DRAFT DATA REPORT
FIELD AND LABORATORY DATA COLLECTION PHASE
COLE'S BAYOU MARSH RESTORATION PROJECT (TV-63)

VERMILLION PARISH, LOUISIANA

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Coastal Protection and Restoration Authority
P.O. Box 44027
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Attention: Ms. Vida Carver, P.E.
Vida.Carver@la.gov

Re: Draft Data Report - Field and Laboratory Data Collection Phase
Cole's Bayou Marsh Restoration Project (TV-63)
Vermillion Parish, Louisiana

We have completed the field exploration and laboratory data collection phase of the Cole's Bayou Marsh Restoration Project (TV-63). A summary of the field exploration and laboratory testing results, along with our evaluation of the data and discussion of geotechnical design property selection, are provided in the attached Draft Data Report. This work was authorized by acceptance of our work plan, AAI File No. 113-13-80-3705PR, dated February 18, 2013.

We will be pleased to discuss any questions you may have concerning this data report.

Sincerely,
ARDAMAN & ASSOCIATES, INC.

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TABLE OF CONTENTS

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VERMILLION PARISH, LOUISIANA

Results and findings of the field exploration and laboratory testing phases of the Cole's Bayou Marsh Restoration project are provided herein. Boring locations, boring logs and generalized subsurface profiles, along with a description of terms and symbols used on the boring logs are provided in Appendix A. Laboratory testing data plots are included in Appendices B, C, and D.

SECTION 1. GENERAL PROJECT INFORMATION

1.1 Project Description

The Cole's Bayou Marsh Restoration project will consist of the creation of approximately 365 acres of brackish marsh, the nourishment of approximately 53 acres of existing brackish marsh, and the increase of freshwater and sediment inflow into the project area. This will yield approximately a net 398 acres of new brackish marsh area over the intended 20-year design life of the project. This will be achieved by hydraulically dredging material from nearby Little Vermillion bay and pumping it into designated fill areas that will be bounded by earthen dikes. In addition, the Coastal Protection and Restoration Authority (CPRA) requested that AAI perform two additional borings for a proposed rock wall adjacent to the freshwater bayou. The scope of work associated with the field and laboratory data collection phase of this project consisted of performing a total of 26 soil borings (B-01 through B-26) to depths ranging from 20 to 80 feet below the existing mudline at locations established by the CPRA.

1.2 Site Location and Description

The project site is located approximately 11 miles southeast of the Town of Cow Island in Vermillion Parish, near Little Vermillion Bay east of White Lake. The site is a wetland area with the majority of its surroundings consisting of marsh. The water depth at the boring locations during the field exploration phase ranged from 0.4 to 5.0 feet, with an average water depth of approximately 1.92 feet. The site is bordered on all sides by open water and marsh areas. The marsh area is contained by Louisiana Highway 333 to the north and by Louisiana Highway 82 to the west. Morgan Shell Landing on the north side of the site was used for mobilizing and launching our airboat-mounted drilling equipment.

1.3 Geology

Geologically, the site is underlain by the Chenier Plain Saline Marsh Deposits of the Holocene Age. These deposits consist of gray to brown to black clay and silt of moderate organic content.

SECTION 2. FIELD EXPLORATION

2.1 Permission and Access

Prior to mobilizing to the site to establish the boring locations or conduct site reconnaissance, landowners, as listed in the *Scope of Services for Geotechnical Investigations, Cole's Bayou Marsh Restoration Project, TV-63* document, were notified in the form of a letter from Ardaman & Associates, Inc. (AAI) to inform the parties involved of our intended geotechnical investigation operations. Through communication with the landowners and Ms. Vida Carver of CPRA, the project engineer for the project, permission was granted to access the site and conduct our geotechnical investigation at all of the boring locations.

2.2 Soil Borings

A total of 26 borings were performed at locations designated by CPRA, as shown on Figure A-1, Boring Location Plan, in Appendix A. Boring locations B-01, B-02, and B-03 were performed within the proposed Marsh Creation Area 1. Boring locations B-04, B-08, and B-11 were performed within the proposed Marsh Creation Area 2. Boring locations B-13, B-14, B-15, and B-16 were performed within the proposed Marsh Creation Area 3. Boring locations B-05, B-06, B-07, B-09, B-10, B-12, B-17, and B-18 were performed at the locations of the proposed culvert structures. Boring locations B-19 through B-24 were performed within the borrow area of Little Vermillion Bay. The remaining two boring locations B-25 and B-26 are appropriated for the proposed rock wall. The 26 soil borings were performed between October 8 and October 26, 2013. As-drilled boring locations were determined in terms of Global Positioning System (GPS) UTM coordinates recorded at each boring location using a hand-held GPS device. The GPS coordinates at each boring location are presented in Table 2.1 below. This information is also presented on the soil boring logs in Appendix A. The water depth at each boring location was measured at the time of drilling and is presented in Table 2.1 below.

All borings were performed using an airboat-mounted, rotary-type drilling rig. Borings were advanced using 4-inch diameter rotary wash methods to depths ranging from 20 to 80 feet below the existing mudline. Discrete samples were obtained continuously within the upper 20 feet at all of the boring locations. Continuous sampling was performed to provide detailed information for near surface stratigraphy. Below the 20-foot depth, the samples were obtained at five-foot sampling intervals. The boreholes were grouted upon completion in accordance with State regulations.

In the cohesive and semi-cohesive soils, relatively undisturbed samples were secured using a 3-inch diameter, 30-inch long, thin-walled Shelby tube. In this sampling procedure, the borehole is advanced to the desired level, and the Shelby tube is lowered to the bottom of the boring. It is then pushed 24 inches into the soil in one continuous stroke.

Upon retrieval, the sample at the end of the tube was visually classified and then the sample was sealed in the tube with plastic caps and expandable disk-type seals. Each sample tube was labeled and placed vertically in a fabricated tube rack to minimize any disturbance to the sample during transport. All samples were transported to our Baton Rouge laboratory for extrusion and testing.

Table 2.1 Soil Boring Details

Boring ID	Depth (ft)	GPS Coordinates		Water Depth at Borehole (ft) †	Project Area *
		Latitude	Longitude		
B-01	30	29°44'21.22"N	92°13'36.23"W	≈ 0	MCA 1
B-02	80	29°44'18.04"N	92°13'31.47"W	≈ 0	MCA 1
B-03	30	29°44'22.63"N	92°13'23.19"W	≈ 0	MCA 1
B-04	30	29°44'5.48"N	92°13'53.45"W	≈ 0	MCA 2
B-05	60	29°43'59.20"N	92°13'44.12"W	≈ 0	Culvert
B-06	60	29°43'58.11"N	92°13'34.78"W	≈ 0	Culvert
B-07	60	29°43'56.67"N	92°13'35.31"W	≈ 0	Culvert
B-08	80	29°43'54.92"N	92°13'59.74"W	≈ 0	MCA 2
B-09	60	29°43'55.42"N	92°12'39.77"W	≈ 0	Culvert
B-10	60	29°43'46.86"N	92°13'1.19"W	≈ 0	Culvert
B-11	30	29°43'41.75"N	92°13'51.92"W	≈ 0	MCA 2
B-12	60	29°43'28.13"N	92°13'14.24"W	≈ 0	Culvert
B-13	30	29°43'0.26"N	92°13'3.62"W	≈ 0	MCA 3
B-14	30	29°42'55.76"N	92°12'52.06"W	≈ 0	MCA 3
B-15	80	29°42'53.40"N	92°14'7.11"W	≈ 0	MCA 3
B-16	30	29°43'4.45"N	92°14'17.10"W	≈ 0	MCA 3
B-17	60	29°42'39.81"N	92°12'6.77"W	≈ 0	Culvert
B-18	60	29°42'2.39"N	92°13'21.90"W	≈ 0	Culvert
B-19	20	29°43'6.50"N	92°10'25.19"W	5	Borrow Area
B-20	20	29°43'6.61"N	92°10'14.90"W	5	Borrow Area
B-21	20	29°43'6.67"N	92°10'4.65"W	5	Borrow Area
B-22	20	29°43'1.06"N	92°10'25.10"W	5	Borrow Area
B-23	20	29°43'1.17"N	92°10'14.83"W	5	Borrow Area
B-24	20	29°43'1.20"N	92°10'4.58"W	5	Borrow Area
B-25	30	29°44'15.76"N	92°13'45.96"W	≈ 0	Rock Wall
B-26	30	29°44'26.64"N	92°13'27.22"W	≈ 0	Rock Wall

* See Figure A-1 for Marsh Creation Area (MCA) designation
† With the exception of the Borrow Area, borings were performed at marsh edge at or very near to the water level

A total of 377 thin-walled tube samples were obtained during the field exploration program. Sample recovery lengths were measured in the field and upon extrusion in the laboratory. In general, sample recovery generally ranged from 18 to 24 inches of recovery out of the total 24-inch sample stroke, with an average recovery of 21 inches (i.e., average 87.5% recovery). In a few instances where less than 80 percent recovery on an individual sample was achieved, the corresponding depth interval was resampled.

SECTION 3. LABORATORY TESTING

3.1 Laboratory Testing Overview

Periodically during and upon completion of our field exploration work, soil samples were transported to our Baton Rouge laboratory. The tube samples were stored in secure racks in an upright position and protected from vibration and the elements. The samples were removed from the sampling tubes in the laboratory using a specially fabricated hydraulic piston-type extruder.

In light of the very soft character of the samples, particularly those obtained from depths less than about 10 feet below the mudline, sample extrusion was coordinated with specimen selection and testing to minimize sample disturbance by avoiding even short-term warping once outside the sampling tube. In order to preserve representative portions for later consolidation and strength testing, the bottom 6-inch section of several tube samples (which typically exhibit the least sampling disturbance effects) were cut using a fine-toothed band saw and sealed in the sampling tube section. The remainder of the tube sample was then extruded, classified and subjected to index and compressive strength testing. Hand-operated Torvane shear strength tests were also performed on the ends of the samples primarily to assess strength variability within the sample group. This procedure enabled evaluation of the corresponding classification and index strength data to guide selection of the most representative or potentially more critical samples for the more sophisticated consolidation and strength testing.

In order to avoid disturbance that might otherwise occur due to bonding of the sample to the inside of the galvanized steel sampling tube during the testing program period, a special technique was used for extrusion of samples from the 6-inch cut sections. Specifically, a thin wire was carefully inserted along the edge of the sample and inside face of the tube and was then held taught to enable wire-cutting around the circumference of the sample prior to careful manual extrusion. This technique was developed and is standard practice in the geotechnical laboratory at the Massachusetts Institute of Technology (Germaine and Germaine, 2009).

Essentially all of the planned laboratory classification and index testing, unconsolidated-undrained shear strength testing, and laboratory vane shear strength testing has been completed to-date. Laboratory incremental consolidation testing of natural ground soil samples is ongoing, with 12 tests completed to-date. A series of consolidated-undrained direct simple shear strength tests are also still in-progress. Results of these ongoing tests, along with results of settling column and slurry consolidation tests on representative composite samples from the proposed borrow area, will be included in the final data report. An overview of the scope of the laboratory testing phase in terms of the type and number of tests performed thus far is presented in Table 3.1 below. Results of the laboratory tests and their implication with respect to design material property selection are presented and discussed in the following sections of this report.

Table 3.1 Laboratory Testing Summary

Test Method	ASTM Reference	Number of Tests Performed
Unconsolidated Undrained (UU) Triaxial Compression Test	ASTM-D2850	185
Consolidation Test	ASTM-D2435	12
Atterberg Limit Determination	ASTM-D4318	190
Organic Content	ASTM-D2974	38
Moisture Content	ASTM-D2216	515
Grain Size Analysis	ASTM-(C136,D1140,D422)	50
Unit Weight Determination	ASTM-D2937	502
Specific Gravity Determination	ASTM-D854	14

3.2 Classification and Index Testing

Although subsurface conditions, in terms of soil classification, geotechnical index properties, effective stress history and undrained shear strength profiles, encountered at the boring locations are generally consistent, the data do exhibit some notable regional variability within the project area. In general, the soil profile encountered with the project area can be characterized as consisting of 10 to 15 feet of soft clay and organic clay underlain by medium to stiff lean clay (CL) down to depths of 80 feet (the maximum depth explored). No significant sand or silt strata were encountered.

Based on review and evaluation of the field exploration and laboratory testing data, the project site was sub-divided into four general regions for the purpose of geotechnical characterization. The subsurface conditions, in terms of geotechnical index and physical properties, within the following four areas are described below:

- Marsh Creation Areas 1 & 2 (MCA 1-2); including the potential Rock Wall revetment area west of MCA 1
- Marsh Creation Area 3 (MCA 3)
- Culvert Structure Locations
- Borrow Area

3.2.1 Visual Classification

Visual classification included description of soil color, consistency and type, and identification of structural conditions (layering, seams, etc.) and variations (organics, oxide inclusions, etc.). Visual classifications for the soil samples obtained from the site are incorporated into the soil boring logs in Appendix A.

3.2.2 Moisture Content and Density

More than five hundred moisture content determinations (ASTM D2216) and total unit weight determinations (ASTM D2937) were performed in conjunction with the sample extrusion process and preparation of test specimens. Total unit weights of the tube samples were computed based on sample volume and weight measurements taken after exclusion of any materials that appeared to have been disturbed during the sampling or extrusion process (occasionally encountered at the top of the tube sample). Moisture content determinations were made for each extruded sample, and dry densities were computed for each sample. Considering that all samples were obtained either at or below the water surface, degrees of saturation were computed to confirm that the density and moisture content values correspond to near 100 percent saturation as a quality control measure. Moisture content and dry density values for each sample are included on the soil boring logs in Appendix A.

Variations in moisture content versus depth below the mudline at all of the soil boring locations within the project area (i.e., excluding the proposed borrow area) are illustrated on Figures B-1A, B-1B and B-1C (Appendix B). Samples obtained from the mudline down to a depth of about 10 feet generally exhibit elevated and highly variable moisture contents ranging from 60 to more than 200 percent. At depths greater than 10 feet, moisture contents are consistently lower, generally ranging from 20 to 50 percent, and tend to be much more uniform.

Variations in total unit weight versus depth below the mudline for boring locations within the project area are illustrated on Figures B-2A, B-2B and B-2C. Samples obtained from the mudline down to a depth of about 10 feet generally exhibit much lower, more highly variable wet densities ranging from 60 to 100 pounds per cubic foot (pcf). At depths greater than 10 feet, total unit weights are consistently higher, generally ranging from 100 to 125 pcf, with significantly less variability.

Dry densities of the tube samples and strength test specimens were computed based on measured total unit weights and moisture contents. Variations in dry density with depth are shown on Figure B-3A, B-3B and B-3C. Samples of the more organic clay soils obtained from the upper 10 feet below the mudline generally displayed more highly variable dry densities ranging from about 10 to 60 pcf. Dry density values in the less organic soils from depths greater than 10 feet are higher and generally less variable, ranging from about 75 to 100 pcf. One notable exception can be seen in Figure B-3C where lower dry densities persist to a greater depth on the order of 20 feet at the location of boring No. B-18 (proposed hydraulic structure location).

3.2.3 Specific Gravity

Specific gravity determinations were performed on samples corresponding to those selected for one-dimensional laboratory consolidation testing. The specific gravity of samples tested ranged from 2.45 to 2.76, with an average value of 2.72. The lower range specific gravity values are consistent with what is expected for highly organic soils such as those encountered at the site.

3.2.4 Organic Content

A total of 38 organic content determinations (ASTM D 2974) were performed on selected samples. The results of organic content tests are presented on the soil boring logs in Appendix A, and are plotted versus depth below the mudline on Figures B-4A, B-4B and B-4C (Appendix B). In general, the soft marsh soils within 10 feet of the mudline or ground surface display variably high organic contents ranging from 10 to 80 percent. Clay soil samples obtained deeper than about 10 feet typically display relatively low organic contents.

3.2.5 Atterberg Limits

A total of 190 Atterberg limit determinations (ASTM D4318) were performed on selected samples to assist in soil classification and to enable correlation to pertinent clay behavior properties. The Atterberg limit data consist of measured liquid limit (LL) and plastic limit (PL) values from which the plasticity index (PI = LL – PL) is derived. The individual test data are included on the boring logs in Appendix A. The test results are also presented in terms of plasticity charts on Figures B-5A through B-5C and variation with depth on Figures B-6A through B-7C (Appendix B). The data indicate that the marsh deposits in the upper 10 to 15 feet below the mudline predominantly consist of highly plastic clay or organic clay, classifying as CH- and OH-type soils in accordance with the Unified Soil Classification System. Liquid limits within the upper 10 feet are generally highly variable, ranging from about 75 to 150 percent. The clay soils encountered at depths greater than about 10 feet are somewhat less plastic, classifying as CL and CH type clay soils with liquid limits typically ranging from about 40 to 80 percent.

The Liquidity Index (LI) is a parameter that characterizes the *in situ* moisture content of a sample in relation to its liquid and plastic limit values (LI= [MC-PL]/PI). Clay soils having high liquidity indices, i.e., approaching or even greater than 1.0, have *in situ* water contents that are near to or above their liquid limit, which is characteristic of very soft and compressible "normally consolidated" or very lightly overconsolidated conditions. As can be seen on Figures B-8A , B-8B and B-8C, these compressible type conditions are prevalent in the marsh deposits from the mudline down to a depth of about 10 feet as evident in the elevated liquidity index values. Liquidity index values throughout the project area tend to decline with depth down to about 25 feet below the mudline. At deeper depths, liquidity index values tend to increase and become more variable. Considering that moisture content values were found to remain relatively constant at the deeper depths (see Figures B-1A, B-1B and B-1C), the observed increase in liquidity index at depths greater about 25 feet reflects a general decline in plasticity (Atterberg limits) with depth within the deeper clay soils.

3.2.6 Particle Size Distribution

A total of 27 hydrometer particle size analysis tests (ASTM D422), 2 full sieve analyses and 21 fines content determinations (ASTM D 1140) were performed on selected samples. The test results, in terms of percent fines (i.e., percent by dry weight finer than the U.S. No. 200 sieve size, 0.074 mm, or combined silt and clay fraction) are included on the soil boring logs in Appendix A. Percentages of gravel, sand, silt, and clay size particles based on results of the hydrometer tests are summarized in Figure B-9 in Appendix B. Individual test results, in terms of grain size distributions curves, are included in Appendix E.

3.2.7 Borrow Area Index Properties

A series of geotechnical index tests were performed on samples obtained from the six soil borings performed within the proposed borrow area for marsh creation (i.e., boring Nos. B-19 through B-24; see Figure A-1 in Appendix A). The test results, including natural moisture content, density, organic content, Atterberg limits and undrained shear strength are plotted versus depth below mudline in Figures B-16 through B-24 (Appendix B). In general, samples obtained from the four boings performed in the western two-thirds of the proposed borrow area displayed relatively uniform soil properties. However, samples obtained from the eastern boring Nos. B-21 and B-24 consistently display higher moisture contents, lower densities and lower shear strength from the mudline down to a depth of about 13 feet. The significantly different properties at these locations appear to suggest that soils within this eastern portion of the borrow area may have recently been deposited within a formerly excavated area or eroded channel. Review of recent past historical aerial images, including the 2006 aerial image shown in Figure B-25 in Appendix B appears to support this hypothesis. Regardless, the very low *in situ* dry density within the eastern portion of the proposed borrow area is not indicative of a desirable dredge borrow area since the volumetric excavation needed to mobilize solids for marsh creation from this area would be significantly higher than from the adjacent natural ground areas. Considering the general uniformity observed in the western portion of the proposed borrow site, we envision that its proposed boundaries may need to be revised and extended in the north-south direction to avoid the low density recent sediments.

3.3 Consolidation Tests

Incremental consolidation tests (ASTM D 2435) were included in the laboratory testing program to enable assessment of stress history and determination of one-dimensional stress-deformation and time-rate of consolidation characteristics of the marsh clay deposits that will dictate post-construction settlement of the terrace berms. To-date, a total of 13 tests have been completed and others are in progress.

Considering the very soft and compressible character of the marsh clay samples in light of the index data discussed above, the laboratory consolidation tests were generally performed using a reduced load increment ratio, LIR, on the order of 0.5 (versus the customary increment ratio of 1.0 where loads are doubled in each increment). The use of a lower LIR improves resolution of the compression curve and provides more data within the low effective stress range around the *in situ* and final design stresses beneath the proposed terrace berms and marsh fill loads. The use of the lower LIR does, however, extend the test duration since the number of load increments normally required to complete a test increases by a factor of two. Tests performed on soft marsh clay samples typically included one unload-reload cycle to enable evaluation of recompression behavior.

During each load increment, the accumulation of vertical displacement with time is measured. In general, each load increment was sustained for a period of 12 to 24 hours. At times, the increment duration was increased to assess long-term drained creep behavior. The vertical displacement versus time data was evaluated using the conventional log-time and square-root time curve fitting techniques to determine the end of primary consolidation (i.e., the point in time at which dissipation of load-induced excess pore water pressures in the sample had dissipated and drained creep ensued for each load increment). The individual test results, in terms of vertical strain versus effective vertical stress, are presented in Appendix C. (Individual load increment time curves will be included in the final report.)

Typical laboratory consolidation test results, considered in terms of vertical strain, ε_v (%; at the end of primary consolidation), versus vertical effective stress, σ'_{vc} (tons/ft^2; log scale), may be simply characterized as being composed of recompression and virgin compression. The flatter recompression portion of the ε_v versus log σ'_{vc} curve occurs at vertical effective stresses lower than the preconsolidation pressure, σ'_p to which the specimen had historically been subjected. The steeper virgin compression portion of the ε_v versus log σ'_{vc} response occurs at vertical effective stresses greater than the maximum past pressure, σ'_p. In the case of the marsh deposits, particularly within 10 to 15 feet of the mudline, the preconsolidation is "apparent", and results largely from post-deposition drained creep.

3.3.1 Compression Characteristics

The compression ratio, CR, is defined as the slope of the virgin compression portion of the ε_v versus log σ'_{vc} curve and can be used to predict the magnitude of consolidation settlements for normally consolidated foundation clays. Compression ratios for the soft marsh clay and organic clay deposit samples tested from depths less than about 10 feet below the mudline, generally range from 0.24 to 0.40, with an average value of 0.32. Samples obtained from depths greater than about 10 feet generally displayed lower CR values, averaging about 0.15. The compression index, C_c, characterizes the slope of the void ratio, e, versus log σ'_{vc} curve, and is equal to (1+eo) x CR where e_o is the initial specimen void ratio. Site-specific correlations between the virgin compression parameters CR and C_c and various index properties are shown in Figure B-10.

The slope of the recompression portion of the laboratory consolidation curve is used to estimate primary consolidation settlement magnitudes for stress increments resulting in final stress levels less than the preconsolidation pressure. Because the initial recompression behavior in the laboratory test can be influenced by sample disturbance (sampling stress relaxation, etc.), an unload-reload sequence is typically included to enable better assessment of *in situ* recompression behavior. The recompression ratio, RR, is defined as the slope of the recompression portion of the ε_v versus log σ'_{vc} curve. Recompression ratios for samples, obtained from depths less than about 10 feet below the mudline within soft marsh deposits, generally range from 0.02 to 0.04, with an average value of 0.03.

The continued accumulation of vertical strain with time subsequent to the end of primary consolidation is referred to as secondary consolidation (or drained creep). This component of clay compression behavior is important to estimating long-term settlements (and to the overall coastal subsidence situation). The coefficient of secondary compression, $C_{\alpha\varepsilon}$, quantifies the creep rate in terms of strain per log cycle of time after the end of primary consolidation. This parameter is derived from the individual load increment time curves generated during the consolidation tests. It is generally acknowledged that the ratio between the coefficient of secondary compression and primary compression ratio (i.e., increment "CR" being the tangential slope of the ε_v versus log σ'_{vc} curve at a given stress level) tends to be a constant value for a given material (Mesri and Castro, 1987). This behavior is relied upon for estimating long-term creep settlement behavior in the numerical models designated for use by CPRA in the engineering phase of this project. The relationship between $C_{\alpha\varepsilon}$ and CR for samples tested to-date is illustrated in Figure B-11 (Appendix B).

3.3.2 Preconsolidation Pressure

Any elements within the natural ground clay having a preconsolidation pressure equal to the *in situ* vertical effective stress (i.e., $\sigma'_{vc} = \sigma'_p$) is considered to be normally consolidated. Elements with *in situ* vertical effective stresses less than the maximum past pressure are considered to be overconsolidated (the higher past stresses are most likely associated with post-deposition drained creep and desiccation related to vegetation within the upper 10 feet at the subject site). These two stresses define the stress history of a clay element which, in turn, strongly influences its undrained shear strength and future compression behavior when loaded. Determination of the maximum past pressure is, therefore, critical to the evaluation. This determination involves estimating the vertical effective stress at which the transition from recompression to virgin compression occurs. Since the actual ε_v versus log σ'_{vc} curves measured in the laboratory do not consist simply of the two linear portions as discussed above, several techniques are conventionally used to provide an estimate of the maximum past pressure. The Casagrande construction (Casagrande, 1936) and strain energy methods (Becker et al., 1987) were used in our evaluation of the laboratory data.

Estimated maximum past pressure, σ'_p, values are included on the individual test summary plots in Appendix C and are summarized versus depth on Figure B-12 (Appendix B). The data indicate very slight degrees of overconsolidation (i.e., $\sigma'_p > \sigma'_{vo}$) within the upper 10 feet below the mudline. Samples taken from deeper than about 10 feet generally display significantly higher preconsolidation pressure values.

3.3.3 Coefficient of Consolidation

The coefficient of consolidation, c_v, is a parameter that quantifies the time-rate of consolidation and is dependent on, among other things, the material type and stress history. Coefficients of consolidation were computed using square-root and logarithm of time curve fitting techniques for each load increment applied during the consolidation tests. The relationship between the laboratory measured coefficient of consolidation (taken as the arithmetic average of the two curve fitting techniques) and the applied effective stress is presented for each test on the figures in Appendix C and are summarized on Figure B-13 in Appendix B.

3.4 Strength Tests

The strength characteristics of the marsh deposits are important for geotechnical engineering analyses, particular related to stability of the terrace berms and associated excavations.

3.4.1 Unconsolidated-Undrained Triaxial Compression Tests

A total of 185 unconsolidated-undrained (UU) triaxial compression tests (ASTM D2850) were performed on specimens trimmed from selected samples. Results of these strength tests are included on the soil boring logs in Appendix A. Individual UU test stress-strain curves are included in Appendix D. Undrained shear strengths from the UU compression tests are plotted versus depth in Figures B-14A, B-14B and B-14C (Appendix B). The test results indicate, as expected, relatively low undrained shear strengths in the surficial marsh deposits with substantially higher and more variable strengths in the deeper clays. It is noteworthy that the transition from soft to stiff behavior at the Marsh Creation Area 3 boring locations was found to be somewhat deeper (about 15 feet) that observed in the Marsh Creation Areas 1 & 2 and Hydraulic Structure location borings. This observation suggests that subsoil conditions within the Marsh Creation Area 3 site will be somewhat more challenging with regard to containment berm stability.

3.4.2 Torvane Index Strength Tests

Hand-operated torvane (TV) index strength tests were performed in conjunction with the sample extrusion process. These test results are considered to be index strengths in that the absolute value of the measured undrained shear strength is generally not considered adequately reliable for use in design. The test results, however, are useful in identifying soil strength variability and trends with respect to depth, material type, etc. A series of laboratory miniature vane (LV) shear strength tests were also performed on selected samples. Measured torvane strengths, along with shear strengths measured in the UU compression tests, are presented versus depth in Figures B-15A and B-15B (Appendix B). As can be seen, the TV and LV index test results are generally consistent with undrained shear strengths and the UU strengths derived from the compression tests.

3.4.3 Laboratory Vane Shear Strength Tests

Mechanical laboratory miniature vane (LV) shear strength tests were performed on selected samples to provide additional data to guide selection of design shear strength profiles. The tests were performed with a calibrated, spring-loaded, motor driven Wykeham Farrance Model 2350 vane shear device. The tests were conducted using a 0.75-inch high, 0.5-inch diameter vane rotated at an approximate rate of 10 degrees per minute until the peak torque was achieved. Undrained shear strength of the soil around the vane is calculated based on the spring constant and vane geometry. Measured laboratory vane strengths are included on Figures B-15A and B-15B (Appendix B).

3.4.4 Consolidated-Undrained Shear Strength Tests

Consolidated-undrained (CK_oU) direct simple shear (CKoUDSS) strength tests are in progress, but have not yet been completed. These tests are being performed on samples consolidated in the laboratory under a vertical effective consolidation stress significantly greater than the preconsolidation pressure in order to determine normally consolidated undrained strength ratios, s_u/σ'_{vc}, and the relationship between strength ratio and overconsolidation ratio in accordance with the SHANSEP (Stress History and Normalized Soil Engineering Properties) design methodology (Ladd & Foott, 1974). These tests will be used, along with the UU and TV strength data discussed above, to select final undrained shear strength profiles for use in design stability analyses.

Undrained shear strength profiles are estimated in accordance with SHANSEP according to the following normalized undrained shear strength, s_u/σ'_v, equation:

$$s_u / \sigma'_v = C \cdot OCR^m$$

Where: $OCR \equiv$ overconsolidation ratio = σ'_{vm} / σ'_v
$C = s_u / \sigma'_v$ for OCR = 1.0 (i.e., normally consolidated)
m = experimentally determined exponent coefficient

For preliminary assessment of the UU, TV and LV shear strength database for this project, SHANSEP strength parameters, C=0.24 and m=0.7, were tentatively selected based on experience with soft clay deposits having plasticity characteristics similar to those encountered at the subject site. An undrained shear strength profile computed according to the SHANSEP methodology is plotted alongside the current UU and TV strength test data on Figures B-15A and B-15B. As can be seen, the computed strength profile based on normalized clay behavior is generally consistent with the lower bound of the measured data.

SECTION 4. REFERENCES

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