This report has been prepared to aid in the evaluation for the proposed construction described in this report. Adequate recommendations have been provided to serve as a basis for design and preparation of plans and specifications. The opinions, conclusions, and recommendations contained in this report are based upon our professional judgment and generally accepted principles of geotechnical engineering. Inherent to these are the assumptions that the earthwork and foundation construction should be monitored and tested by an engineering technician acting under the guidance of a licensed geotechnical engineer.

These analyses and recommendations are, of necessity, based on the information available at the time of the actual writing of the report and on the site conditions, surface and subsurface, that existed at the time the exploratory borings were drilled. Further assumption has been made that the limited exploratory borings, in relation both to lateral extent of the site and to depth, are representative of conditions across the site.

The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. We cannot accept the responsibility for designs based on recommendations in this report unless site conditions are monitored and tested during construction, by a well-qualified, adequately insured and experienced engineering technician acting under the guidance of a licensed geotechnical engineer, both of whom are acceptable to Baker, to validate that the subsurface conditions exposed during construction are in general conformance with Bakers design assumptions and to ascertain that, in general, the work is being performed in compliance with the contract documents.

Our professional services have been performed in accordance with generally accepted engineering principles and practices; no other warranty, expressed or implied, is made. Baker assumes no responsibility for interpretations made by others on the work performed by Baker. We recommend that this report be made available in its entirety to contractors for informational purposes only. The boring logs and laboratory test data contained in this report represent an integral part of this report and incorrect interpretation of the data may occur if the attachments are separated from the text.

Michael Yang 11/30/12

Michael Yang, P.E. (New Jersey License No. 24GE04826000) Date Signed