REVISION 2

DEVELOPMENT OF GEOTECHNICAL DESIGN PARAMETERS FOR LAKEBED SEDIMENTS IN ONONDAGA LAKE CAPPING AREAS

This report presents a summary of the process used to estimate general geotechnical design parameters for the Onondaga lakebed sediments based on results of in situ investigations and laboratory testing. Strength and consolidation parameters used for geotechnical slope stability analyses in the various capping areas are based on the geotechnical parameters that are developed in this report. The summary of geotechnical parameters and methods used to characterize the lakebed sediments is presented in Table 1.

INTRODUCTION

The evaluation of strength data collected between 2012 and 2015 for lakebed sediments in remediation areas A through F (RA-A through RA-F) indicate that the sediments in portions of these remediation areas are significantly softer than anticipated based on pre-design investigations (PDI) conducted prior to 2012. The PDI strength assumptions (developed from in situ vane shear tests (VST)), were generally representative of the subsurface materials in water depths less than about 20 feet. VST data could not be collected in water depths greater than 20 feet due to depth limitations of the equipment. A comparison between the estimated strength parameters from the field VST data (from the PDI) and the post-PDI VST data conducted after 2012 indicates that, in general, the PDI data showed higher shear strengths for the shallow sediments than the recent data. Further discussion of the difference between the VST data from the PDI and the recently collected FFP data is presented in Appendix A.

Since the recent FFP data showed lower shear strengths than originally anticipated, the stability of the design caps detailed in the report *Onondaga Lake Capping, Dredging, Habitat, and Profundal Zone Final Design* (Final Design, submittal of March 2012) needed reevaluation. Reevaluation of cap stability in various RAs are being conducted using the geotechnical parameters of very soft to soft lakebed sediments developed from the in situ and laboratory test results collected from 2012 to 2015.

For the geotechnical evaluation of cap stability, *sediment* is considered as a general term for materials that are present at shallow depths below the lake-bottom (depths of 0 to 20 feet below the lake-bottom which includes natural soil and Solvay waste) and are more likely to impact the

geotechnical stability of the cap. In general, the subsurface materials at depths greater than 20 ft below the lake-bottom consist of Marl, silt, and clay layers.

Geotechnical site investigations were performed in 2012, 2013, 2014, and 2015 by Parsons, Anchor QEA, Geosyntec, ConeTec, and Atlantic Testing Laboratories (ATL) in the RAs. Full flow penetrometer (FFP, or ball penetration tests, BPT), piezocone penetration tests (CPT), and pore-pressure dissipation (PPD) tests were performed by ConeTec using spud and/or anchor drilling barges operated by ATL. Geotechnical laboratory testing was conducted by GeoTesting Express (GeoTesting) using the disturbed samples collected by vibracore (VC) and/or shallow/gravity core (GC) methods. Attempts were made to collect undisturbed VC and GC samples. However, the collected samples were observed to be too disturbed to be considered undisturbed for the geotechnical testing. As a result, undisturbed sampling using a thin-walled Shelby tube sampler and extraction of sediment samples for laboratory testing was not performed as it was deemed to be infeasible. Since undisturbed samples could not be collected, mechanical properties of the sediments were developed based on in situ geotechnical testing (e.g., FFP and CPT tests). The PDI geophysical data, such as data from the subbottom profiler (SBP), were reviewed to develop a preliminary subsurface stratigraphy. However, the entrapped gas in the subsurface materials in gas-charged areas around the lake prevented a proper reflection from the subsurface layers during profiling which resulted in generated cross sections that were transparent. Since the physical properties of the sediments varied significantly with depth and location and geophysical investigation methods were not successful in developing correlations between in situ investigation locations, field investigation programs including a variety of in situ testing and coring options were developed and conducted to characterize the sediments in the various RAs.

This report presents a brief summary of in situ and laboratory geotechnical testing results and the process used to develop the physical and mechanical parameters of the lakebed sediments used in the slope stability analyses of the cap in RA-A through RA-F. This report consists of: (i) an overview of the geotechnical site investigation; (ii) a summary of the physical properties of the lakebed sediments; (iii) a summary of the mechanical (strength and consolidation) parameters of the sediments; and (iv) the method for calculating shear strength gain within the sediments after cap placement. Detailed strength parameters of sediments used for slope stability analyses are documented in the calculation packages for the modified protective cap areas submitted under separate cover.

OVERVIEW OF GEOTECHNICAL SITE INVESTIGATIONS

Parsons has conducted four geotechnical site investigations between 2012 and 2015 to acquire in situ testing data and multiple investigations to collect samples at depths less than 10 ft of the lake-bed sediments. ConeTec performed the in situ testing and Parsons, Anchor QEA and/or ATL collected the core samples.

Samples of Lakebed Sediments

Given the soft to very soft nature of the lakebed sediments, undisturbed samples were unable to be collected. Laboratory testing was primarily conducted to measure the index properties of the sediments from samples obtained using the shallow/gravity core and vibracore methods. The samples collected using shallow/gravity cores or vibracores were considered disturbed and, as a consequence, inappropriate for conducting laboratory testing to obtain strength or compressibility parameters. However, a few laboratory miniature vane tests (MV) were conducted on remolded specimens to measure undrained shear strength as a quality control check to evaluate the shear strength profiles estimated from the FFP and CPT tests.

Four-inch diameter core samples were collected using gravity/shallow and vibracore coring systems. Parsons and Anchor QEA personnel conducted the core processing at a temporary station by splitting the cores in half and visually classifying the sediments. Selected samples of sediment from the cores were preserved either in glass jars or subsections of the core liner and sent to the GeoTesting laboratory.

In Situ Geotechnical Testing

In situ geotechnical testing included CPT, FFP, and PPD tests. In situ VST tests were not performed in the site investigation because other in situ tests (e.g., FFP tests) require fewer corrections to the data (no correction for index properties) and provide a continuous profile of data with depth.

Cone Penetration Test (CPT)

ConeTec used compression type cones with independent tip and friction sleeve load measurements for the CPT testing. This system is equipped with strain gauge load cells for tip and sleeve friction, a diaphragm transducer for pore pressure, a platinum resistive temperature

device, an accelerometer type dual axis inclinometer, and a geophone sensor for seismic signals. The cones used in this project have projected areas of 10 cm^2 and 15 cm^2 with a net area ratio of 0.8 and cone tip apex angle of 60°. Per ASTM D5778, the cone was advanced into the lakebed sediments using a string of 1.0-m length rods at a penetration rate of approximately 2 cm/s.

ConeTec provided the CPT data, which consisted of calculated total tip resistance (q_t), measured cone sleeve friction (f_s), and pore pressure (u_2) with depth. The q_t was calculated using the following correction:

$$q_t = q_c + (1 - a)\, u_2$$

where:

q_c: measured cone tip resistance

a: net area ratio for the cone (0.8 for this investigation)

Full Flow Penetrometer (FFP)

FFP or ball penetration tests were conducted using the CPT thrust system with an attached spherical (ball) tip of 100 cm^2 projected area. The CPT is generally unable to accurately register the penetration values of very soft to soft materials because the surface area of its cone is smaller (approximately 10 times) than that of the FFP tip. Moreover, the full flow around the sediment that typically occurs in FFP tests also minimizes the need for additional correction of tip resistance due to in situ stress. Thus, FFP tests were considered to be a suitable alternative for testing the lakebed sediments. However, CPT tests were advanced at locations where the FFP tests reached refusal at shallow depths (less than 5 feet) to develop a subsurface profile for depths below the FFP refusal. A comparison between data collected from CPT and FFP tests is included in Appendix B.

Cyclic FFP tests were also conducted at selected locations to estimate in situ remolded, undrained shear strengths and sensitivity of the lakebed sediments. The cyclic FFP tests involved performing approximately 5 to 10 cycles of penetration and retrieval over 1 to 1.5-ft intervals at selected depths.

ConeTec provided the uncorrected ball tip resistance (q_b) and pore pressure (u_2) profiles with depth for the FFPs.

Pore-Pressure Dissipation (PPD)

PPD tests were conducted at selected depths using the CPT system (for cones with projected areas of 10 cm^2 and 15 cm^2). The cone was stopped at the selected depths and allowed to record pore pressure data at 5-second intervals for 30 minutes, or until 50% dissipation of the difference between the initial pore pressure and the hydrostatic value was achieved. The dissipation of excess pore water pressure measured with time after advancement of the cone was used to estimate the horizontal coefficient of consolidation.

PHYSICAL PARAMETERS OF SEDIMENTS

Geotechnical Laboratory Testing

The geotechnical laboratory testing program was conducted by GeoTesting to measure the index properties of the soft sediments. GeoTesting laboratory results were provided to Geosyntec by Parsons in tabular and graphical formats via several email communications (GeoTesting projects GTX-303390, GTX-300454, and GTX-12287).

The laboratory testing program consisted of the following:

- Visual classification of sediments per ASTM D2488
- Determination of water (moisture) content per ASTM D2216
- Atterberg limit tests per ASTM D4318
- Particle-size distribution and amount of material finer than 75-µm (No. 200) sieve per ASTM D6913 and/or ASTM D1140
- Determination of density (unit weight) per ASTM D7263
- Miniature vane shear test (remolded) per ASTM D4648

Visual Classification

The core samples were visually classified and documented. Collected samples consist of high plasticity black silts (Unified Soil Classification System designation MH – elastic silt), high plasticity clays (CH – fat clay), and Solvay waste (MH – elastic silt).

Moisture Content

Figure 1 presents the moisture content of lakebed sediments with depth. Moisture content in the lakebed sediments mainly ranges from 40% to 250%. The moisture content data show that lakebed sediments are highly variable, and thus, geotechnical properties of sediments are expected to be highly variable, both spatially and with depth.

Atterberg Limits

Figure 2 also shows a plot of plasticity index (PI) and liquidity index (LI) with depth below the lake-bottom. PI values generally vary from approximately 10% to 81%. LI values range from 0.7 to greater than 3.0, but are typically greater than 1 (i.e., the moisture content is greater than the liquid limit, which is defined as the moisture at which the mechanical response of the soil is similar to that of a liquid). High LI and PI suggest that the lake sediments are highly compressible and sensitive, and can potentially have pronounced time-dependent behavior or creep [Mitchell and Soga, 2005]. Additionally, the high LI value is typical of low strength soils upon remolding. Figure 2 plots the samples tested on the plasticity chart (ASTM D2487). In general, the sediment samples consist of silts and elastic silts (ML and MH, respectively), and low plasticity or lean clays (CL) approaching the A-Line in Figure 2.

Sieve Analyses

Figure 3 presents the percent passing the No. 200 sieve for the collected samples with depth. The lakebed sediments mainly consist of fine-grained particles (silt- and clay-sized). The percentage passing the No. 200 sieve for the sediment samples varies between approximately 44% and 99%.

Organic Content

Figure 3 also presents the organic content of lakebed sediments with depth. The organic content of the sediments is generally low to medium (ISO 14688-2), varying from 1 to 14 % with an average value of approximately 6%. The sediments with decomposed organic matter are usually dark gray or black in color. In general, the unit weight and shear strength of soils decreases and moisture content increases with an increase in the amount of decomposed organic content [Franklin, et al., 1973 and Coutinho and Lacerda, 1987].

Specific Gravity

The specific gravity (G_s) of the sediments with depth is plotted in Figure 4. For the purposes of geotechnical evaluations to support cap design, a G_s = 2.55 was selected for the lakebed sediments, which is lower than typical values for silts and clays [Bowles, 1978]. The lower G_s for the lakebed sediments is likely related to the observed organic content [Coutinho and Lacerda, 1987]. A large variation in G_s (between 2.3 and 2.7) is also observed in Figure 4.

Density

Figure 4 presents the measured and theoretically estimated total unit weight (γ) of lakebed sediments with depth. The measured total unit weight was calculated by dividing the measured weight of sediments by the volume of the cores for which the weight was measured.

The theoretical total unit weight of the lakebed sediments was estimated using the following equation with the assumption that the collected sediment samples were fully saturated:

$$\gamma = \gamma_w \left[\frac{G_s\,(1+w)}{1+w\,G_s}\right]$$

where:

- γ: theoretical total unit weight
- γ_w: unit weight of fresh/lake water (= 62.4 pcf)
- G_s: specific gravity from laboratory test results
- w: water/moisture content (in decimal form)

For the calculation of γ, if G_s measurement for a soil sample was not available, G_s= 2.55 was used. There is a significant variation in the estimated γ (between 75 and 105 pcf), which is likely caused by the highly variable moisture content throughout the lake. A magnitude of γ = 85 pcf was selected by Geosyntec for the lakebed sediments for purposes of slope stability analyses.

Available index test results for the PDI samples and the samples collected from 2012 to 2015 are compared in Appendix C.

STRENGTH AND CONSOLIDATION PARAMETERS OF SEDIMENTS

Strength Properties of Sediments

Undrained Shear Strength

Undrained shear strength (s_u) values were derived from in situ FFP and CPT test data using the following relationships:

For FFP data: $s_u = \frac{q_{b\,net}}{N_t}$

For CPT data: $s_u = \frac{q_{net}}{N_{kt}}$

where:

$q_{b\,net}$: corrected ball tip resistance, $q_{bt} = q_b + [(1-a)\,u_2 - \sigma_v]\,A_s/A_p$
A_s: shaft area
A_p: ball plan area
N_t: empirical bearing capacity factor or ball penetrometer factor;
N_t = 11 was selected for this project, based on published values by Low et al. [2010] and Weemees et al. [2006]
q_{net}: net CPT tip resistance, $q_{net} = q_t - \sigma_v$
N_{kt}: empirical bearing capacity factor or cone factor;
N_{kt} = 15 was selected for this project from the published data by Karlsrud [2004] and Lunne et al. [2002].
σ_v: in situ total vertical stress at the base of the cone
a: net area ratio for the cone;
0.8 for the CPT and FFP in this investigation

Since laboratory measured shear strength of soft sediments using unconsolidated-undrained (UU), consolidated-undrained (CU) triaxial tests, or direct simple shear tests (DSS) was not possible due to the inability to collect undisturbed samples, s_u values were also obtained using miniature vane (MV) tests performed on reconstituted specimens per ASTM D4648. The samples for the MV tests were reconstituted at the as-received moisture content and approximate total unit weight of 85 pcf to represent a normally consolidated condition. Residual s_u values

were measured after ten rapid revolutions of the vane in the reconstituted sediment sample during the MV tests.

Figure 5 presents s_u with depth at locations where both MV and in situ testing data have been obtained. Residual s_u values from both cyclic FFP and MV tests are also included in Figure 5. The relationship $s_u = 0.3 \text{ x } \sigma'_{v0}$ (or shear strength ratio of 0.3; where σ'_{v0} is the effective vertical stress at a given depth) is included in Figure 5 as a graphical reference for the behavior of sediments in a normally consolidated state with shear strength ratio of 0.3. In the original cap design, no strength gain due to the loading from the cap was considered, so the shear strength ratio was not previously reported. The relationship between shear strength gain and the shear strength ratio is explained further in the Shear Strength Gain section.

As shown in Figure 5, the estimated s_u of sediments from FFP tests seem to be lower than that from MV tests at corresponding test locations. Thus, the selected N_t and N_{kt} of 11 and 15, respectively, seem to be reasonable, conservative assumptions for estimating the shear strength of lakebed sediments. As discussed in Appendix B, the shear strength data from the CPT and FFP tests are consistent for the selected cone factors.

Soil Sensitivity

The sensitivity of lakebed sediments (S_t) is calculated from undisturbed and residual s_u values (see Figure 5), using:

$$S_t = s_{u\,undisturbed} / s_{u\,residual}$$

Sensitivity also indicates the potential of the sediment to undergo a reduction in s_u under deformation or creep. Typically, creep (i.e., deformation at constant stress that is lower than the peak strength) in highly sensitive materials due to external loading and secondary consolidation is likely [Mitchell and Soga 2005]. Creep in soils in which the mobilized stress is relatively high (e.g., 80 to 90% of shear strength and even lower in some instances) can cause large deformations that could further reduce the effective shear strength due to strain softening.

Figure 6 shows that the measured range of sensitivity for the lakebed sediments is generally between 1.5 and 4.0, averaging approximately 3. An average sensitivity of 3 suggests that the s_u of the sediments could potentially be reduced to approximately 1/3 of their undisturbed s_u as a result of large deformations or creep.

Compression Parameters

Overconsolidation Ratio

Overconsolidation ratio (OCR) was used to evaluate the stress history of the lakebed sediments. In lieu of laboratory test measurements on undisturbed samples, OCR can be calculated from the CPT data using the following equation:

$$OCR = k\frac{q_{net}}{\sigma'_v}$$

where

k: overconsolidation ratio constant; a k value of 0.33 (for intact clay) was selected and used in this project [Mayne 2007], since no high-quality undisturbed samples could be collected to run one-dimensional oedometer tests.

The above OCR expression is related to q_{net}, which is calculated for the CPTs, but not the FFPs. q_{net} can be approximated by using the $q_{b\,net}$ from the FFPs, using the following relationship (developed by equating the estimations of s_u from FFP and CPT):

$$q_{net} = q_{b\,net} \text{ x } (N_t/N_{kt})$$

where values of N_t = 11 and N_{kt} = 15 were used.

Pre-overburden pressure (POP) was used to evaluate the additional loads (from capping) needed for the lakebed sediments to reach a normally consolidated state. The POP was calculated using:

$$POP = \sigma'_p - \sigma'_v$$

where σ'_p is preconsolidation pressure.

The overconsolidation condition of the lakebed sediments has been estimated from the FFP and CPT data. Figure 7 presents plots of OCR, POP, and s_u from example FFP data. As an example, at location OL-FFP-CPTu-19, the OCR is estimated to be approximately 1.0, indicating that the approximately top 7 ft of sediments are in a normally consolidated state. However, at

location FFP-15-B11, the OCR is estimated to be greater than 1, indicating the sediments are overconsolidated (i.e., they have experienced a higher overburden stress in the past than currently exists). Figure 7 also shows the POP for the sediments at OL-FFP-CPTu-19 and FFP-15-B11. The POP values are approximately zero for the upper sediments at OL-FFP-CPTu-19 and about 100 psf in the top 3 ft of sediments at location FFP-15-B11. This suggests that at FFP-15-B11, where sediments seemed to be overconsolidated, placement of approximately 1.7 ft of cap material would cause sediments to reach a normally consolidated state; therefore, it does not cause significant consolidation and limited strength gain due to capping can be considered for sediments at this location. The cause of the oversonsolidation of the sediments could be related to chemical and/or physical processes, such as chemical bonding between sediment particles, removal of shallow sediments, etc.

Undrained Shear Strength Ratio

Undrained shear strength of normally-consolidated materials (such as clay, silt, and sediments) may increase proportionally with an increase in effective vertical stress (σ'_v), termed the undrained shear strength ratio (S). To properly estimate S for the calculation of strength gain, lakebed sediments were classified as sediments in normally consolidated and overconsolidated states.

For sediments in normally consolidated state (example location OL-FFP-CPTu-19 in Figure 7), the undrained shear strength of the sediments can be estimated using:

$$s_u = S \text{ x } \sigma'_v$$

$S = 0.3$ (selected from the estimated S for normally consolidated clay from the PI of approximately 50 as shown in Figure 7 [Skempton and Henkel, 1953]) for the sediments at the example location OL-FFP-CPTu-19.

For sediments in overconsolidated state (example location FFP-15-B11 in Figure 7), the measured S are higher ($S = 0.5$ and 0.7 are shown in Figure 7). The higher S of sediments at the example location FFP-15-B11 may be attributed to the overconsolidated state of the sediments, as described by Ladd et al. [1977]. The following equation is used to estimate S_{oc} for overconsolidated sediments:

$$S_{oc} = \frac{s_u}{\sigma'_v} = S \text{ x } OCR^{0.8}$$

Assuming an OCR of 3 and an S of 0.3 for normally consolidated sediments for the top few feet of FFP-15-B11, the calculated S for the overconsolidated sediments at this location is 0.7, which is consistent with the in situ S measured at FFP-15-B11. For sediments in normally consolidated state, $S_{oc} = S$

Coefficient of Consolidation

Figure 8 shows the coefficient of consolidation (C_{vh}) with depth measured from the pore water pressure dissipation tests, which uses the following equation [Houlsby and Teh, 1988]:

$$c_h = \frac{T_{50}\, a_c^2\, \sqrt{I_r}}{t_{50}}$$

where

- T_{50}: dimensionless factor, which is 0.245 for 50% dissipation [Houlsby and Teh, 1988]
- a_c: cone radius, which is 1.78 cm for a 10 cm^2 cone, and 2.18 cm for a 15 cm^2 cone
- I_r: rigidity index which is equal to the ratio of the shear modulus (G) and s_u; per equation presented in Mayne (2007).
- t_{50}: time at 50% of pore pressure dissipation, obtained from pore water pressure dissipation data

Over 80% of the c_h values interpreted from the pore water pressure dissipation tests for the sediments are between 1 x 10^{-3} cm^2/s and 1 x 10^{-1} cm^2/s. The typical published ranges of c_h values is 7 x 10^{-5} cm^2/s (Holtz and Broms, 1972) to 4 x 10^{-3} cm^2/s (Ladd and Luscher, 1965) for clays, and 0.9 x 10^{-5} cm^2/s to 1.5 x 10^{-4} cm^2/s (Leonards and Girault, 1961) for high plasticity silts (MH). The measured c_h in general are higher than the published values, indicating that the lakebed sediments tend to consolidate in faster rate than clays and silts published in literature. A c_h value of 4 x 10^{-3} cm^2/s was selected as a reasonable lower estimate (in top 6 ft of lakebed sediments). In the original design of the cap, the impact of sediment consolidation was not considered in the slope stability analyses, so the c_v was not reported. The c_v used for evaluating

the degree of consolidation and shear strength gain due to capping was assumed to be equal to c_h. Typically the value of c_v could be an order of magnitude smaller than c_h.

SHEAR STRENGTH GAIN

Pore Water Pressure Dissipation

Figure 9 presents calculated pore water pressure dissipation with time for the lakebed sediments. The sediments are assumed to have a one-way drainage path toward the top (lakebed) since the sediments are covered by the coarser-grained cap and underlain by fine-grained materials, which likely inhibit downward drainage. Figure 9 shows that the degree of consolidation, U_c (i.e., equal to *100% – ΔU*, where *ΔU* is the ratio of the excess pore water pressure at time t and the initial excess pore water pressure expressed in percent. In turn, the initial excess pore water pressure is equal to the submerged weight of the cap assuming one-dimensional loading conditions), is greater in the upper portion of the sediments, but decreases significantly with depth and increased drainage distance.

Shear Strength Gain

The shear strength gain caused by the increase in vertical effective stress (in this case, as a result of cap placement) is proportional to the degree of consolidation. The shear strength gain, Δs_u, can be calculated for normally consolidated soils using the equation below, where undrained shear strength ratio (S) and the increase in vertical effective stress ($\Delta\sigma'_v$), are known:

$$\Delta s_u = \Delta\sigma'_v \text{ x } S$$

The increase in vertical effective stress is calculated by:

$$\Delta\sigma'_v = (\gamma_{sc} - \gamma_w) \times t_c \times U_c$$

where

γ_{sc}: saturated unit weight of cap (assumed to be 120 pcf)
γ_w: unit weight of water
t_c: cap lift thickness
U_c: degree of consolidation for the cap lift

S was selected to be 0.3 based on the results of in situ FFP tests for normally consolidated sediments, as discussed above. It is noted that the increase in undrained shear strength of sediments in a normally consolidated state due to each lift of cap material should be calculated independently. For example, if the first lift of cap is placed at time = 0 and the second lift of cap is placed at time = 7 days, the strength gain of sediments at time = 14 days is the sum of strength gain from the first lift during the 14 days and that from the second lift during the last 7 days. Figure 10 illustrates the concept of strength gain as a result of placing lifts of cap at different times.

The shear strength gain due to sediment consolidation described above applies to normally consolidated sediments. For overconsolidated sediments, the cap placed must result in an $\Delta\sigma'_v$ that is equal to or greater than the POP of the sediments before shear strength gain is considered (i.e., sediments become normally consolidated after dissipation of the excess pore water pressure caused by the cap).

In general, the strength gain of sediments deeper than 6 ft below the lakebed was neglected due to the relatively long drainage path and negligible degree of consolidation that would occur within the few weeks of waiting periods between the placements of cap.

The selected geotechnical parameters of sediments (including S and c_h) and methodology for calculating the strength gain of sediments under the cap loading were validated by comparing pre-capping and post-capping s_u profiles with depth obtained at two locations in RA-E. Figure 11 presents the measured pre- and post-capping s_u profiles at these two locations. The predicted s_u due to strength gain based on the as-built construction data provided Parsons are also shown in Figure 11. The upper six feet of sediments experienced strength gain due to consolidation after the cap lifts were placed. The predicted s_u profile is less than the measured s_u, thus the selection of shear strength ratio, coefficient of consolidation, and methodology for estimating the strength gain in sediments appeared to be appropriate for slope stability analysis.

VARIABILITY

This report noted the variation in the geotechnical properties and parameters of the lakebed sediments, spatially over the area of interest and with depth; therefore, a simplified subsurface geotechnical and geological model could not be developed for all remediation areas around the lake. Because of these variations, several sets of in situ and laboratory tests were performed

from 2012 to 2015. However, the behavior of the sediments could not be systematically evaluated because obtaining undisturbed samples was unsuccessful and advanced geotechnical laboratory testing could not be performed. In general, the selected parameters and methodology in this report tends to capture the variability in geotechnical parameters.

CONCLUSIONS

This report presented a summary of the process used to estimate geotechnical design parameters for the Onondaga lakebed sediments based on results of in situ investigations and laboratory testing. In general, undisturbed sampling of the lakebed sediments was not considered feasible. Therefore, mechanical properties of the sediments were developed based on in situ geotechnical testing, such as the FFP and CPT tests and strength and stress history estimates using correlations without site-specific verification.

Based on the laboratory index and in situ test results, lakebed sediments are highly variable, regionally and with depth. Lakebed sediments exhibit variable stress history with some locations showing normally consolidated conditions and other locations showing overconsolidated conditions. Therefore, the sediments were assigned specific strength profiles for each cross section used in the slope stability analyses based on data from in situ tests performed in the vicinity of the cross sections, as presented in separate slope stability calculation submittals for modified protective cap areas.

The normalized undrained shear strength ratio was assumed to be 0.3 for sediments in a normally consolidated state. For sediments in an overconsolidated state, the cap placed must result in an increase in effective vertical stress ($\Delta\sigma'_v$) that is greater than the pre-overburden pressure (POP) of the sediments before shear strength gain is considered (i.e., sediments reach a normally consolidated state).

Based on the calculated c_h from the in situ test results and published values in the literature, a value of c_h= 4 x 10^{-3} cm^2/s was selected to estimate the time for consolidation under loading from the cap placement.

After comparing the measured and predicted undrained shear strength gain of sediments due to cap placement, the selection of shear strength ratio, coefficient of consolidation, and methodology for estimating the strength gain appeared to be appropriate for slope stability analysis.

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Table 1 - Summary of Geotechnical Properties of Sediments in Onondaga Lake Capping Areas

Total Unit Weight [pcf]	**Shear Strength Ratio in Normally Consolidated State**	**Undrained Shear Strength, s_u**	**Sediments in Normally or Over-consolidated State?**	**Coefficient of Consolidation, C_{vh} (cm^2/s)**
85	0.3	Presented in Calculation Packages for Each Remediation Area	Based on Estimated OCR and POP	4×10^{-3}

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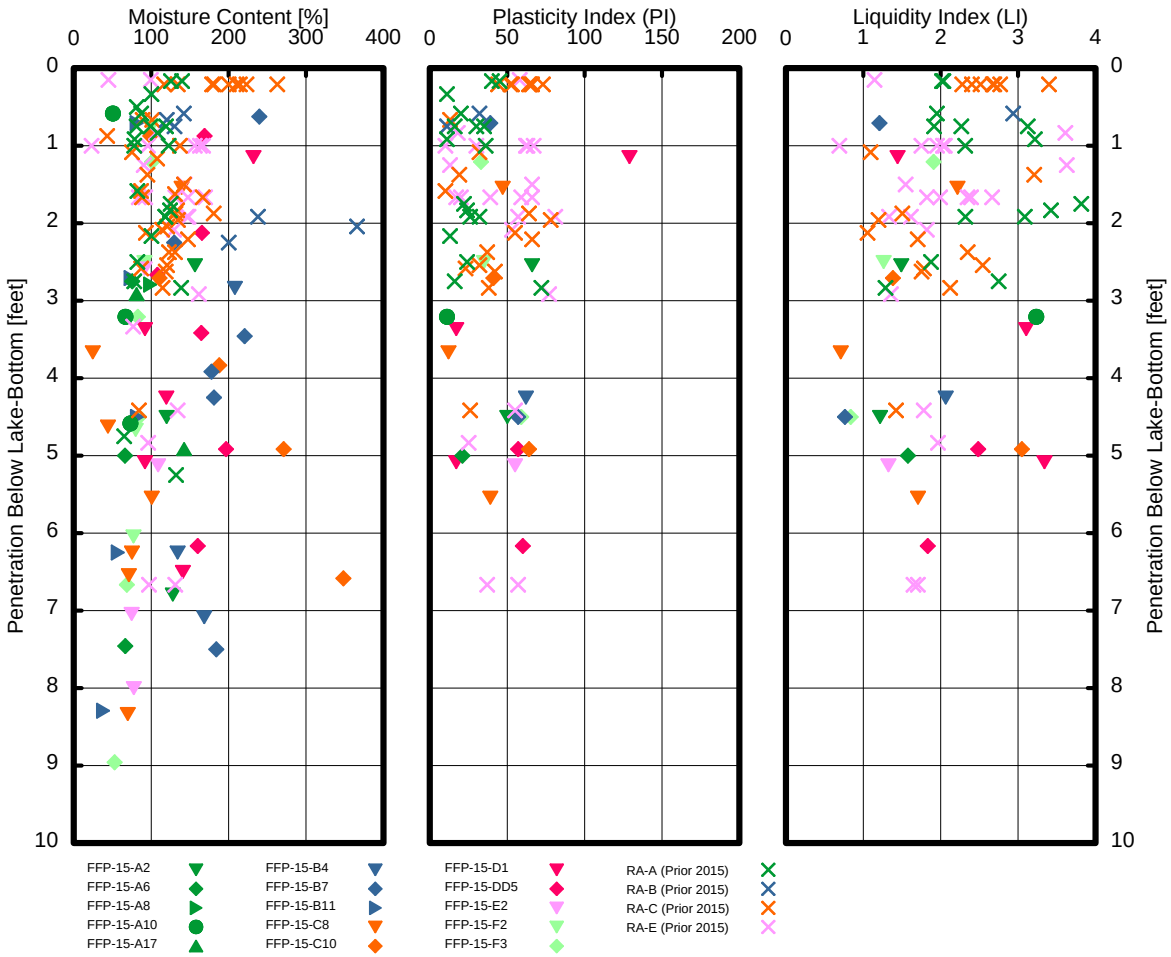

Figure 1 – Moisture Content, Plasticity Index, and Liquidity Index of Lakebed Sediment in RA-A through RA-E

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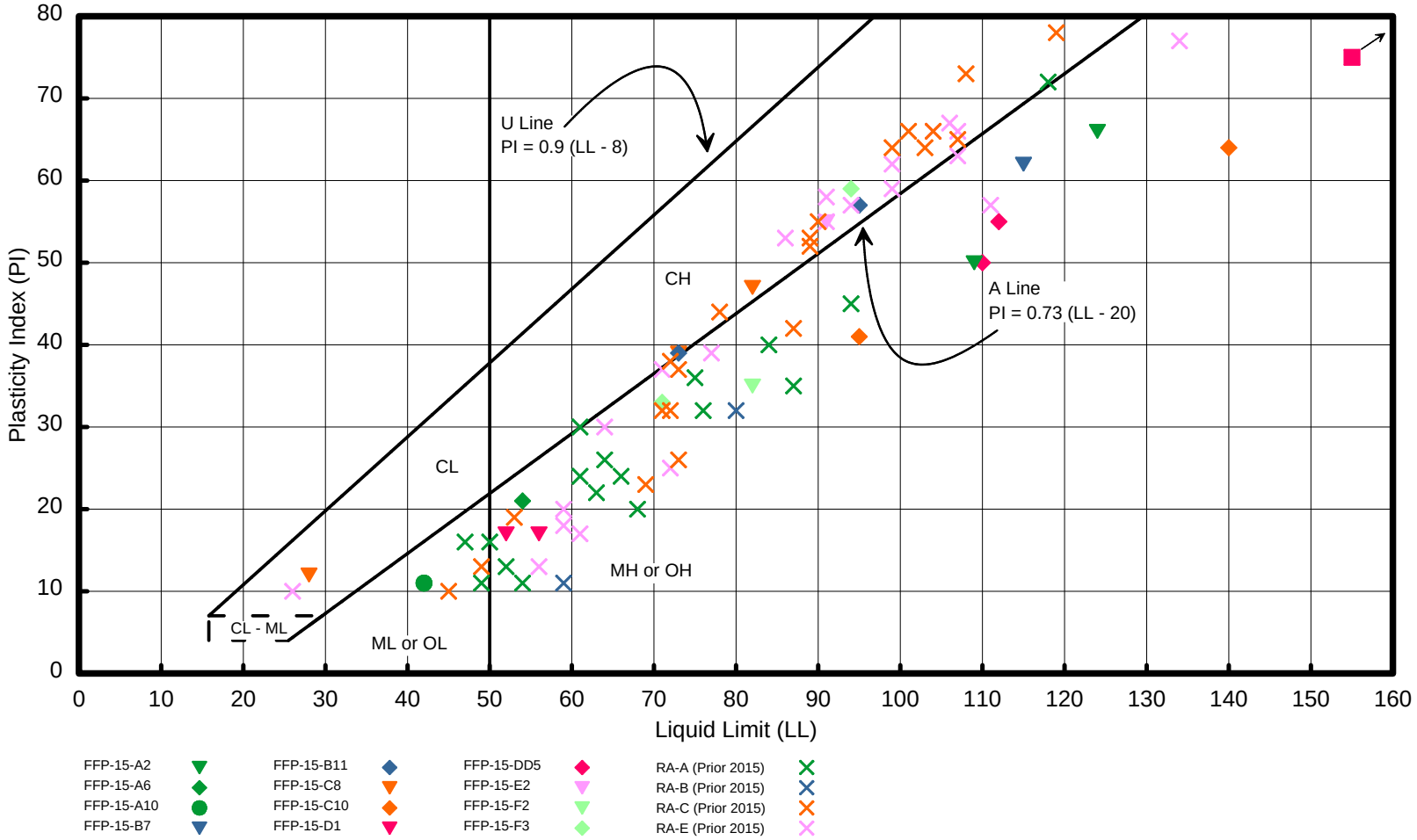

Figure 2 – Plasticity Chart of Lakebed Sediments in RA-A through RA-E

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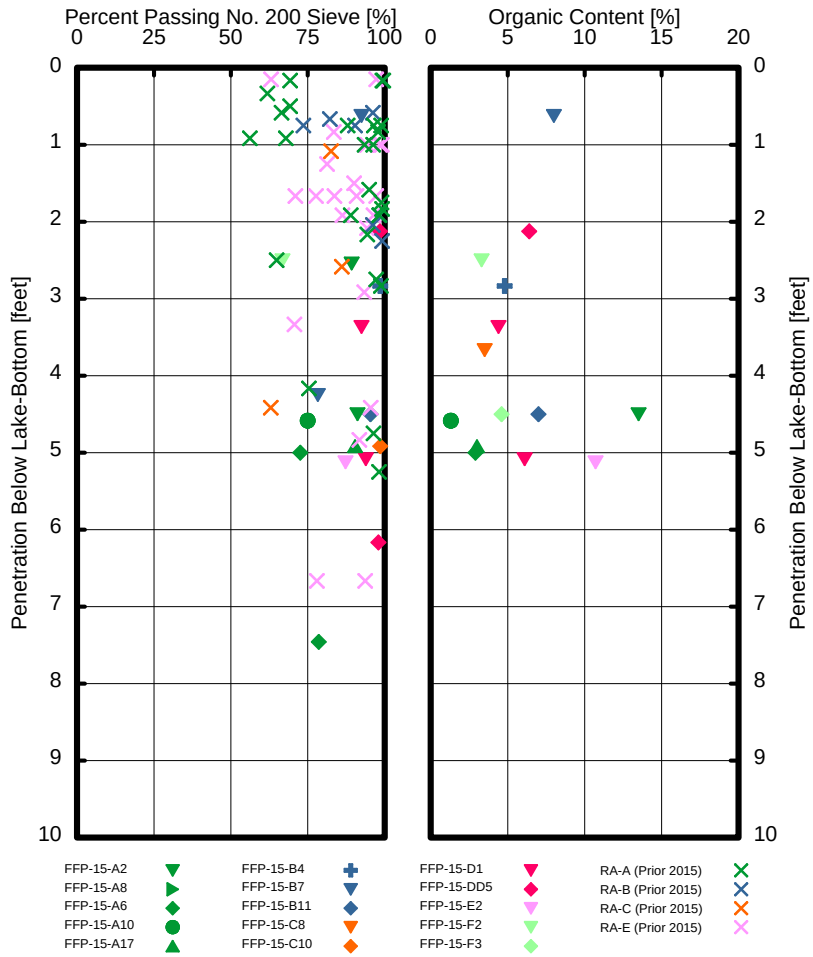

Figure 3 – Percent Passing No. 200 Sieve and Organic Content of Sediments with Depth

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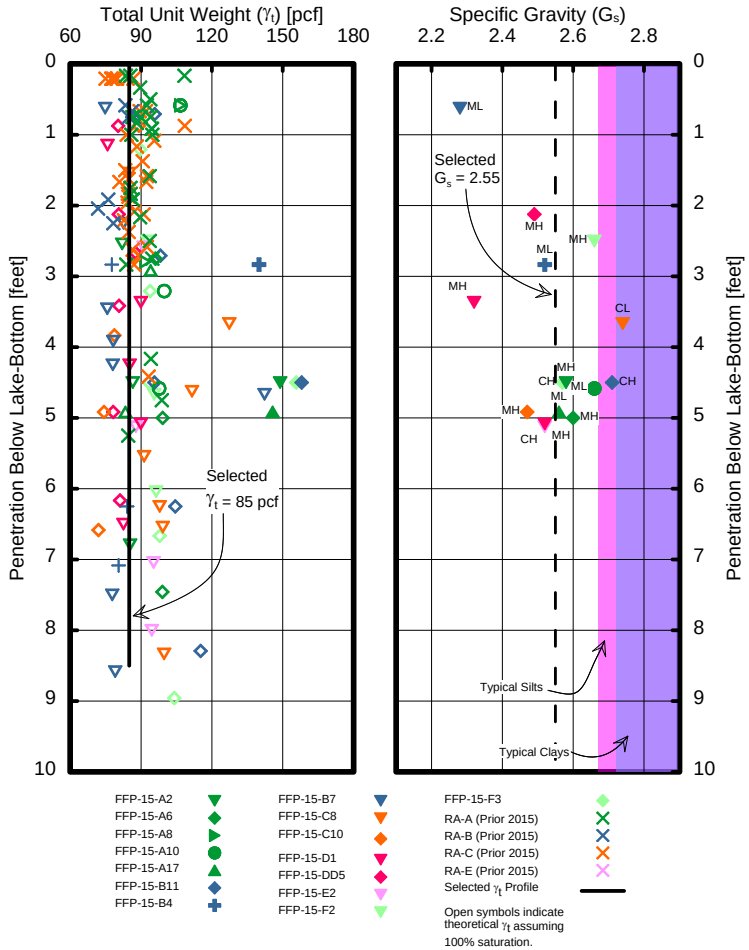

Figure 4 – Total Unit Weight and Specific Gravity of Sediments with Depth

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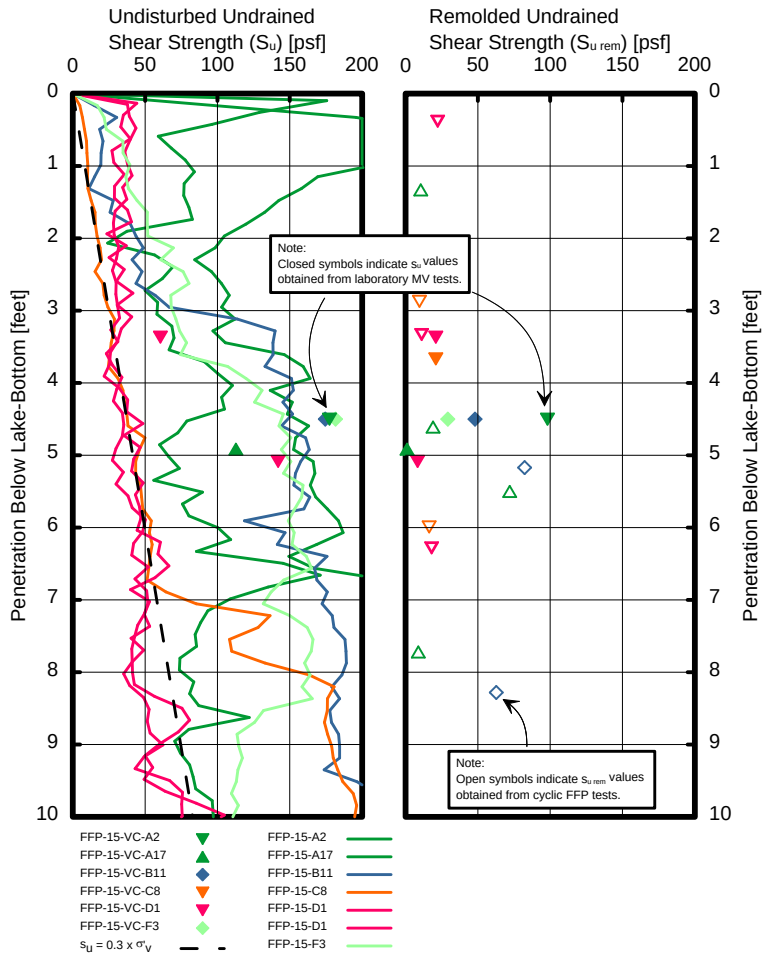

Figure 5 – Undisturbed and Residual Undrained Shear Strength of Sediments with Depth

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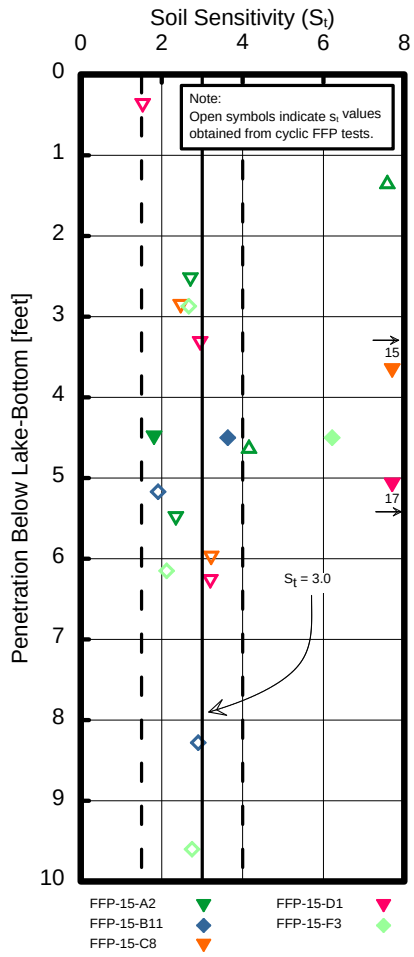

Figure 6 – Sensitivity of Sediments with Depth

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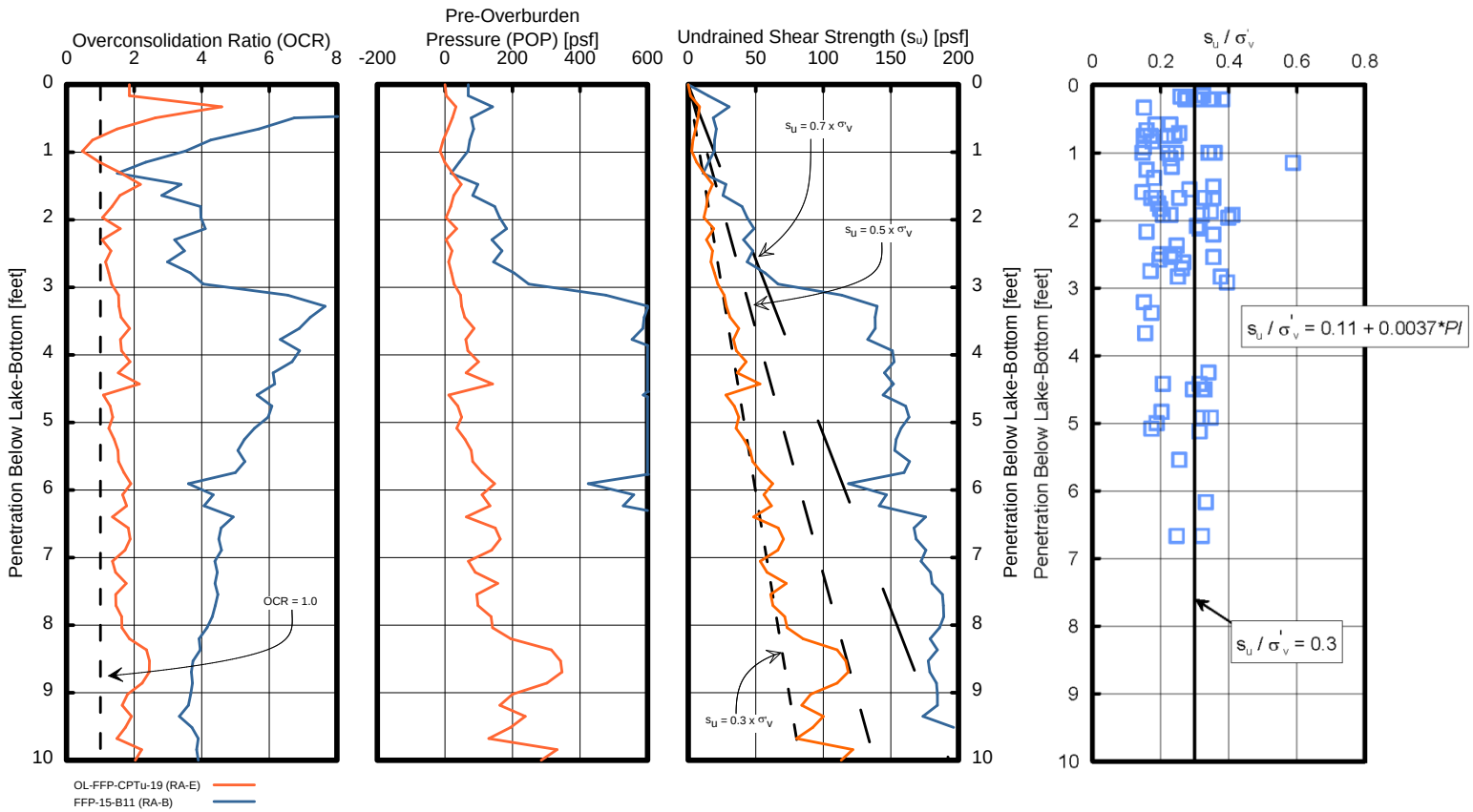

Figure 7 –Overconsolidation Ratio, Pre-overburden Pressure, and Undrained Shear Strength of Measurements and Estimated s_u Ratio for Normally Consolidated Clay from Index Test Results (equation from Skempton and Henkel [1953].

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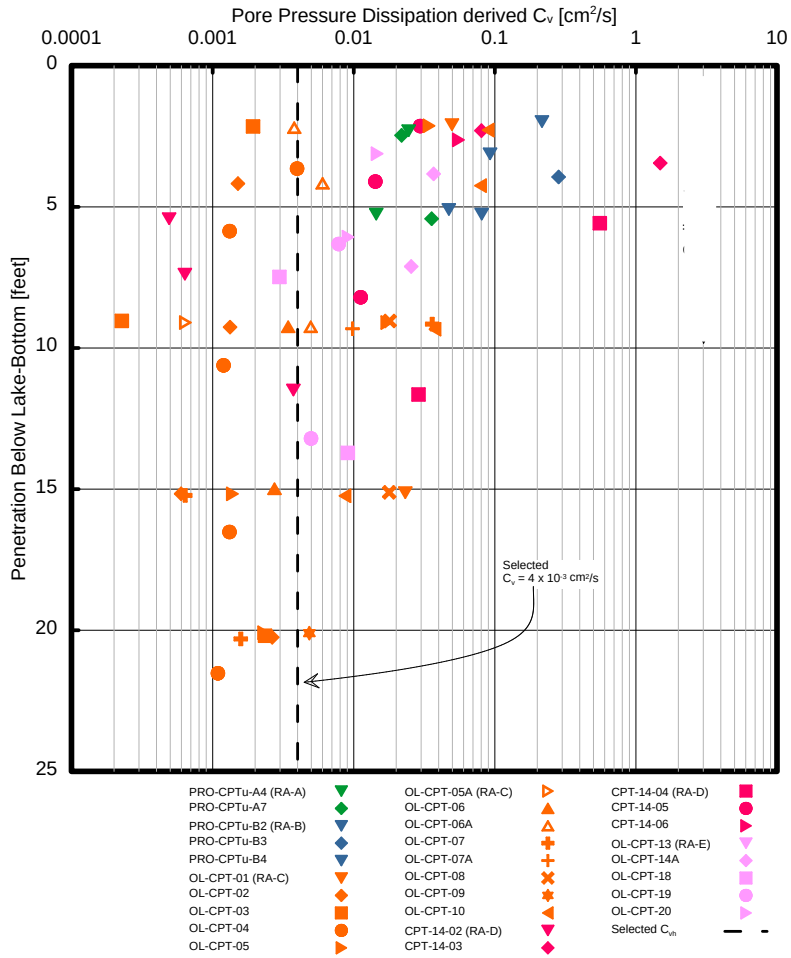

Figure 8 – Coefficient of Consolidation

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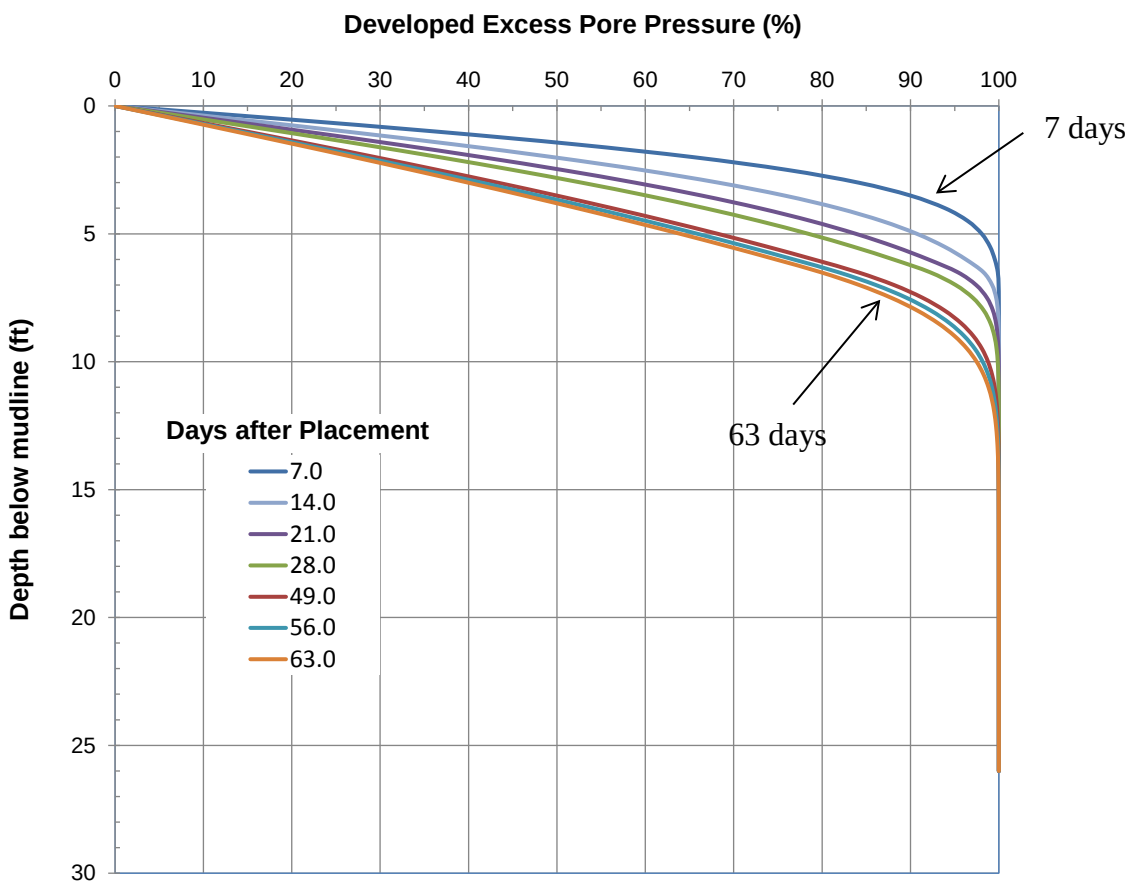

Figure 9 – Pore-Water Pressure Dissipation with Time for Soft Sediments

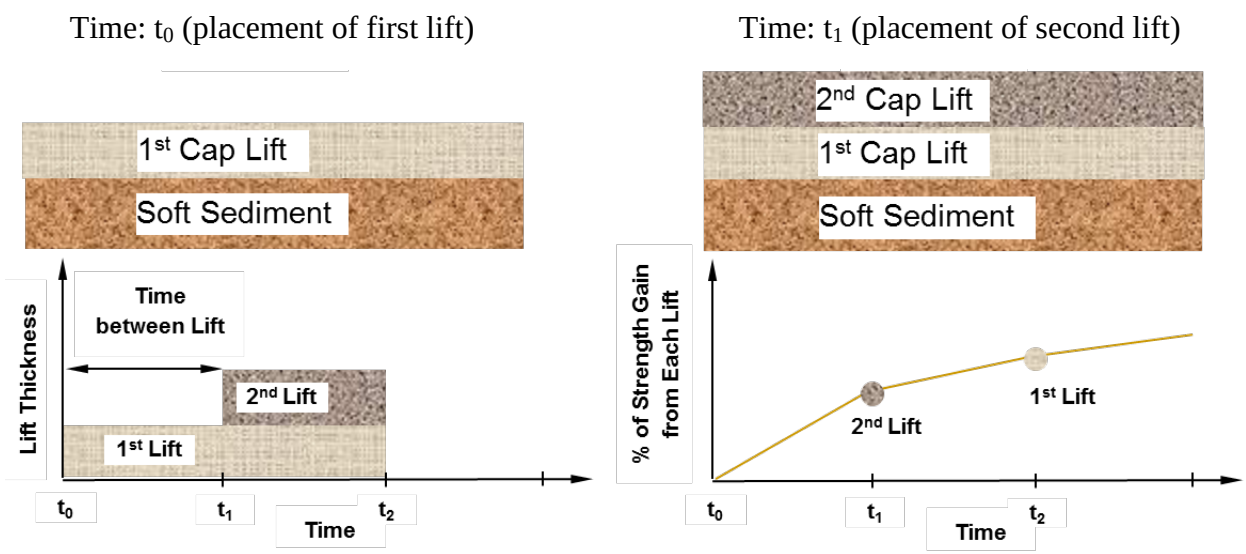

Figure 10 – Concept of Shear Strength Gain for Soft Sediments under Different Lifts of Cap

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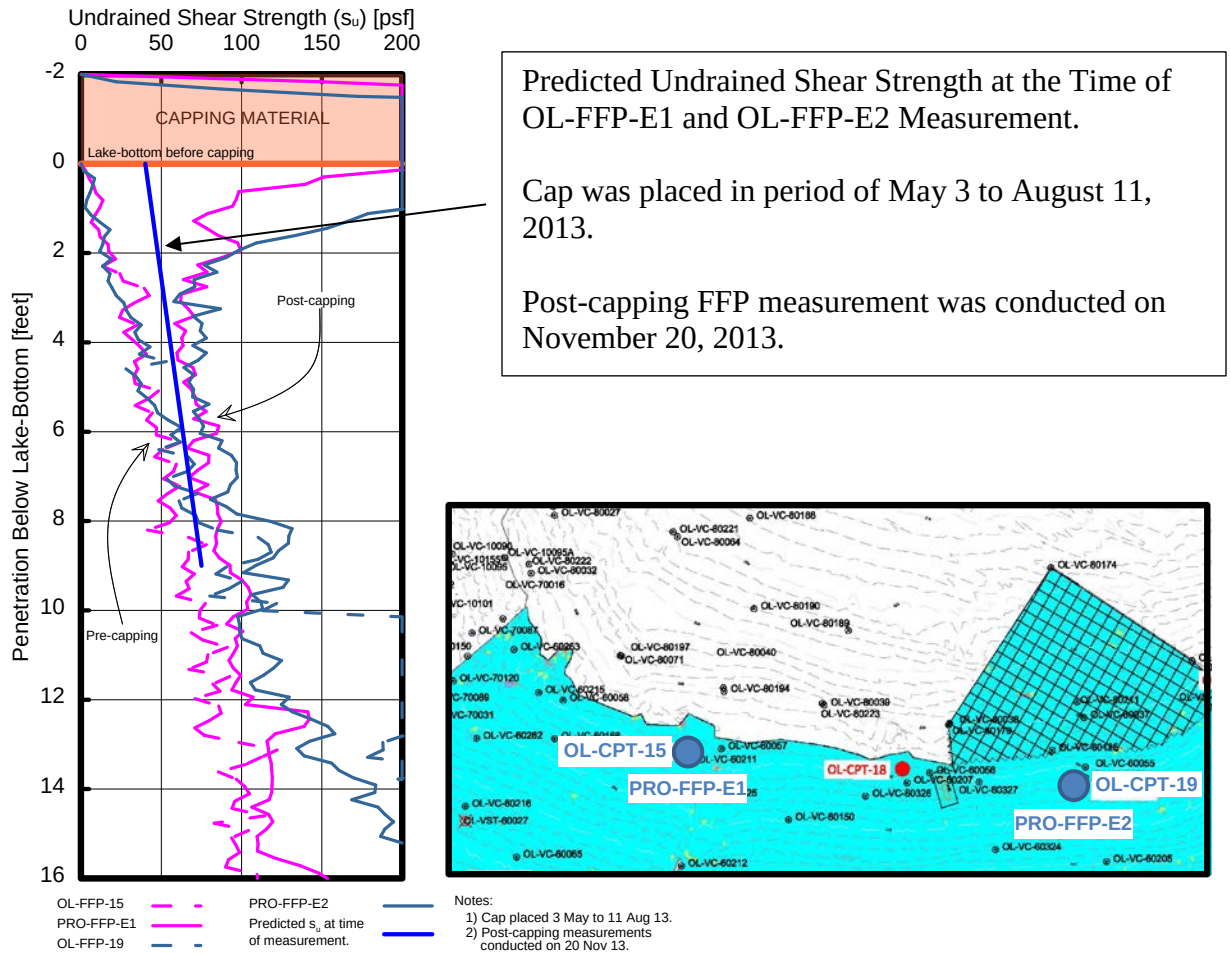

Predicted Undrained Shear Strength at the Time of OL-FFP-E1 and OL-FFP-E2 Measurement.

Cap was placed in period of May 3 to August 11, 2013.

Post-capping FFP measurement was conducted on November 20, 2013.

Figure 11 – Comparison of Pre and Post Capping Measured s_u (from FFP) and Predicted Strength Gain in Lakebed Sediments

APPENDIX A

Comparison between Undrained Shear Strength Collected during PDI and Post PDI

The evaluation of strength data collected between 2012 and 2015 for lakebed sediments in remediation areas A through F (RA-A through RA-F) indicate that the sediments in portions of these remediation areas are significantly softer than anticipated based on pre-design investigations (PDI) conducted prior to 2012. The PDI strength assumptions (developed from in situ vane shear tests (VST)), were generally representative of the subsurface materials in water depths less than about 20 feet. VST data could not be collected in water depths greater than 20 feet due to depth limitations of the equipment. A comparison between the estimated strength parameters from the field VST data (from the PDI) and the post-PDI laboratory and in situ VST data conducted after 2012 indicates that, in general, the PDI data showed higher shear strengths for the shallow sediments than the recent data.

Statistical analyses were performed on the shear strength measurements of the lakebed sediments from the PDI (Figure A.1). The lower fifth percentile of all shear strength measurements (i.e., 26 psf) was used in the Final Design report as a lake-wide value for bearing capacity calculations and maximum cap lift thickness recommendations discussed in Section 4.5.4 of the Final Design Report.

Figure A.1 presents a cumulative frequency of all in situ and laboratory VST measurements from the PDI broken into two depth intervals; 0 to 1 foot and deeper than 1 foot. Some of the measurements may have been taken in sediments with a coarse-grained fraction, as determined in the field during vane rotation and/or by visual inspection of the sediment that adhered to the vane after retrieval. Since the VST is not applicable to purely coarse-grained sediments, those measurements in suspected coarse-grained areas were identified as such.

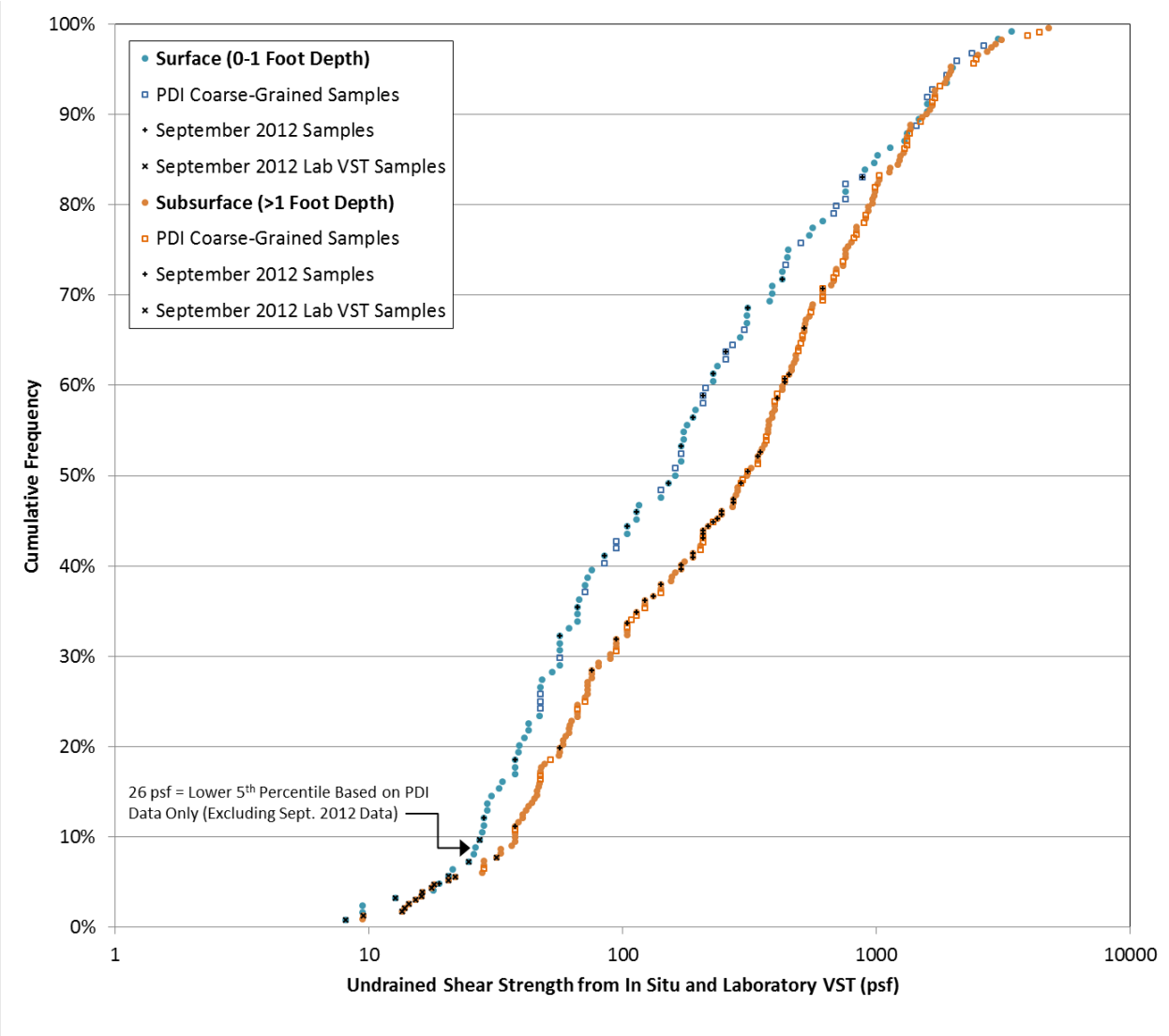

Figure A.1 Comparison between PDI and Post PDI (September 2012) Undrained Shear Strength of Sediments

The majority of the measurements of shear strength of surface sediments in and adjacent to the disturbed areas collected after the first movement in RA-C was observed (i.e., September 2012 Samples in Figure A.1), which were typical of much of the surface sediment strength data collected in areas where softer sediments were identified, were generally below the lower fifth percentile strength selected for the original design.

APPENDIX B

Comparison between Undrained Shear Strength Estimated from FFP and CPT

The CPT data were compared with the FFP data where the results of these two tests were available in close proximity. An example comparison between the calculated shear strength from the two FFP tests and one CPT test collocated in RA-B is shown below. The cone factors (N_{kt} of 15 for CPT and N_t of 11 for FFP) used in this document resulted in similar shear strength profiles.

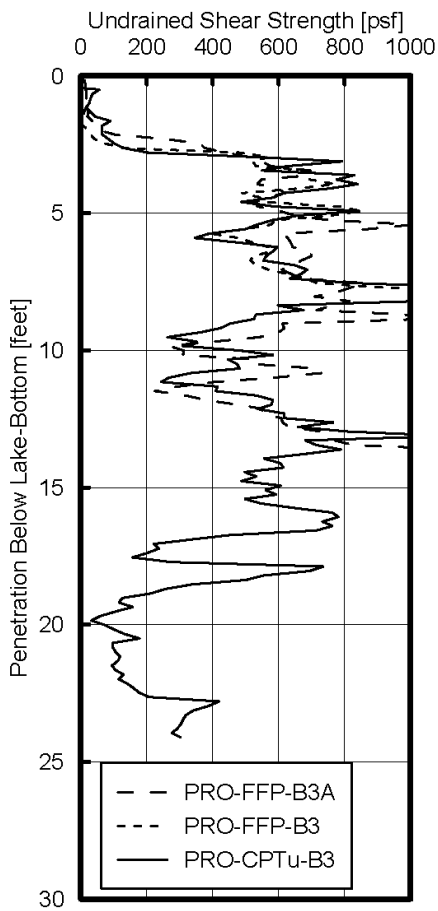

Figure B.1 Example for Comparison between Undrained Shear Strength Estimated from FFP and CPT

APPENDIX C

Comparison between PDI and Post PDI Index Properties

The comparison between the index test results for the PDI and the recently collected samples for the lakebed sediments are shown in Figure C.1. The 2012 to 2015 data show the sediments have higher LL and PI compared to the PDI data.

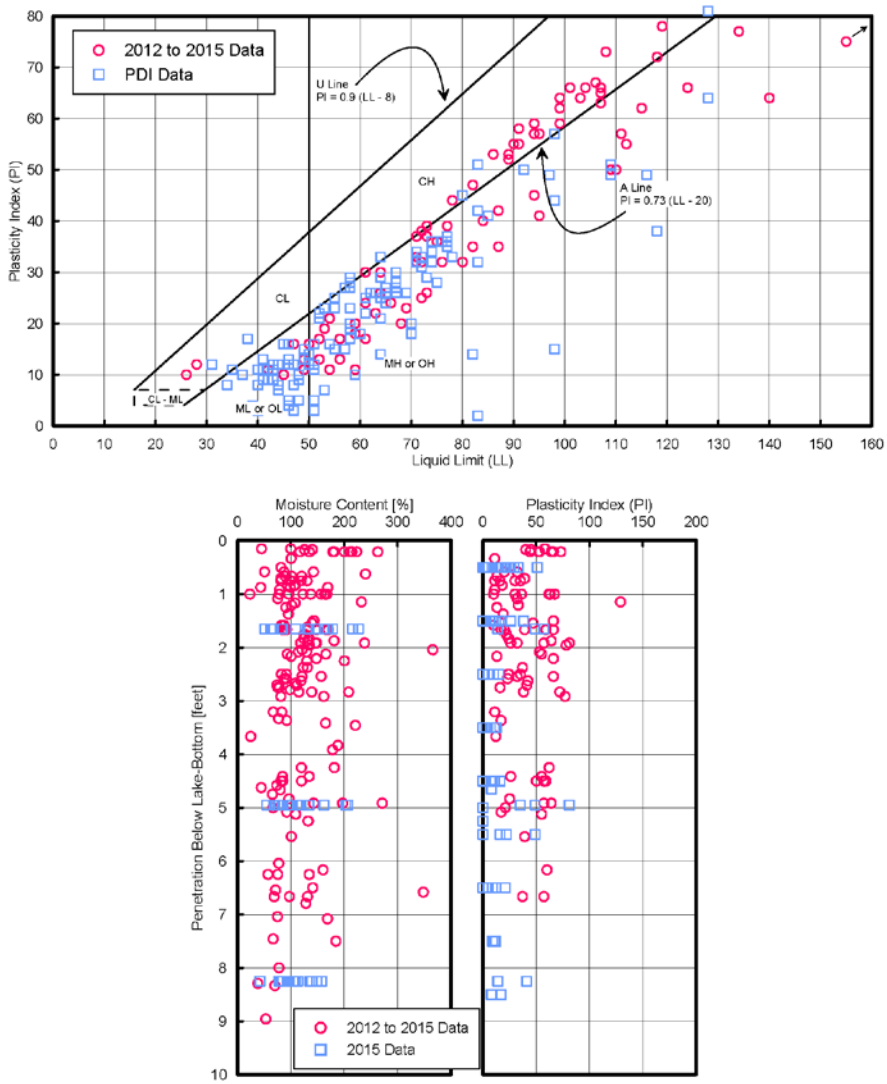

Figure C.1 Comparison between PDI and Post PDI (2012-2015) Index Properties of Sediments