FINAL DESIGN REPORT
COLE'S BAYOU MARSH RESTORATION PROJECT (TV-63)

VERMILLION PARISH, LOUISIANA

Ardaman & Associates, Inc.

OFFICES

Orlando - 8008 S. Orange Avenue, Orlando, Florida 32809 - Phone (407) 855-3860
Alexandria - 3609 Mac Lee Drive, Alexandria, Louisiana 71302 - Phone (318) 443-2888
Bartow - 1525 Centennial Drive, Bartow, Florida 33830 - Phone (863) 533-0858
Baton Rouge - 316 Highlandia Drive, Baton Rouge, Louisiana 70884 - Phone (225) 752-4790
Cocoa - 1300 N. Cocoa Blvd., Cocoa, Florida 32922 - Phone (321) 632-2503
Fort Myers - 9970 Bavaria Road, Fort Myers, Florida 33913 - Phone (239) 768-6600
Miami - 2608 W. 84th Street, Hialeah, Florida 33016 - Phone (305) 825-2683
Monroe - 1122 Hayes Street, West Monroe, Louisiana 71292 - Phone (318) 387-4103
New Orleans - 1305 Distributors Row, Suite I, Jefferson, Louisiana 70123 - Phone (504) 835-2593
Port St. Lucie - 460 Concourse Place NW, Unit 1, Port St. Lucie, Florida 34986 - Phone (772) 878-0072
Sarasota - 78 Sarasota Center Blvd., Sarasota, Florida 34240 - Phone (941) 922-3526
Shreveport - 7222 Greenwood Road, Shreveport, Louisiana 71119 - Phone (318) 636-3673
Tallahassee - 3175 West Tharpe Street, Tallahassee, Florida 32303 - Phone (850) 576-6131
Tampa - 3925 Coconut Palm Drive, Suite 115, Tampa, Florida 33619 - Phone (813) 620-3389
West Palm Beach - 2200 North Florida Mango Road, Suite 101, West Palm Beach, Florida 33409 - Phone (561) 687-8200

MEMBERS:
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Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

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AAI File: 13-80-3705

Coastal Protection and Restoration Authority
P.O. Box 44027
Baton Rouge, LA 70804

Attention: Ms. Amanda Taylor, E.I.
Amanda.Taylor@la.gov

Re: Final Design Report
Cole's Bayou Marsh Restoration Project (TV-63)
Vermillion Parish, Louisiana

We have completed the geotechnical analyses associated with the Cole's Bayou Marsh Restoration Project (TV-63). A summary of the results of our analyses and recommendations for the proposed marsh creation areas are provided herein. 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.

STATE OF LOUISIANA
ROBERT J. WERNER
License No. 36633
PROFESSIONAL ENGINEER IN CIVIL ENGINEERING

FOR
MEGAN G. BOURGEOIS, P. E.
PROJECT ENGINEER

ROBERT J. WERNER, P.E.
BRANCH MANAGER/PRINCIPAL ENGINEER

RJW/nma

(6) bound originals

316 Highlandia Drive 70810, Post Office Box 83710, Baton Rouge, Louisiana 70884 Phone (225) 752-4790 Fax (225) 752-4878
Louisiana: Alexandria, Baton Rouge, Monroe, New Orleans, Shreveport
Florida: Bartow, Cocoa, Fort Myers, Miami, Orlando, Port Charlotte, Port St. Lucie, Sarasota, Tallahassee, Tampa, West Palm Beach

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FINAL DESIGN REPORT
COLE'S BAYOU MARSH RESTORATION PROJECT (TV-63)

VERMILLION PARISH, LOUISIANA

This Final Design Report summarizes results of our geotechnical engineering analyses and presents basic design and construction recommendations for proposed marsh creation areas associated with the Cole's Bayou Marsh Restoration Project (TV-63). Typical design cross sections for earthen containment dikes having a range of construction grades and corresponding 20-year marsh elevations are presented on Figures A-1 through A-10 in Appendix A. These sections were developed based on results of stability and settlement analyses that incorporated results of the geotechnical field exploration and laboratory testing phase of the project as documented in our March 31, 2014 Final Data Report (Appendix C). Results of slope stability analyses and settlement calculations are included in Appendices A and B, respectively.

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.

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 Coastal Protection and Restoration Authority (CPRA). A detailed description of the field exploration and laboratory testing phases of the project, along with boring logs, boring location plan, subsurface profiles, plots of laboratory data, and strength and consolidation test results are presented in our Final Data Report, dated March 31, 2014. A copy of the Final Data Report is provided in Appendix C.

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 1.9 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.

SECTION 2. SOIL CONDITIONS

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

A total of 26 borings were performed in October 2013 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 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 were assigned along a proposed rock wall alignment.

The soil conditions encountered at all of the boring locations generally consisted of very soft to soft clay (CH, per the Unified Soil Classification) and silty clay (CL) with organics to depths ranging from about 10 feet below the mudline in Marsh Creation Areas 1 & 2 to about 15 feet below the mudline in Marsh Creation Area 3. These soft surficial soils are generally underlain by medium to stiff clay (CH) or silty clay (CL) soils down to the 60-foot total depth of the soil borings. A detailed geotechnical characterization of the site, along with supporting field exploration and laboratory testing results, are documented in the March 28, 2014 Final Data Report (Appendix C).

SECTION 3. CONTAINMENT DIKE DESIGN

3.1 Containment Dike Analyses Overview

As requested by CPRA, a range of marsh creation construction grades, and hence a range of containment dike construction grades were analyzed. It is our understanding that the influence of water control devices currently in use by the landowner prevents monitoring normal water level fluctuations within the site area and thus there is some uncertainty in establishing representative site-specific Mean High Water (MHW) and Mean Low Water (MLW) elevations. As directed by CPRA, we have based the analyses on a "Low Water" elevation of -0.01 feet, NAVD88, and a "High Water" elevation of +1.26 feet, NAVD88. A projected 20-year Sea Level Rise (RSLR) of +0.69 feet for the Cole's Bayou site was provided by CPRA (combined sea level rise and subsidence).

Bathymetric transect survey data for the Cole's Bayou marsh restoration areas were provided by CPRA in the form of a spreadsheet file listing all of the survey point locations and elevations.

These data were processed using ArcGIS software and superimposed on a rectified aerial photograph to produce mudline contour maps. Based on these data, a representative average mudline elevation of 0.0 feet NAVD88 was selected for the two smaller northern marsh restoration areas (Areas 1 and 2). The average mudline elevation within the larger Marsh Area 3 is slightly lower, at about -1.0 feet NAVD88.

3.2 Slope Stability

Slope stability analyses were performed for each containment dike design case, i.e., with varying dike crest construction grades. Analyses were performed for "end of dike construction" and "end of dredging" cases. The "end of dredging" analyses considered an outboard water level of +0.0 feet NAVD 88 (current "low water" elevation) and an inboard water level corresponding to a freeboard of 1 foot relative to the crest of the containment dike. Steady state seepage conditions through the containment dike were assumed. Typical design natural ground soil profiles and material properties were selected based on our evaluation of the field exploration and laboratory testing results. Selection of properties used in the stability analyses, and basis for that selection, are discussed in the following paragraphs.

Marsh Creation Areas 1-2

Very soft highly plastic clay and silty clay to a depth of 10 feet below the existing mudline, was assigned a total unit weight of 85 lb/ft^3 and an undrained shear strength, s_u linearly increasing from 100 lb/ft^2 at the top of the layer to 150 lb/ft^2 at the bottom. These low shear strengths reflect only a slight degree of overconsolidation (possibly due to mild historic post-deposition desiccation related to exposure and/or plant root uptake during long-past periods when the sediments were above or very near to the marsh water level). Design shear strengths were selected recognizing that the marsh deposits will exhibit strain rate sensitivity and inherent strength anisotropy, i.e., the horizontally deposited and consolidated clay will exhibit higher strengths in compression than in simple shear, the latter being more representative of the field shear mode. Results of laboratory consolidation tests performed on samples of the very soft marsh clay, when assessed in terms of vertical strain magnitudes during initial recompression versus the *in situ* vertical effective stress, indicated that relatively high sample quality was achieved during the field exploration and sampling phase. For such minimally disturbed high quality samples, undrained shear strengths derived from unconsolidated-undrained (UU) triaxial compression tests will tend to be higher than *in situ* strengths along horizontal or mildly inclined failure surfaces when sheared at field strain rates. Accordingly, the selected shear strength profile is nearer to the lower bound of the range of laboratory-measured undrained shear strengths, particularly those derived from laboratory UU-type triaxial compression tests.

Stiffer clay soil encountered at depths greater than 10 feet below the mudline were assigned total unit weights increasing from 90 to 120 lb/ft^3 and undrained shear strengths increasing with depth from 400 to 800 lb/ft^2.

Marsh Creation Area 3

Based on results of the field and laboratory data, subsurface conditions within the Marsh Creation Area 3 site include very soft highly plastic clay that extends somewhat deeper than within the two smaller areas (i.e., Areas 1 and 2). Marsh clay from the mudline to a depth of 15 feet was assigned total unit weights increasing from 85 to 90 lb/ft^3 and undrained shear strength linearly increasing from 100 to 200 lb/ft^2. Stiffer clay soil encountered at depths greater than 15 feet below the mudline was assigned a total unit weight of 120 lb/ft^3 and undrained shear strength of 800 lb/ft^2.

3.2.1 Method of Analysis

Two-dimensional limit equilibrium stability analyses for the containment dike/marsh creation cross sections were performed using the method of slices and Spencer's method (i.e., assuming a common inclination of inter-slice forces). The SLOPE/W (GeoStudio Version 7.03) computer program was used for calculating slope stability factors of safety for trial failure surfaces. This program incorporates automated search routines that, when properly managed, can systematically converge on kinematically admissible critical circular arc and sliding wedge type failure surfaces. Results of the slope stability analyses for containment dike and borrow excavation cross sections are presented and discussed in the following sections.

3.2.2 Containment Dike Cross Section

Stability analyses were performed for the more critical "end of construction" case and considering a water level of 0.0 feet NAVD88 (i.e., the "low water" condition). Analyses for the containment dike slope stability and overall global stability reflect dike crest elevations necessary to provide at least 1 foot of freeboard during filling to the proposed marsh restoration construction grades. Details regarding construction elevations, side slopes, and factors of safety for the containment dike cases are presented below in Table 3.1.

Table 3.1 Containment Dike Stability Analyses Results

Marsh Area(s)	Estimated Berm Crest El. (ft. NAVD 88)	Borrow Excavation Offset (ft.)	Berm Side Slopes	Factors of Safety		
				Circular Arc in Berm Slope	Circular Arc Bearing Capacity	Global Sliding Toward Borrow Pit
1	+3.0	20	4H:1V	2.02	2.74	2.66
	+4.0			1.61	2.22	2.18
2	+3.0	20	4H:1V	3.97	4.89	3.31
	+4.0			2.71	2.86	2.62
3	+3.0	20	4H:1V	2.02	3.10	2.65
	+4.0			1.61	2.38	2.08

The computed factors of safety for all terrace berm geometries analyzed are greater than 1.5. From a stability standpoint somewhat steeper slopes than the given 4H:1V could be used.

3.2.3 Borrow Excavation Cross Section

It is our understanding that the perimeter containment dikes for Marsh Creation Areas 1 and 2 will be constructed using materials excavated from the interior of the proposed marsh creation areas at an offset from the containment dike alignment. These "temporary" borrow excavations will be subsequently filled with dredged soil in conjunction with marsh creation. A series of stability analyses were performed to develop slope and depth recommendations for these interior borrow excavations (for Marsh Creation Areas 1 and 2) that will be required to generate the fill materials needed to construct the marsh creation area perimeter containment dikes A marsh water level corresponding to the current "low water" elevation of 0.0 feet NAVD88 was used in these analyses. Based on results of these submerged cut slope analyses, a borrow excavation slope of 2H to 1V was adopted for use in development of the typical containment dike cross sections for Marsh Creation Areas 1 and 2 (Appendix A).

In order to provide for post-construction hydraulic conveyance around the area, the containment dikes for Marsh Creation Area 3 will be constructed using soils excavated to form a channel along the outside perimeter of the area. Because this borrow channel has a "permanent" function, and considering its potential influence on the long-term stability of the marsh area containment berm, the cut slope was analyzed under somewhat different (more conservative) conditions. Moreover, we recognized that the cut slope may become exposed under occasional extreme low water cycles and be subject to at least some wave action and erosion. Slope stability analyses for this borrow excavation were made incorporating a reduced shear strength profile to reflect the potential for residual strength conditions (e.g., a condition that could develop if the submerged excavation was inadvertently cut to a very steep slope), and a reduced outboard water level to reflect an extreme low water condition (as can occasionally occur). Based on results of these analyses, a borrow excavation slope of 4H to 1V was adopted for use in development of the typical containment dike cross sections for Marsh Creation Area 3 (Appendix A).

Global stability of the containment dike and borrow excavation section depends on the offset distance between the dike and edge of the excavation as well as on cut depth. A minimum offset distance between the toe of the containment dike slope and top of the borrow excavation slope of 20 feet was adopted to provide adequate factors of safety as well as adequate width for excavation equipment access between the proposed cut and fill areas for a maximum excavation depth of 8 feet. This offset distance can be greater than 20 feet, and should be increased if the excavation depth will exceed 8 feet, particularly for the outboard Marsh Creation Area 3 borrow excavation.

3.3 Settlement Evaluation

The natural ground marsh clay deposits on which the proposed marsh area containment dikes will be constructed are characteristically soft and compressible. Detailed calculations were performed for each trial dike geometry to estimate post-construction settlement magnitudes and rates. In addition, estimates of undrained settlement that will occur due to displacement and deformation of the marsh clay deposits as the fill is placed during construction to the target crown grade to enable calculation of the cut-to-fill ratio were calculated.

3.3.1 Initial Undrained Settlement (During Construction)

When fill is placed to construct a containment dike, shear stresses induced in the marsh clay cause lateral deformation of the soil resulting in vertical settlement of the fill, particularly when those shear stresses approach the undrained shear strength of the clay. This initial undrained settlement was estimated using the method developed for two-dimensional strip loading on soft plastic and organic clays by Foott and Ladd, 1981. The method utilizes undrained shear modulus values derived from the consolidated-undrained direct simple shear strength test performed during the laboratory testing phase (Appendix C), case-specific shear stress ratio (inverse of slope stability factor of safety), and elastic theory to estimate undrained deformations in the marsh clay beneath the containment dike.

The estimated initial undrained settlement magnitudes for the Cole's Bayou marsh restoration area containment dikes range from 5 to 7 inches for design dike crest elevations ranging from +3 to +4 feet NAVD88. This estimate is based on shear stresses corresponding to the end of construction for dike sections rapidly built to the proposed grades shown on Figures A-1 through A-10, Appendix A.

3.3.2 Primary Consolidation Settlement

Primary consolidation refers to volume changes in the foundation soil resulting from drainage of pore water that occurs as load-induced excess pore water pressures dissipate over time. A series of 23 incremental consolidation tests were performed during the laboratory testing phase 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 dictate post-construction settlement of the containment dikes.

Typical laboratory and field consolidation behavior, considered in terms of vertical strain, ε_v (%; at the end of primary consolidation), versus vertical effective stress, σ'_{vc} (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 clay 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.

The compression ratio, CR, is defined as the slope of the virgin compression portion of the ε_v versus log σ'_{vc} curve and is used to predict the magnitude of consolidation settlements for normally consolidated foundation clays. The compression ratio values listed in Table 3.3 below were selected for use in design settlement analyses based on results of the laboratory tests, and on the site-specific correlation derived from the test results (see Figure B-10 in Appendix C).

The recompression ratio, RR, is defined as the slope of the recompression portion of the ε_v versus log σ'_{vc} curve and 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 included to enable better assessment of *in situ* recompression behavior. The recompression ratio values listed in Table 3.2 below were selected for use in design settlement analyses based on results of the laboratory tests.

Table 3.2 Selected Primary Compression Parameters

Clay Layer No.	Depth Range (feet)	CR	RR
1	0 – 15	0.30	0.04
2	15 - 60	0.18	0.03

Elements with *in situ* vertical effective stresses less than the maximum past pressure are considered to be overconsolidated. These two stresses define the stress history of a clay element which, in turn, influences its compression behavior when loaded (i.e., whether the clay exhibits virgin compression or recompression behavior). The estimated maximum past pressure, σ'_p, profile for the Cole's Bayou site based on the laboratory test results is shown on Figure B-12 in Appendix C.

For the case of containment dikes, the applied load is two-dimensional and therefore the intensity to which the applied stress increase associated with construction of the dike is "felt" in the marsh clay soils varies with location and depth. A finite element computer program (SIGMA/W; Geostudio Version 7) utilizing linear elastic theory, was used to compute the distribution of vertical stress increase with depth beneath each trial containment dike cross section. The distribution of stress increase beneath the crown of the dike section derived from the elastic solutions were added to the computed existing *in situ* vertical effective stress profile to develop the final vertical effective stress profiles for each case. These profiles, along with the preconsolidation pressure profile established for the site, were used to compute the virgin compression and recompression components of primary consolidation. The primary consolidation calculations are summarized in Appendix B, and yielded settlement magnitudes ranging from approximately 6 to 9 inches for the construction crest grades analyzed.

3.3.3 Primary Consolidation Rate

The rate at which load-induced excess pore water pressures dissipate, and hence the rate at which primary consolidation settlements are realized, under one-dimensional loading conditions, can be formulated using classic Terzaghi consolidation theory. 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. The relationship between coefficient of consolidation and effective stress ratio derived from results of the laboratory consolidation tests is shown on Figure B-13 in Appendix C. It is well-established that the *in situ* coefficient of consolidation determined under field conditions is typically 5 to 10 times higher than laboratory-measured values from tests on discrete samples. The lower laboratory values can be attributed to specimen size effects (i.e., small lab test specimens often do not reflect the influence of thin interbedded silt and sand seams that can significantly affect consolidation rate behavior).

For the case of containment dikes, the applied load is two-dimensional which affects consolidation rate behavior to some extent since a lateral pore pressure dissipation component is introduced. The consolidation time curves developed by Christian, et al. (1972) were used to predict the time required to achieve various degrees of consolidation. Based on the calculations presented in Appendix B, the estimated time that will be required for post-construction primary consolidation settlements to take place is approximately 10 years (i.e., to achieve average degrees of consolidation greater than 90 percent). This settlement component is reflected, along with future low and high water trends developed using the RSLR projection provided by CPRA, on the containment dike settlement curves in Figures B-2 through B-4 (Areas 1 and 2) and Figures B-6 through B-8 (Area 3), Appendix B.

3.3.4 Secondary Compression (Drained Creep)

The continued accumulation of vertical strain with time subsequent to the end of primary consolidation is referred to as secondary consolidation (or drained creep). Secondary compression occurs as soil particles deform, or creep, under constant effective stress conditions. 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 laboratory 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 was relied upon for estimating long-term creep settlement behavior. The relationship between $C_{\alpha\varepsilon}$ and CR for Coles Bayou samples tested is illustrated in Figure B-11 (Appendix C). For design calculations, an average $C_{\alpha\varepsilon}$/CR (or $C_{\alpha\varepsilon}$/RR) ratio value of 0.046 was used. The secondary compression component of expected

containment dike settlement, occurring between post-construction years 10 through 20, is approximately one inch.

3.3.5 Containment Dike Fill Consolidation and Shrinkage

The containment dikes will be constructed using native marsh clay mechanically excavated from an underwater channel that will be formed alongside, and offset a minimum distance of 20 feet from the berm. It is anticipated that the saturated fill materials will be shaped, but not compacted, and will likely have an initial as-placed unit weight that approximates the *in situ* unit weight within the proposed borrow area. With time, the fill material will consolidate under its own weight and will be subject to long-term creep compression. Moreover, the above-water portion of the fill will desiccate towards its field capacity moisture content due to gravity drainage and evapotranspiration. For the purpose of estimating total post-construction subsidence of the terrace berm crown, we have included a 10 percent shrinkage factor to the fill height to account for post construction consolidation and shrinkage of the fill itself.

3.4 Borrow Excavation Cut to Containment Dike Fill Ratio

Although the expected initial total unit weight of the un-compacted berm fill materials is expected to be approximately equal to the existing *in situ* unit weight in the borrow area, the excavated borrow volume will need to exceed the neat design fill quantity in order to compensate for material "lost" to initial undrained settlement during construction. In addition, other "losses" can occur either during excavation of borrow soils from below water or due to loss after or during placement (e.g., material eroded or sloughed to areas outside the design cross section limits).

Considering a nominal total containment dike length of approximately 43,000 feet, we have computed approximate total neat in-place design fill volumes for each of the berm grade cases. This neat fill volume, along with estimated cut volume requirements and corresponding cut/fill ratios are summarized in the following table.

Table 3.5 Summary of Containment Dike Cut / Fill Volumes

Marsh Area	Containment Berm Length (ft.)	Design Crest Elevation (ft. NAVD88)	Design Fill Volume (Cubic Yards)	Required Cut Volume (Cubic Yards)	Cut / Fill Ratio
1	6,200	+3.0	19,125	24,175	1.26
		+4.0	28,450	35,050	1.23
2	11,300	+3.0	10,850	15,400	1.42
		+4.0	21,300	28,025	1.32
3	25,500	+3.0	79,125	99,975	1.26
		+4.0	117,750	145,050	1.23

The majority of the difference between the neat design fill quantity and required cut volume is compensation for initial undrained deformations. Fill placement means and methods, lift sequencing and placement details will all be important considerations in construction planning and control to limit the cut/fill ratio, and hence construction costs. Fill placement should generally be undertaken in successive horizontal lifts that encompass the entire dike width and should be limited to lift heights that do not cause excessive local subsidence or mud-waving.

From a construction efficiency, and hence cost, perspective, we envision that a single-phase, single-pass berm construction approach is preferred. In order to achieve the target over-built berm crown elevation and required slope geometry in a single-pass operation, the required volume of borrow material would need to be accessible to an excavator situated within the 20-foot wide offset between the containment dike and borrow excavation. That volume will be dictated primarily by the ground-level reach of the excavator. It is anticipated that a sufficient volume of borrow materials can be accessed using a conventional marsh-compatible long-reach excavator (having a center-pin reach on the order of 60 feet).

SECTION 4. MARSH CREATION AREA

4.1 Marsh Creation Area Overview

Within the containment dike, borrow material will be hydraulically dredged to create approximately 365 acres of new brackish marsh and nourish an additional 53 acres of existing brackish marsh. Typical containment dike/marsh creation area cross section were developed based on results of stability and settlement analyses. The marsh creation containment dike will have a design freeboard of 1.0 foot above the marsh construction elevation.

4.2 Settlement Evaluation

Post construction subsidence of the marsh creation areas will occur as a result of "self-weight" consolidation and drained creep of the dredged fill, primary and secondary consolidation of the *in situ* marsh soils induced by placement of the fill, and potential seasonal desiccation of the fill surface. The analyses were undertaken using classical one-dimensional Terzaghi consolidation theory through superposition of foundation and fill increments and ramp-loading solutions.

4.2.1.1 Primary Consolidation

Post construction subsidence of dredged fill surfaces resulting from primary consolidation of the underlying *in situ* soil were calculated using the same general theory and properties discussed in Sections 3.3.2 and 3.3.3, except that equations pertinent to one-dimensional conditions were used. The primary consolidation calculations are included in Appendix B, and yielded 20-year settlement magnitudes ranging from approximately 4 to 6 inches for the component related to primary consolidation of the foundation soils. Based on the calculations presented in Appendix B, the estimated time required for complete post-construction primary

consolidation in the foundation soils is about 15 years. This primary foundation settlement component is included in the marsh area settlement curve in Figures B-1 through B-8, Appendix B.

4.2.1.2 Secondary Compression (Drained Creep)

Post construction subsidence of dredged fill surface resulting from secondary compression of the *in situ* soils underlying the marsh creation areas was calculated using the same general theory and properties discussed in Section 3.3.4. As indicated in Appendix B, the calculated time required to achieve an average degree of consolidation of 90 percent is about 15 years. However, 90 percent or greater dissipation of excess pore water pressures occurs earlier in the upper depths of the soil profile, and it was assumed that secondary compression would ensue once that degree of dissipation was achieved (starting at post-construction year 5 in the uppermost 10 feet).

The marsh subsidence component related to secondary compression of the underlying natural ground soils, occurring during post-construction years 5 through 20, is approximately one inch.

4.2.1.3 Marsh Fill Consolidation and Shrinkage

Marsh creation will be accomplished using hydraulic dredging of native clay from a designated underwater borrow area. Geotechnical field and laboratory testing data for the proposed dredge material borrow site were obtained during the data collection phase (Appendix C).

Once deposited via dredge discharge into the marsh creation areas, the fill will initially settle under water and begin to compress under its own weight and will then consolidate under the weight of gradually added fill. The initial gravity settling and subsequent "compression settling" behavior of the proposed borrow materials were characterized by performing settling column tests (Final Data Report, Appendix C). Consolidation behavior of the settlement fill, was investigated by performing one incremental one-dimensional slurry consolidation test (Final Data Report, Appendix C). Results of settling column and slurry consolidation tests were evaluated and used to estimate initial settled, compressed void ratios and to select fill properties and consolidation parameters for use in settlement analyses included in Appendix B.

Primary consolidation and secondary compression calculations for the dredged fill are included in Appendix B. These calculations account for the fact that a portion of the self-weight consolidation occurs during the dredge fill placement process (and hence is compensated for through continued fill placement up until the design surface elevation is achieved). Total post-construction marsh area subsidence related to primary consolidation of the dredged fill is estimated to range from about 7 to 13 inches for the construction cases analyzed and is expected to occur within the first few years after fill placement. Subsequent secondary compression is expected to result in up to one additional inch of settlement over a 20-year post construction period. As reflected in the calculations included in Appendix B, the post-

construction consolidation subsidence stated above applies for dredge materials placed at a uniform rate corresponding to an average dredge excavation rate of 2,250 cubic yards per hour and relatively uniform discharge over the proposed marsh creation areas. The anticipated dredge filling duration for the largest area (i.e., Area 3) is expected to be on the order of 3 months.

It is our understanding that the marsh containment berms will be breached at strategic locations at some time after construction to allow more natural inflow-outflow patterns. Until such time the newly created marsh area is expected to remain essentially saturated (considering particularly that rainfall normally exceeds evaporation in southern Louisiana). As illustrated in the settlement time curves on Figure B-2 in Appendix B, subsidence of the marsh surface occurs most rapidly during the early post-construction years. Therefore, the marsh surface elevation will subside to a level less than 1 foot above the "high water" level relatively shortly after construction. It seems likely that capillarity and rainfall will act to maintain essentially saturated conditions. Assuming that accumulation of organic matter approximately compensates for moisture content reduction within the vegetation root zone, it seems reasonable to conclude that desiccation and shrinkage will not have a significant impact on post-construction behavior of the marsh creation area.

4.3 Borrow Excavation Cut to Marsh Fill Ratio

Based on testing results of soil borings performed in the proposed dredge borrow area (Boring Nos. B-19, B-20, B-22, and B-23) the average *in situ* dry density of soils within 15 feet below the mudline in the proposed borrow area is about 90 pcf. The calculated average dry density of the settled, compressed and slightly consolidated fill in the marsh creation area 30 days after completion of dredging is 30 pcf (Appendix B). Even if significant "losses" of fines to carryover through the marsh area decant occur due to dredge fill placement conditions, the dredge fill-to-cut ratio is still expected to be at least 1.5.

SECTION 5. CONCLUSION AND ADDITIONAL RECOMMENDATIONS

Typical design cross sections for the marsh creation area containment dikes are presented on Figures A-1 through A-10 in Appendix A. Post-construction (and post-dredging) settlement-time rate curves for the containment terrace berm sections are shown on Figures B-1 through B-8 in Appendix B. The cut/fill ratio for these design sections, which considers initial undrained subsidence during construction, is estimated to range from about 1.25 to 1.4.

Post construction settlement-time rate curves for the marsh creation interior areas are shown on Figures B-1 and B-5 in Appendix B.

5.1 Additional Exploration and Testing

Boring locations B-19 through B-24 were performed within the borrow area of Little Vermillion Bay. As is discussed in Section 3.2.7 of the Final Data Report (Appendix C), 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 2006 aerial images 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. Based on the relative uniformity observed in samples obtain from the four borings performed within the western portion of the borrow area, it should not be necessary to obtain additional data unless the borrow area is substantially relocated.

In light of the relatively low degree of variability observed in results of the exploration and testing phase of the study, the data collection phase is considered adequate for reliable characterization and design property selection. No additional exploration and testing along the proposed containment dike alignments is necessary at this time unless significant revisions to the location and alignments are made during final design and construction phases.

5.2 Borrow Excavations

As noted above, it is recommended that the eastern portion of the proposed borrow area (i.e., area encompassing Boring Nos. B-21 and B-24, be avoided 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. The water depth at the six borrow area boring location at the time of our field exploration was about 5 feet. Based on the soil shear strength profile characteristic of the western portion of the proposed borrow area, we would not envision significant borrow area slope stability issues provided the average cut slope is not steeper than 3H to 1V for dredge excavations in the range of 15 to 20 feet below the existing mudline.

SECTION 6. REFERENCES

Becker, D. E., Crooks, J. H. A., Been, K., and Jefferies, M. B. (1987), "Work as a Criterion for Determining In Situ and Yield Stresses in Clays", Canadian Geotechnical Journal, Vol. 24, No. 4, pp 549-564.

Casagrande, A. (1936), "The Determination of the Preconsolidation Load and its Practical Significance", Proceedings, First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Vol. 3, pp 60-64.

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Ladd, C. C. and Foott, R. (1974), "New Design Procedure for Stability of Soft Clays," ASCE, JGGE, Vol. 100, No. G57, pp 763-786.

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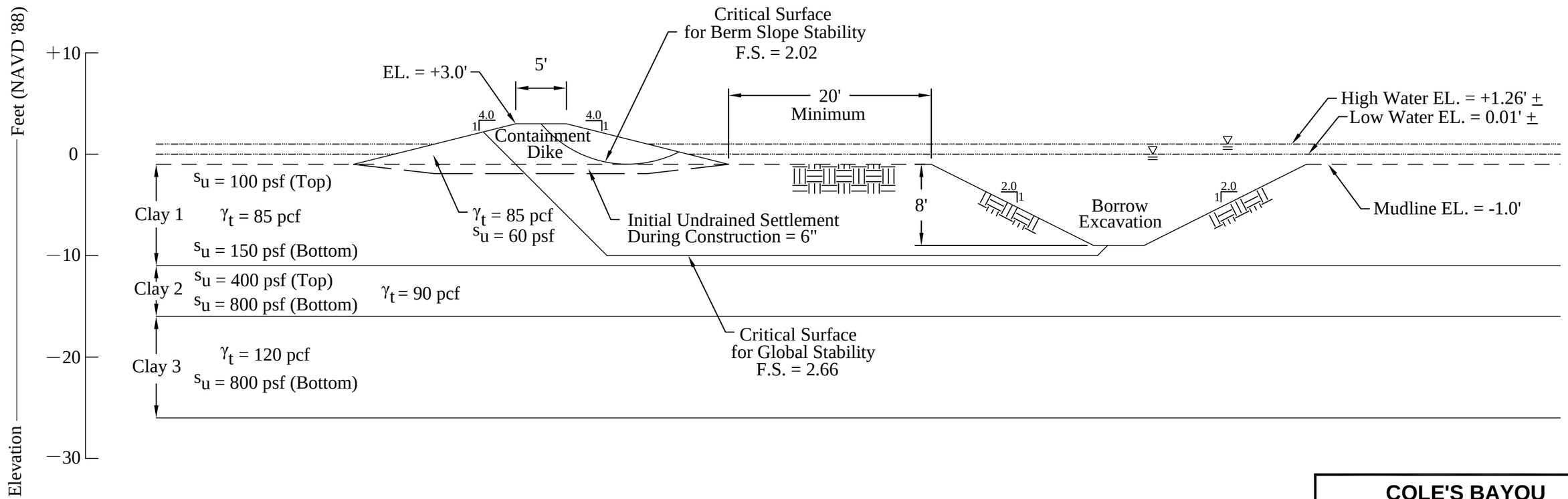
APPENDIX A. STABILITY ANALSYES

This Appendix contains the following:

- Figure A-1 – Typical Section A – Marsh Area 1 (Dike at Crest El. +3.0’)
- Figure A-2 – Typical Section A – March Area 1 (Marsh Construction Grade +2.0’)
- Figure A-3 – Typical Section A – Marsh Area 1 (Dike at Crest El. +4.0’)
- Figure A-4 – Typical Section A – March Area 1 (Marsh Construction Grade +3.0’)
- Figure A-5 – Typical Section B – Marsh Area 2 (Dike at Crest El. +3.0’)
- Figure A-6 – Typical Section B – March Area 2 (Marsh Construction Grade +2.0’)
- Figure A-7 – Typical Section B – Marsh Area 2 (Dike at Crest El. +4.0’)
- Figure A-8 – Typical Section B – Marsh Area 2 (Marsh Construction Grade +3.0’)
- Figure A-9 – Typical Section C – Marsh Area 3 (Marsh Construction Grade +2.0’)
- Figure A-10 – Typical Section C – Marsh Area 3 (Marsh Construction Grade +3.0’)



Typical Section A - Marsh Area No. 1

Scale: 1" = 10'

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VERMILLION PARISH, LOUISIANA

for

COASTAL PROTECTION AND RESTORATION AUTHORITY
BATON ROUGE, LOUISIANA

AAI
Ardaman & Associates, Inc.
Baton Rouge, LA Jefferson, LA

Project Engineer:	Drawn by:	Checked by:
R. WERNER	NMA	MGB
File No.: 13-80-3705	Date: 03-28-14	Figure No.: A-1

Title:
TYPICAL SECTION A - MARSH AREA NO. 1

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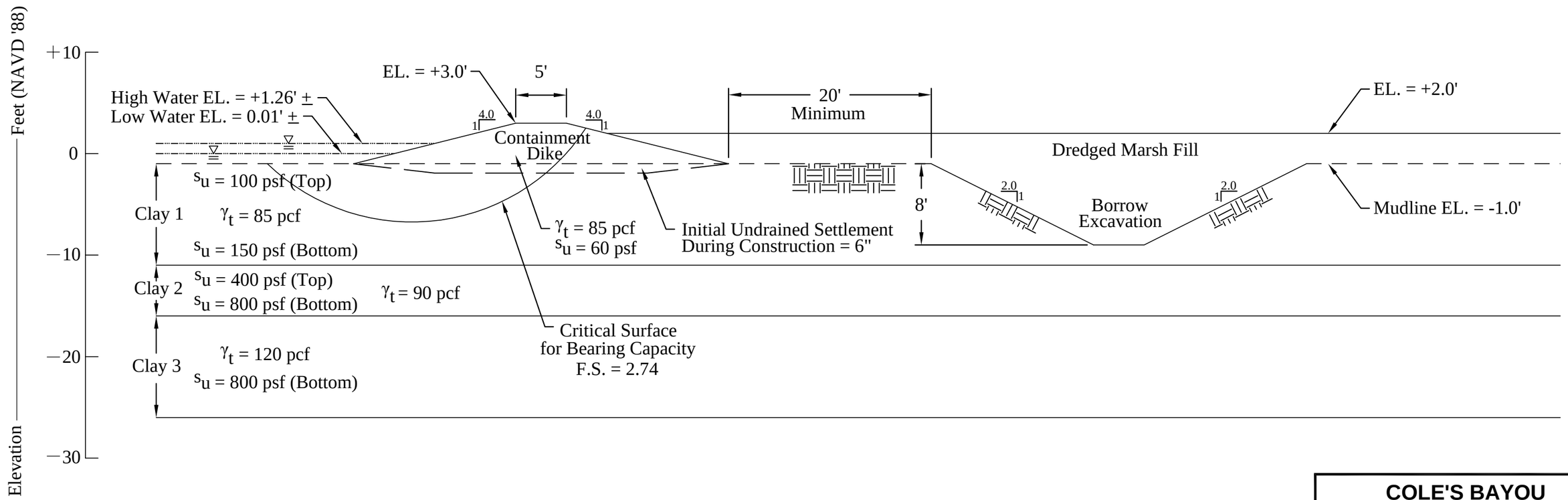

Typical Section A - Marsh Area No. 1

Scale: 1" = 10'

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Title:
TYPICAL SECTION A - MARSH AREA NO. 1

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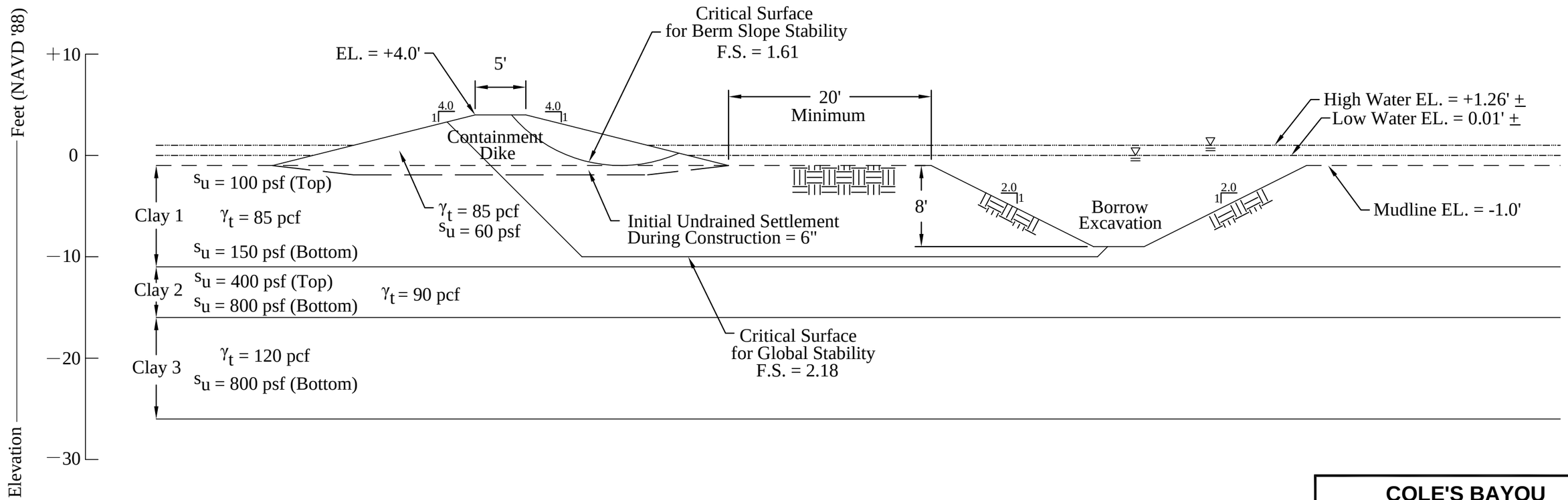

Typical Section A - Marsh Area No. 1

Scale: 1" = 10'

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Title:
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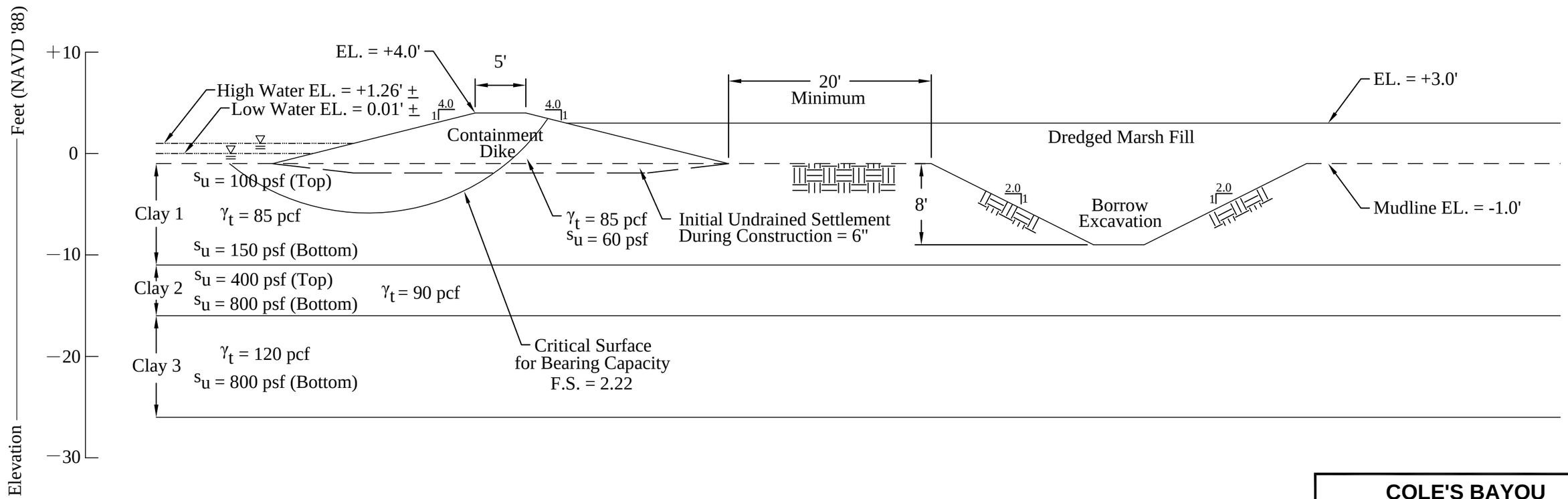

Typical Section A - Marsh Area No. 1

Scale: 1" = 10'

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Title:
TYPICAL SECTION A - MARSH AREA NO. 1

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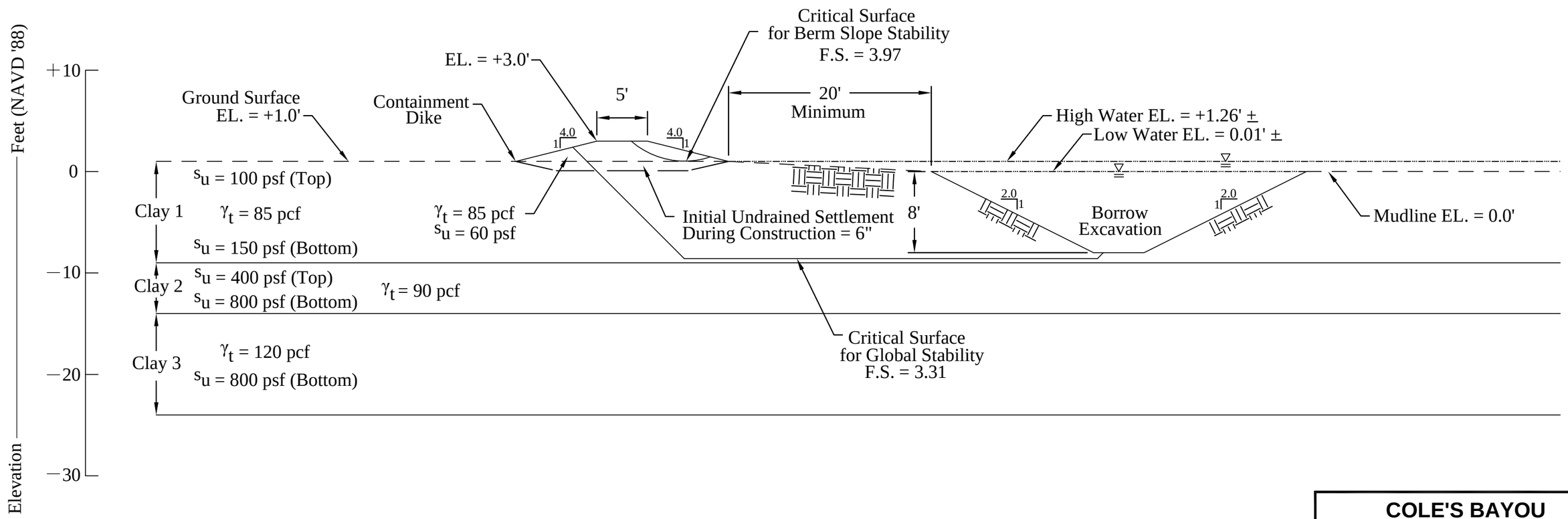

Typical Section A - Marsh Area No. 2

Scale: 1" = 10'

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Title:
TYPICAL SECTION A - MARSH AREA NO. 2

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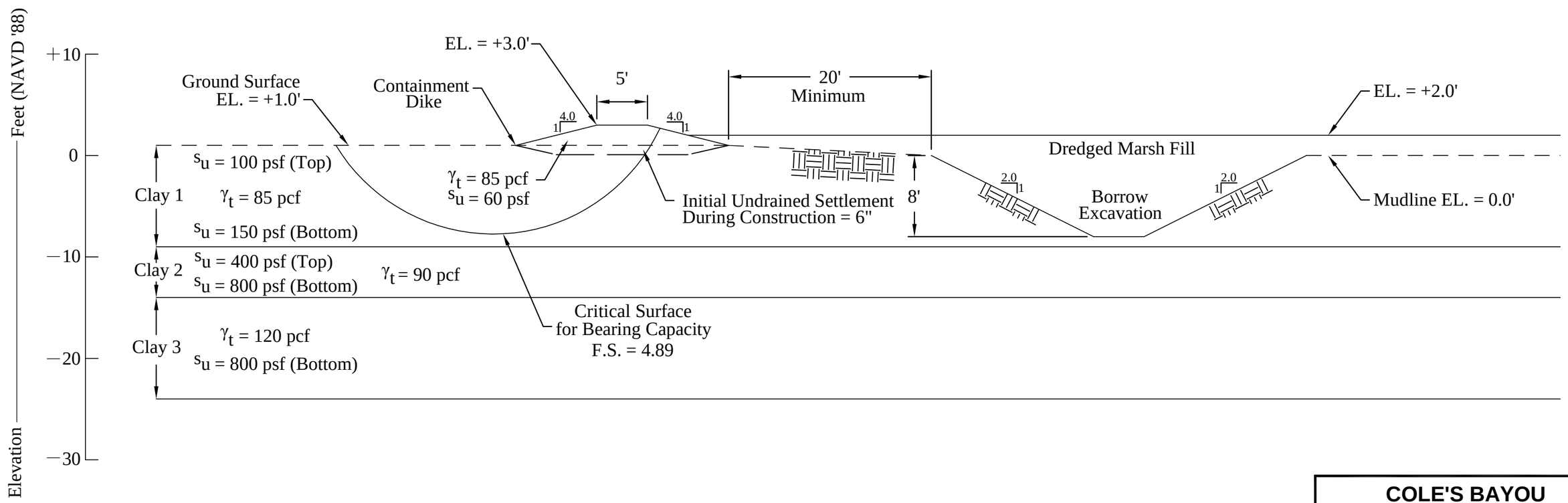

Typical Section A - Marsh Area No. 2

Scale: 1" = 10'

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Title:
TYPICAL SECTION A - MARSH AREA NO. 2

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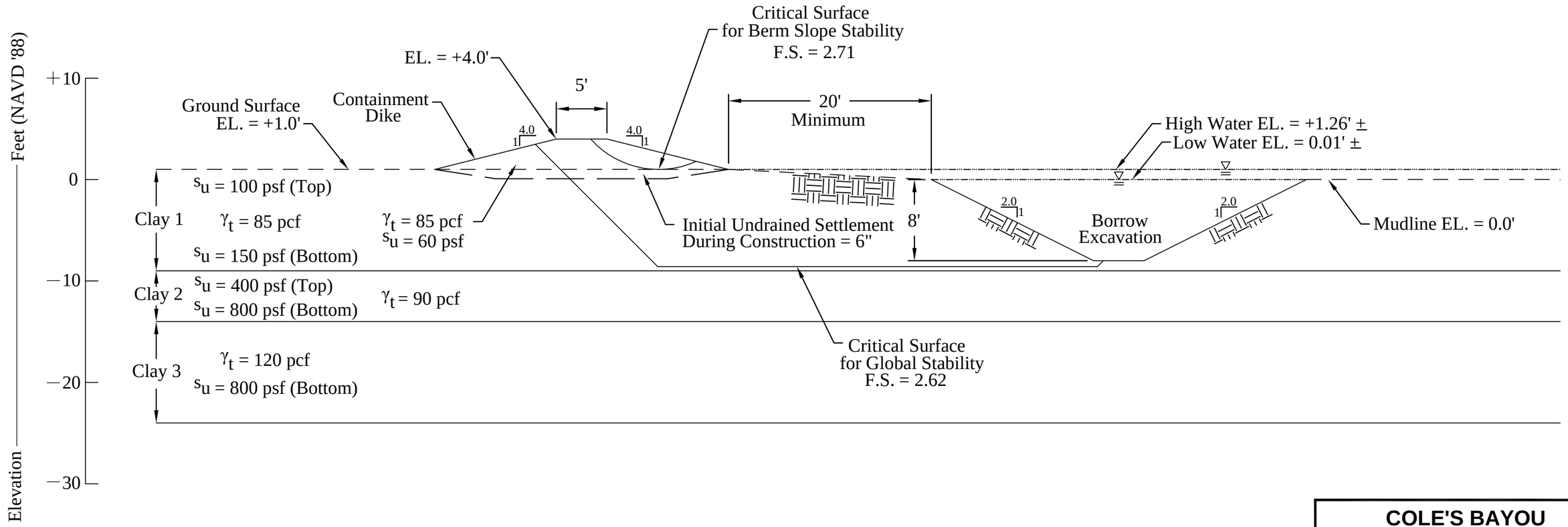

Typical Section A - Marsh Area No. 2

Scale: 1" = 10'

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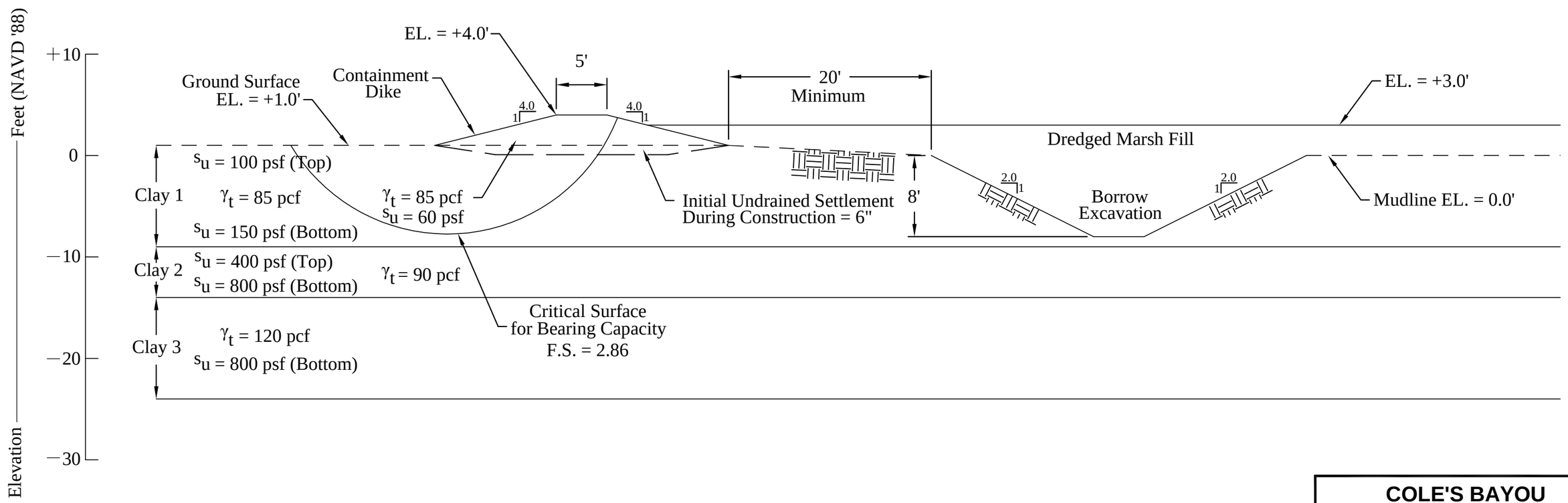

Typical Section A - Marsh Area No. 2

Scale: 1" = 10'

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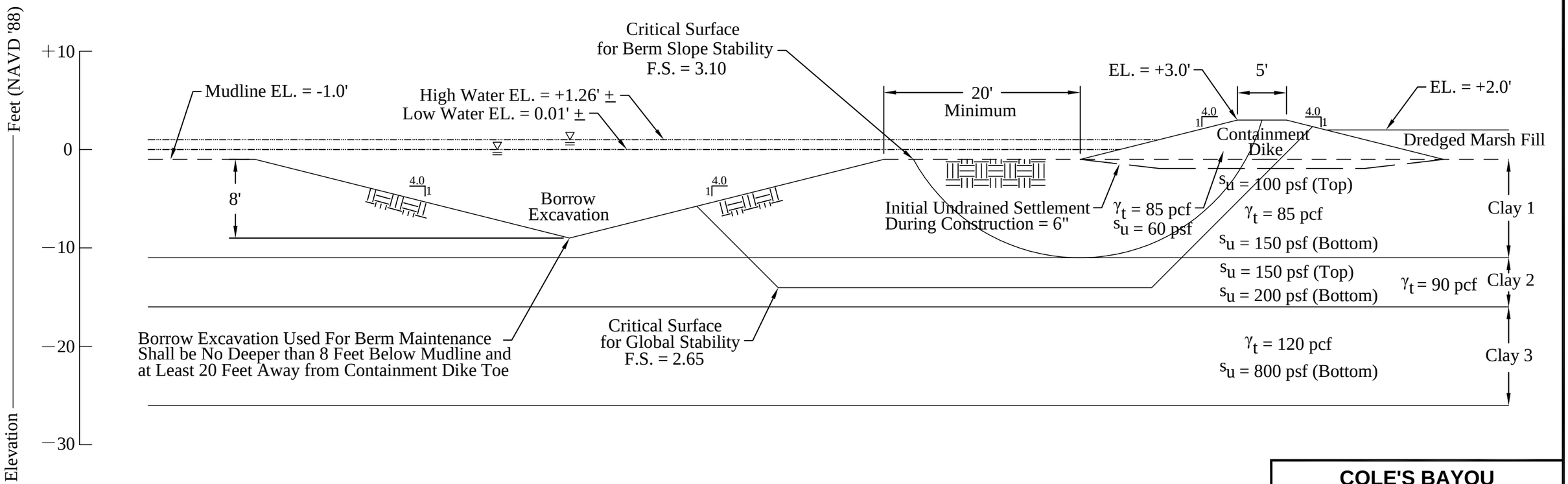

Typical Section A - Marsh Area No. 3

Scale: 1" = 10'

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Title:
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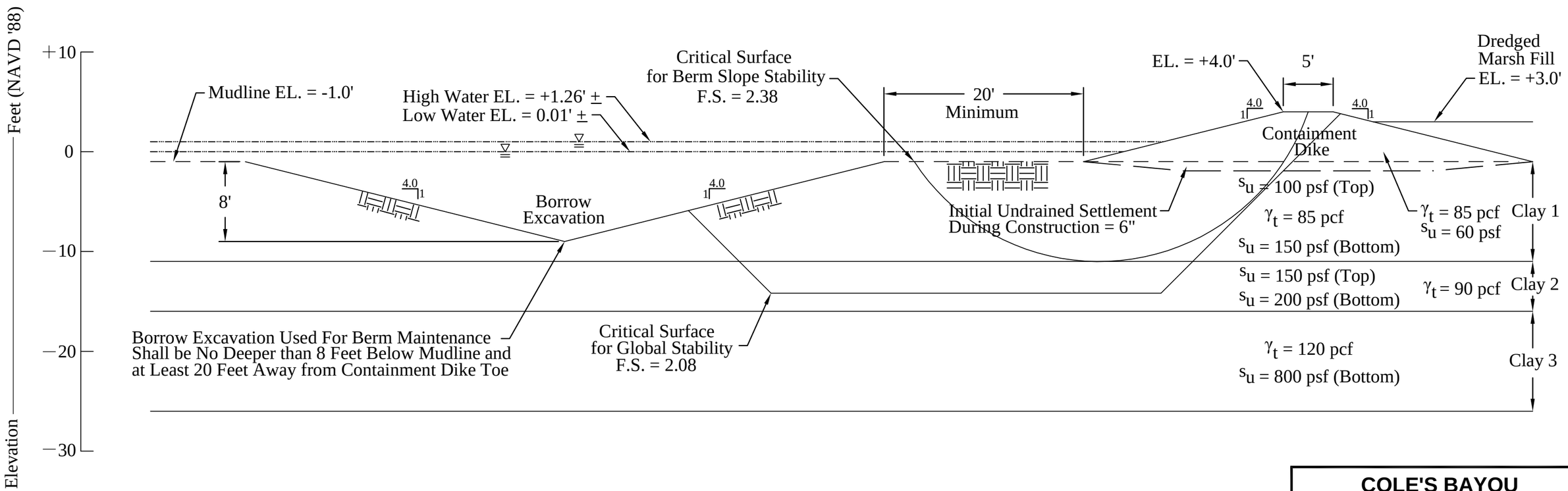

Typical Section A - Marsh Area No. 3

Scale: 1" = 10'

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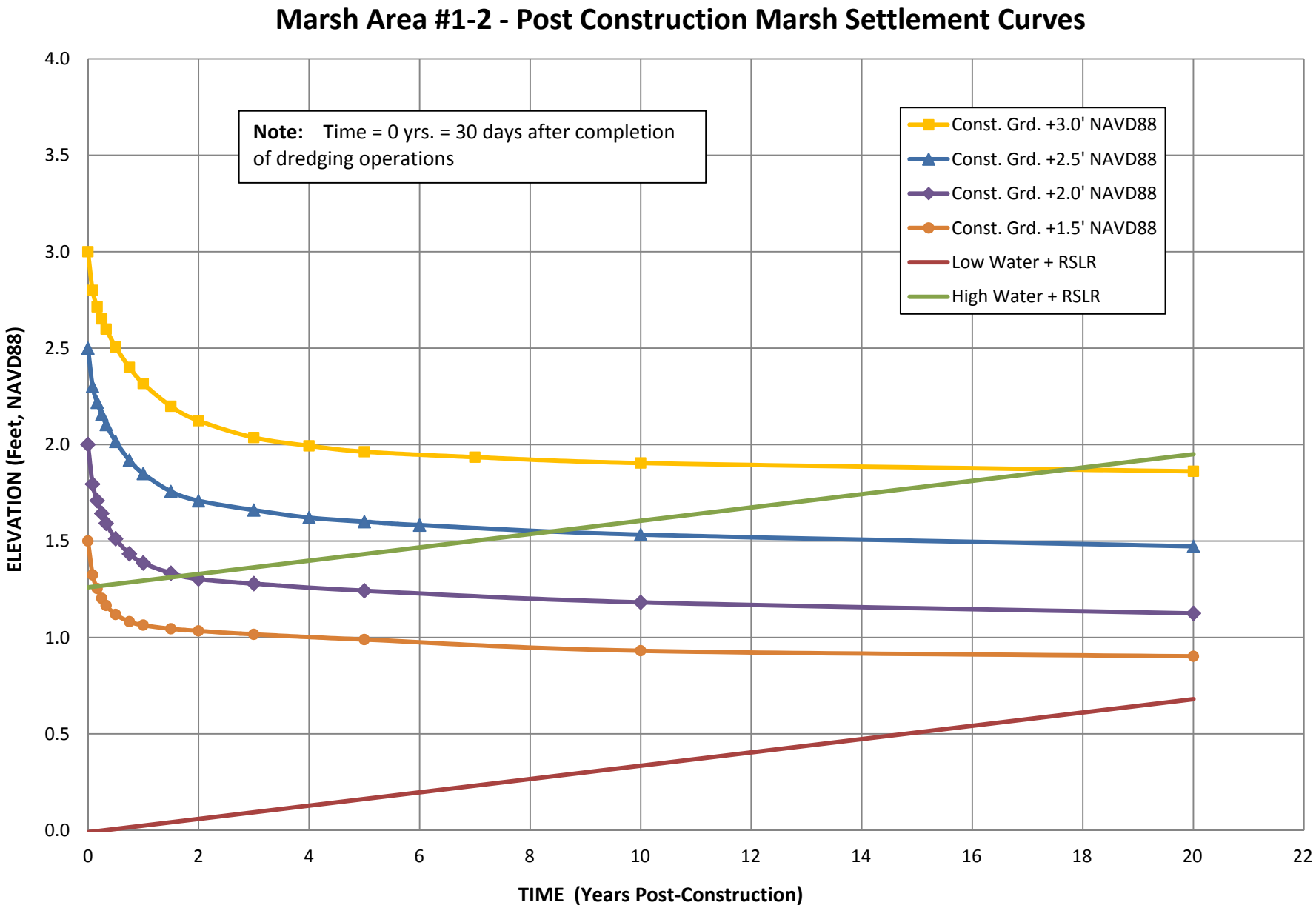
APPENDIX B. CALCULATIONS

This Appendix contains the following:

- Figures B-1 through B-4 – Settlement-Time Rate Curves – Marsh Areas 1-2
- Figure B-5 through B-8 – Settlement-Time Rate Curves – Marsh Area 3
- Tables B-1 through B-3 – Settlement-Time Rate – Marsh Areas 1-2
- Tables B-4 through B-6 – Settlement-Time Rate – Marsh Area 3
- Tables B-7 through B-12 – Containment Dike Settlement – Marsh Areas 1-2
- Tables B-13-B-15 – Marsh Interior Foundation Settlement – Marsh Areas 1-2
- Tables B-16 through B-21 – Containment Dike Settlement – Marsh Area 3
- Tables B-22-B-24 – Marsh Interior Foundation Settlement – Marsh Area 3
- Initial Undrained Settlement Calculations
- Primary Consolidation Rate Calculations
- Dredge Fill Settlement Calculations
- Cut/Fill Volume Calculations


Marsh Area #1-2 - Post Construction Marsh Settlement Curves
Note: Time = 0 yrs. = 30 days after completion of dredging operations
Const. Grd. +3.0' NAVD88
Const. Grd. +2.5' NAVD88
Const. Grd. +2.0' NAVD88
Const. Grd. +1.5' NAVD88
Low Water + RSLR
High Water + RSLR
ELEVATION (Feet, NAVD88)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post-Construction)

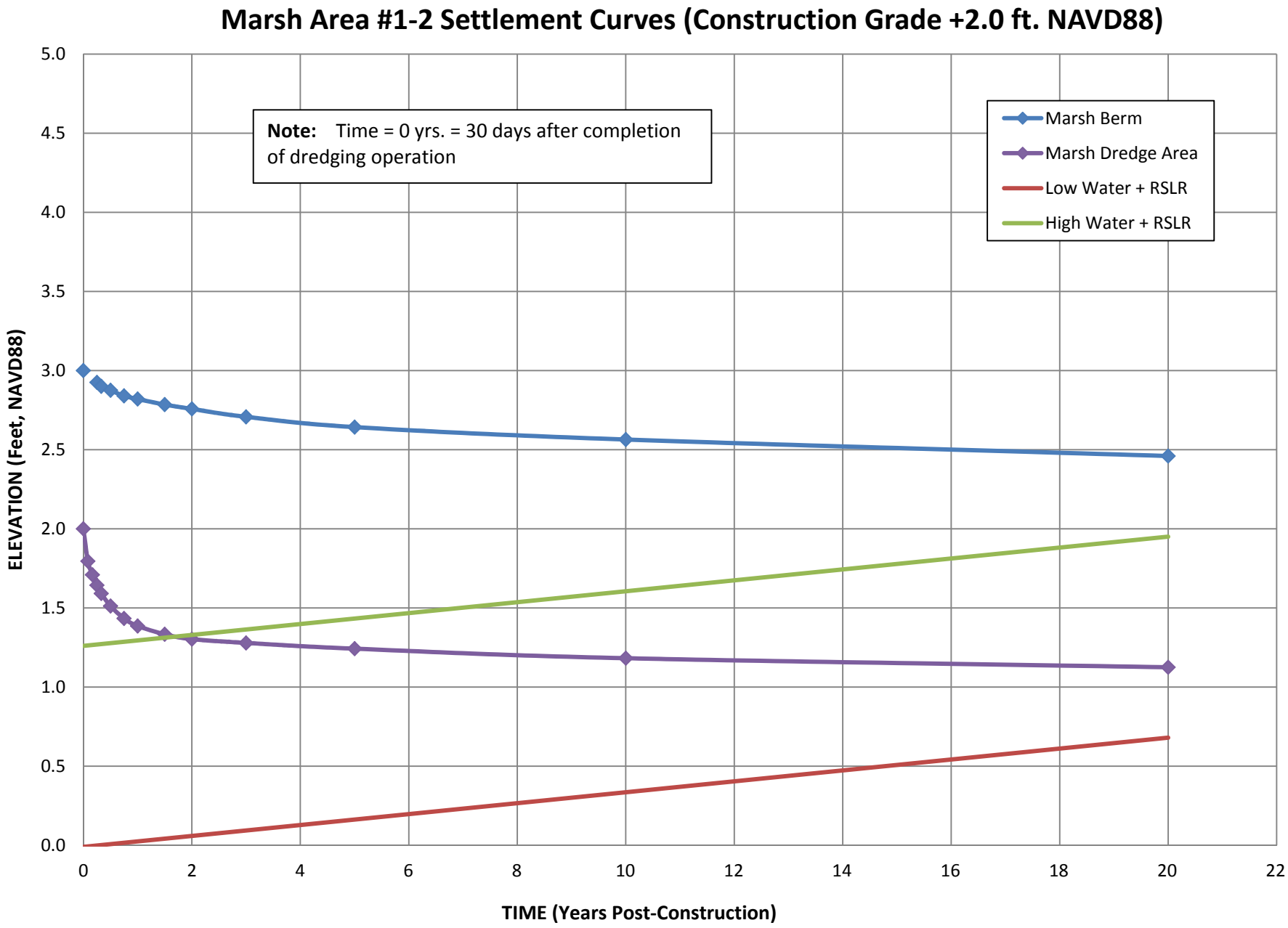
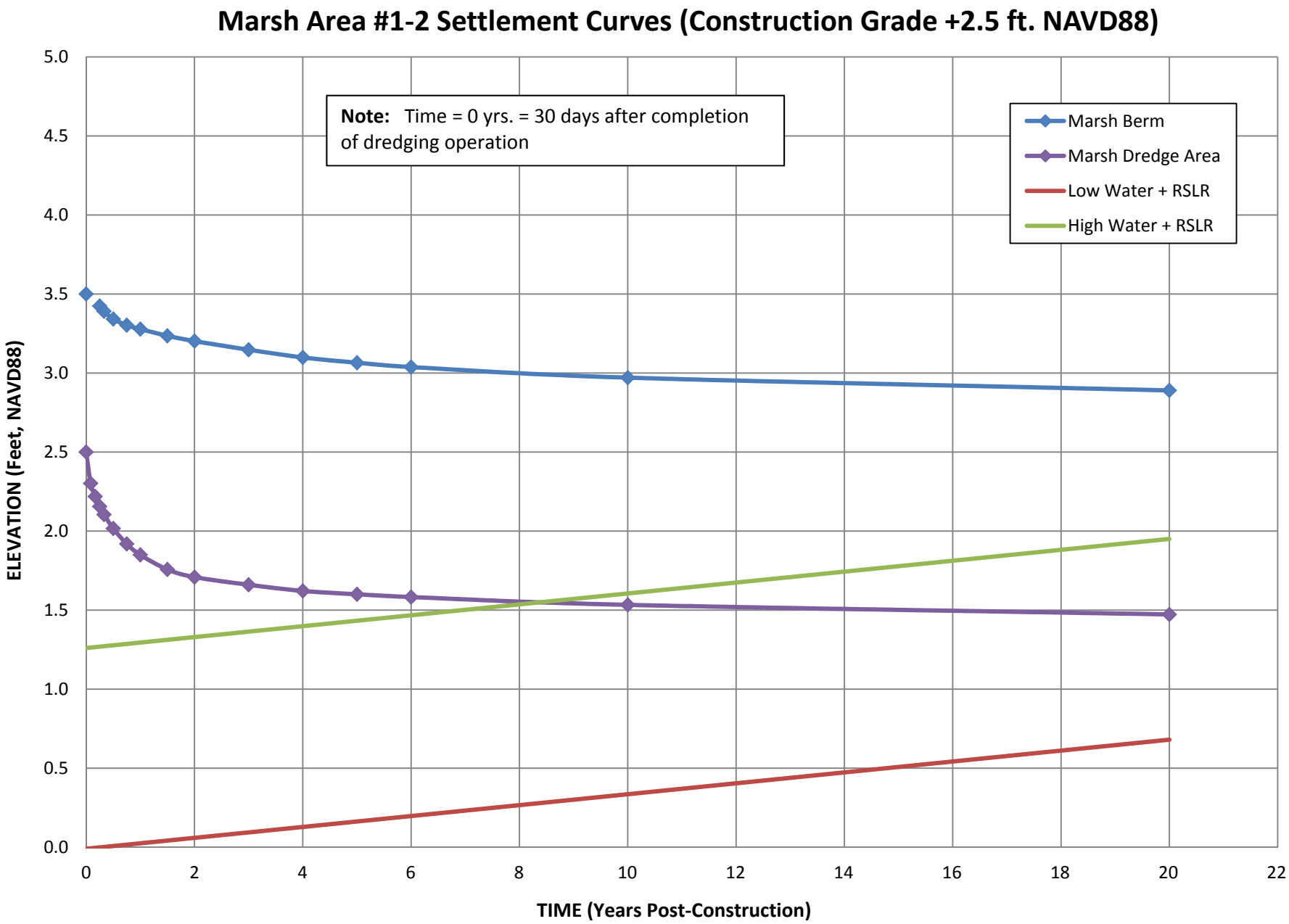

Marsh Area #1-2 Settlement Curves (Construction Grade +2.0 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water + RSLR
High Water + RSLR
ELEVATION (Feet, NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post-Construction)

Figure B-2


Marsh Area #1-2 Settlement Curves (Construction Grade +2.5 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water + RSLR
High Water + RSLR
ELEVATION (Feet, NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post-Construction)

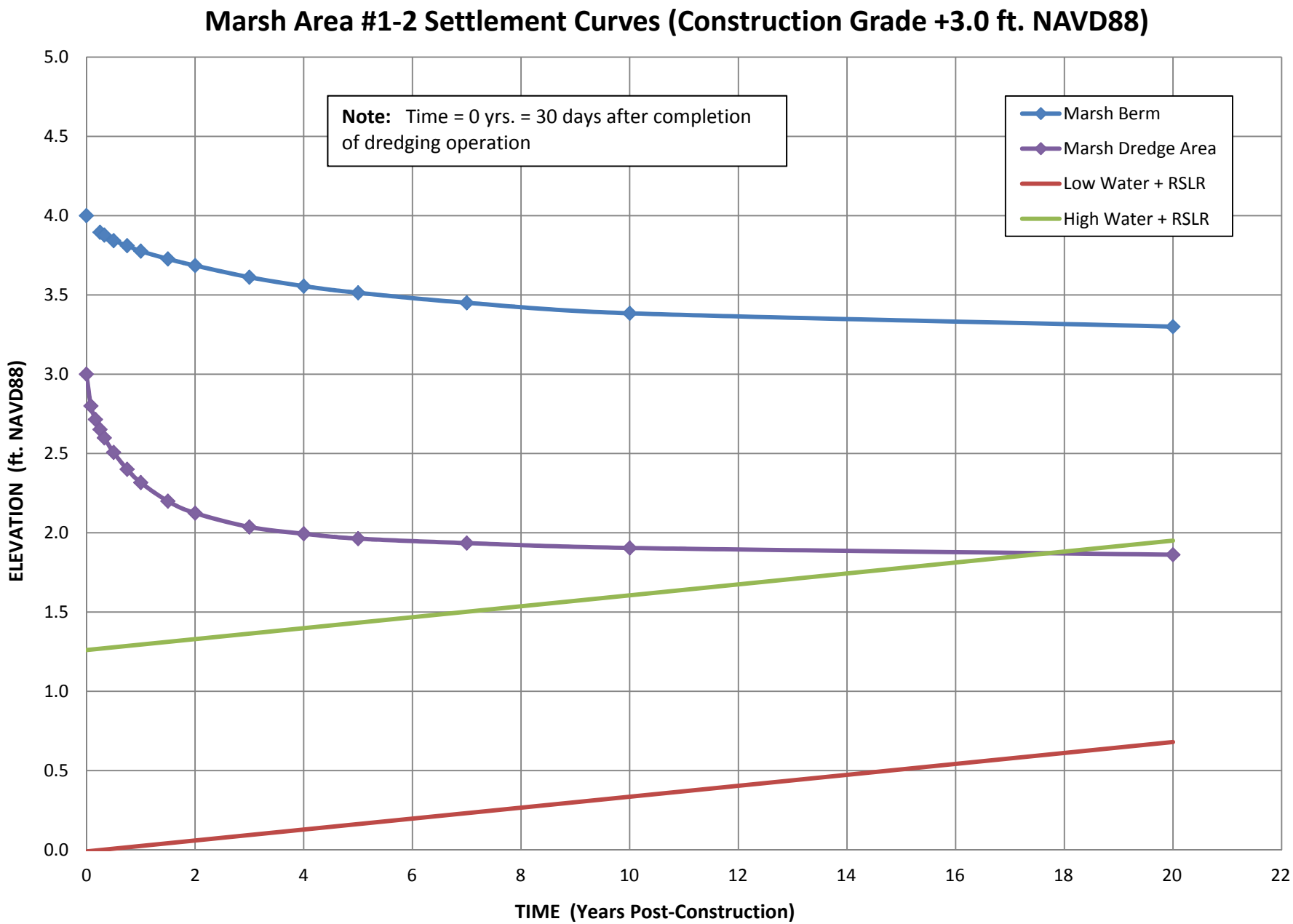
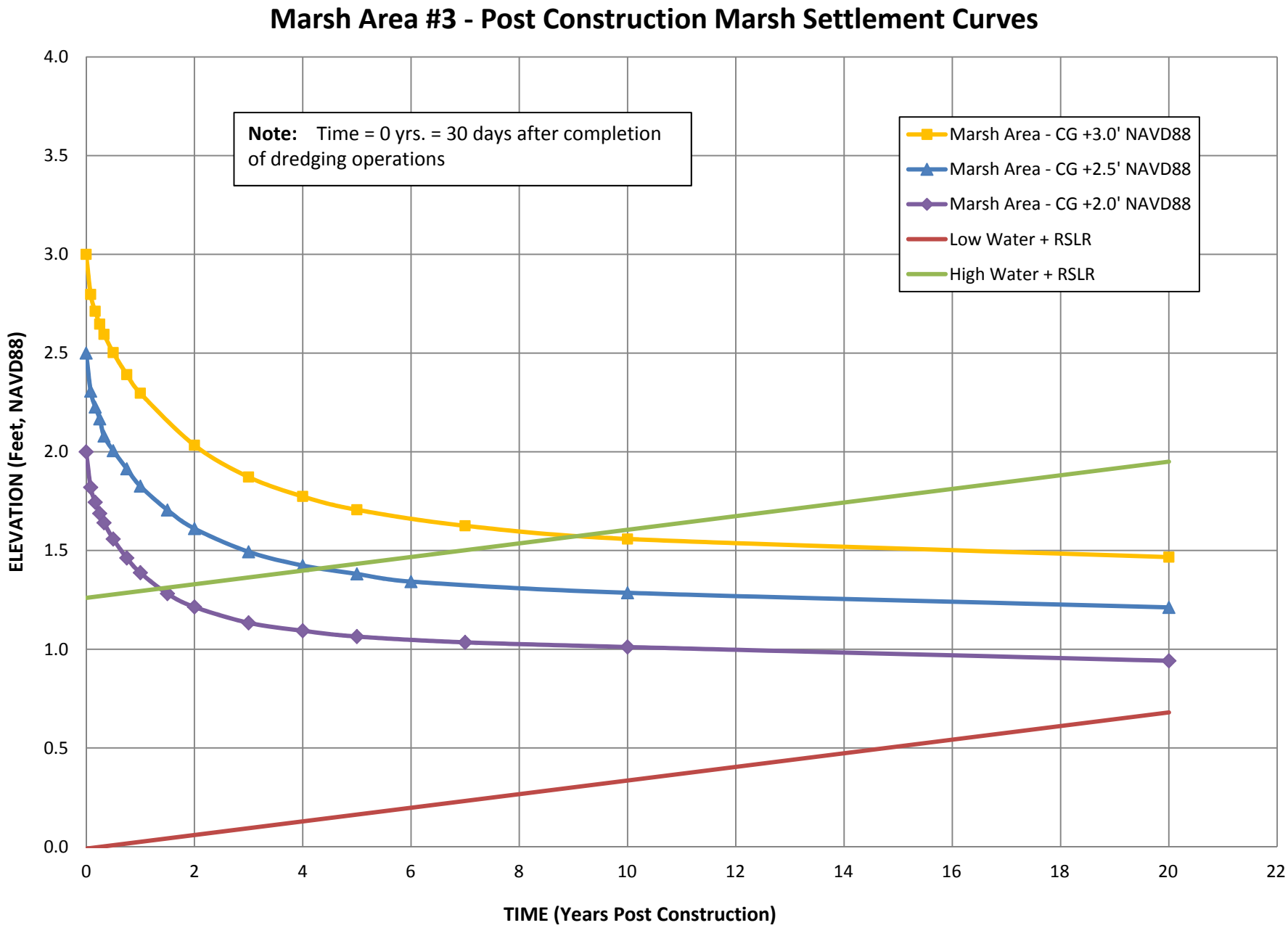
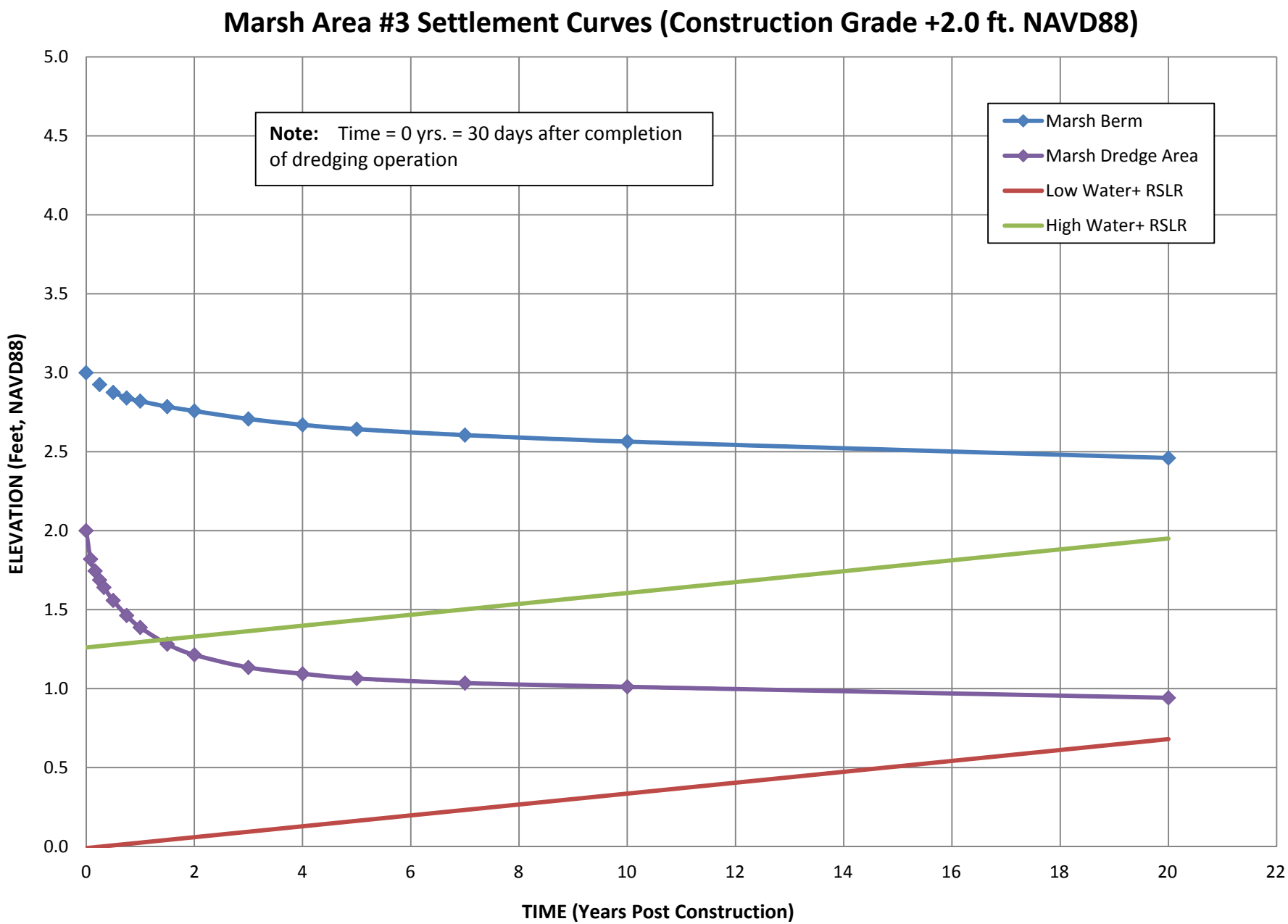
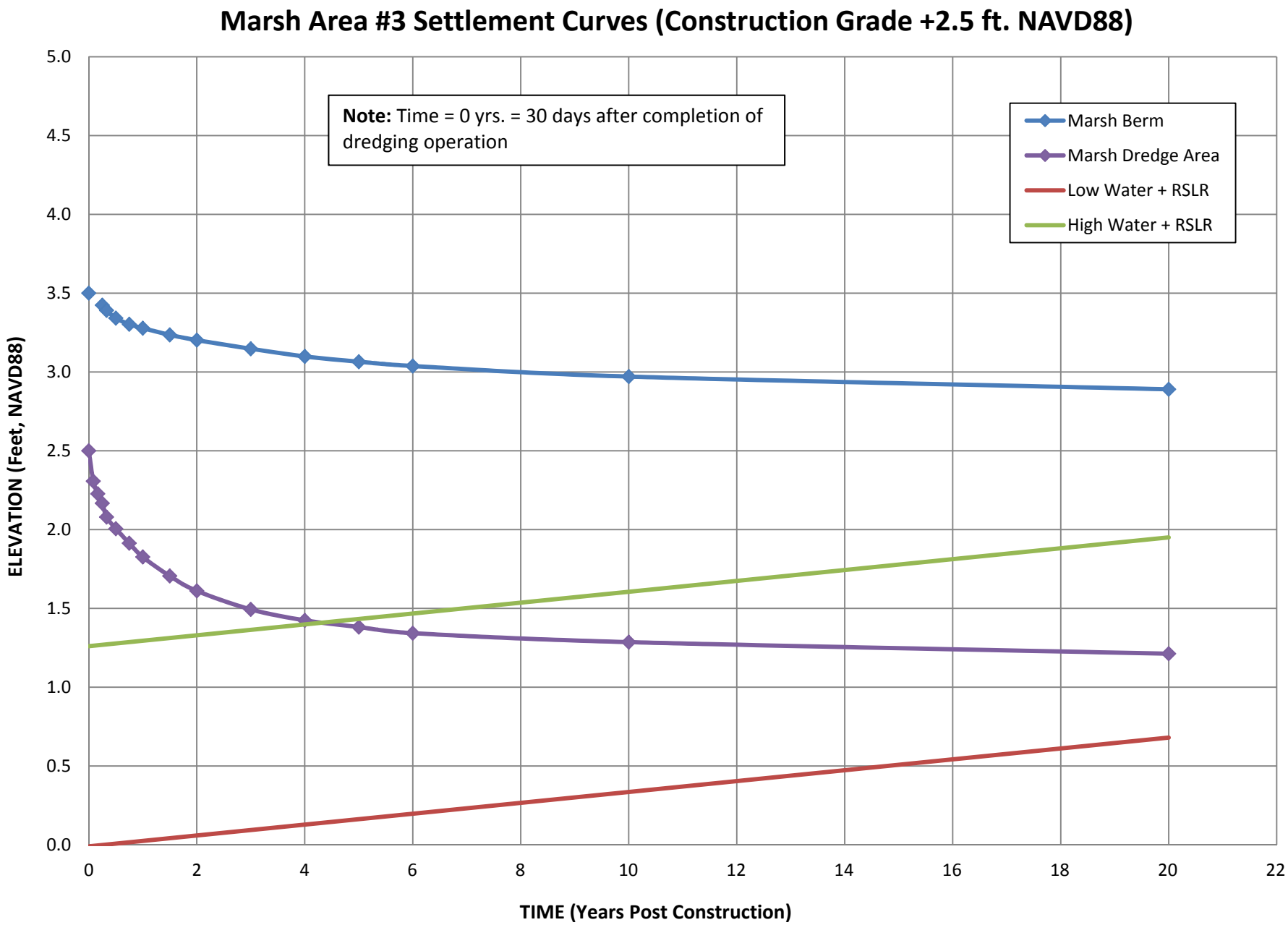
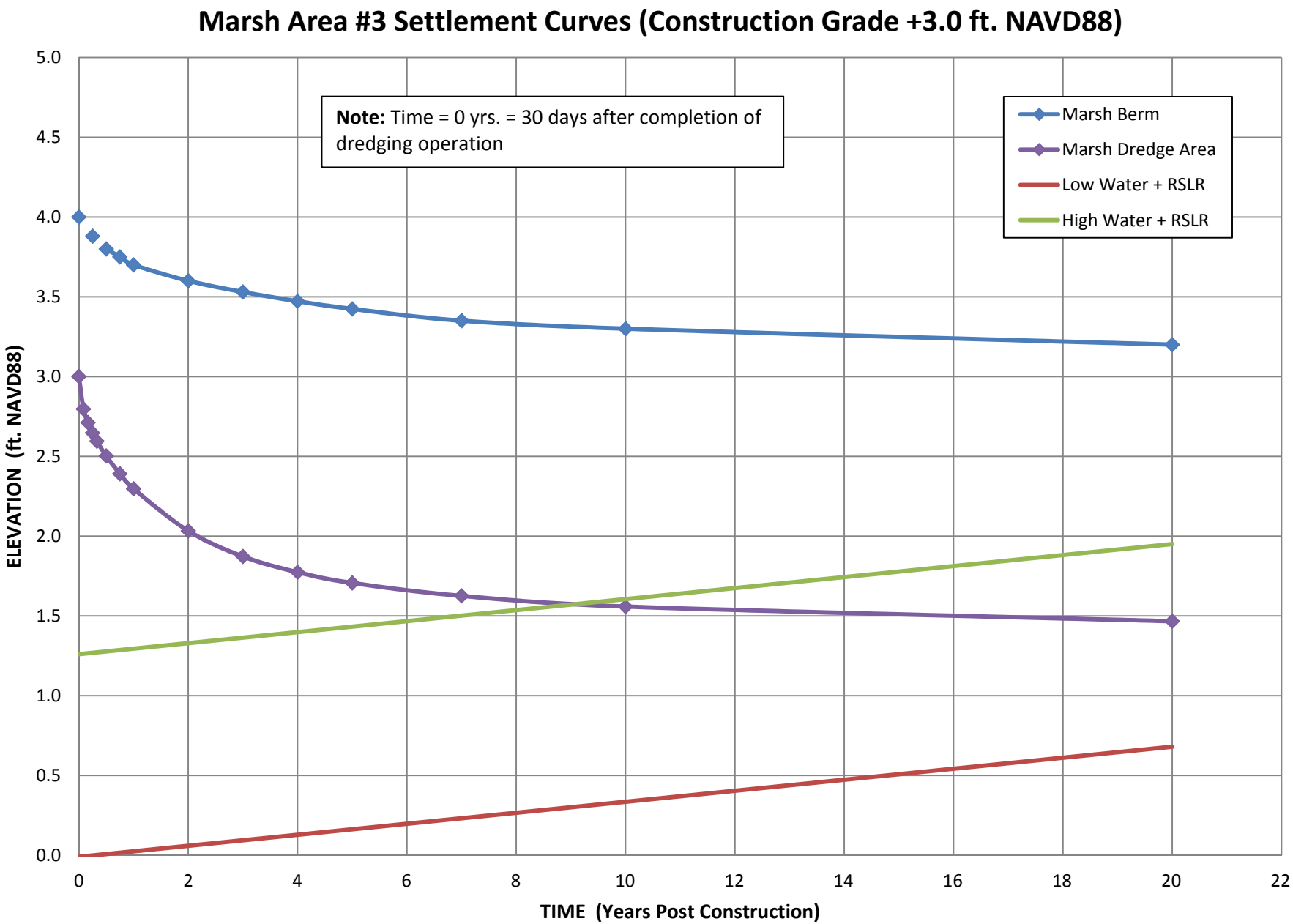

Marsh Area #1-2 Settlement Curves (Construction Grade +3.0 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water + RSLR
High Water + RSLR
ELEVATION (ft. NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post-Construction)

Figure B-4


Marsh Area #3 - Post Construction Marsh Settlement Curves
Note: Time = 0 yrs. = 30 days after completion of dredging operations
Marsh Area - CG +3.0' NAVD88
Marsh Area - CG +2.5' NAVD88
Marsh Area - CG +2.0' NAVD88
Low Water + RSLR
High Water + RSLR
ELEVATION (Feet, NAVD88)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post Construction)


Marsh Area #3 Settlement Curves (Construction Grade +2.0 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water+ RSLR
High Water+ RSLR
ELEVATION (Feet, NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post Construction)


Marsh Area #3 Settlement Curves (Construction Grade +2.5 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water + RSLR
High Water + RSLR
ELEVATION (Feet, NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post Construction)


Marsh Area #3 Settlement Curves (Construction Grade +3.0 ft. NAVD88)
Note: Time = 0 yrs. = 30 days after completion of dredging operation
Marsh Berm
Marsh Dredge Area
Low Water + RSLR
High Water + RSLR
ELEVATION (ft. NAVD88)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
2
4
6
8
10
12
14
16
18
20
22
TIME (Years Post Construction)

COLE'S BAYOU
MARSH AREAS #1 & #2
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 2.0

Marsh Berm Elevation (ft. NAVD88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Total Dike Settlement (ft)
3.000	2.000	0	0	0	0	0
	1.795	0.083	0.186	0.019	0.205	
	1.710	0.167	0.264	0.026	0.290	
2.925	1.644	0.25	0.324	0.033	0.357	0.075
2.900	1.592	0.33	0.371	0.038	0.409	0.100
2.875	1.512	0.5	0.442	0.046	0.488	0.125
2.840	1.434	0.75	0.510	0.056	0.566	0.160
2.820	1.386	1	0.549	0.065	0.614	0.180
2.785	1.333	1.5	0.587	0.080	0.667	0.215
2.758	1.303	2	0.605	0.092	0.697	0.243
2.708	1.279	3	0.609	0.113	0.721	0.293
2.643	1.242	5	0.613	0.145	0.758	0.358
2.564	1.182	10	0.618	0.200	0.818	0.436
2.460	1.125	20	0.623	0.252	0.875	0.540

Confidential Information:
Privileged and Confidential
Work Product

COLE'S BAYOU
MARSH AREAS #1 & #2
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 2.5

Marsh Berm Elevation (ft. NAVD 88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Total Dike Settlement (ft)
3.500	2.500	0	0	0	0	0
	2.302	0.083	0.174	0.024	0.198	
	2.219	0.167	0.246	0.035	0.281	
3.424	2.156	0.25	0.302	0.042	0.344	0.076
3.391	2.104	0.33	0.347	0.049	0.396	0.110
3.342	2.017	0.5	0.424	0.060	0.484	0.158
3.302	1.919	0.75	0.508	0.074	0.581	0.198
3.278	1.850	1	0.566	0.085	0.651	0.222
3.235	1.757	1.5	0.635	0.108	0.743	0.265
3.202	1.709	2	0.671	0.120	0.791	0.298
3.147	1.660	3	0.693	0.147	0.840	0.353
3.099	1.621	4	0.710	0.169	0.879	0.402
3.065	1.600	5	0.712	0.188	0.900	0.435
3.038	1.583	6	0.714	0.204	0.918	0.462
2.971	1.533	10	0.719	0.248	0.967	0.529
2.890	1.473	20	0.725	0.302	1.027	0.610

Confidential Information:
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Work Product

COLE'S BAYOU
MARSH AREAS #1 & #2
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 3.0

Marsh Berm Elevation (ft. NAVD88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Total Dike Settlement (ft)
4.000	3.000	0	0	0	0	
	2.800	0.083	0.182	0.019	0.201	
	2.714	0.167	0.259	0.027	0.286	
3.895	2.651	0.250	0.316	0.033	0.349	0.105
3.878	2.599	0.330	0.363	0.038	0.401	0.123
3.843	2.507	0.500	0.447	0.047	0.494	0.158
3.811	2.400	0.750	0.543	0.057	0.600	0.190
3.776	2.316	1.000	0.618	0.066	0.684	0.224
3.727	2.199	1.500	0.721	0.081	0.801	0.273
3.685	2.124	2.000	0.783	0.093	0.876	0.315
3.612	2.036	3.000	0.850	0.114	0.964	0.389
3.556	1.994	4.000	0.875	0.132	1.006	0.445
3.514	1.963	5.000	0.891	0.146	1.037	0.487
3.451	1.935	7.000	0.895	0.171	1.065	0.550
3.384	1.904	10.000	0.899	0.197	1.096	0.616
3.300	1.861	20.000	0.903	0.236	1.139	0.700

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COLE'S BAYOU
MARSH AREA #3
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 2.0

Marsh Berm Elevation (ft. NAVD88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Dike Settlement (ft)
3.000	2.000	0	0	0	0	
	1.820	0.083	0.162	0.019	0.180	
	1.744	0.167	0.229	0.026	0.256	
2.925	1.688	0.25	0.280	0.033	0.313	0.075
	1.640	0.33	0.322	0.038	0.360	0.100
2.875	1.558	0.5	0.396	0.046	0.442	0.125
2.840	1.462	0.75	0.481	0.056	0.538	0.160
2.820	1.387	1	0.548	0.065	0.613	0.180
2.785	1.282	1.5	0.639	0.080	0.718	0.215
2.758	1.214	2	0.694	0.092	0.786	0.243
2.708	1.134	3	0.754	0.113	0.866	0.293
2.670	1.094	4	0.776	0.130	0.906	0.330
2.643	1.064	5	0.791	0.145	0.936	0.358
2.605	1.035	7	0.795	0.171	0.965	0.395
2.564	1.011	10	0.789	0.200	0.989	0.436
2.460	0.941	20	0.807	0.252	1.059	0.540

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COLE'S BAYOU
MARSH AREA #3
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 2.5

Marsh Berm Elevation (ft. NAVD88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Total Dike Settlement (ft)
3.500	2.500	0	0	0	0	
	2.307	0.083	0.171	0.022	0.193	
	2.227	0.167	0.242	0.031	0.273	
3.424	2.167	0.25	0.295	0.038	0.333	0.076
3.391	2.080	0.33	0.376	0.045	0.421	0.110
3.342	2.006	0.5	0.440	0.054	0.494	0.158
3.302	1.914	0.75	0.520	0.066	0.586	0.198
3.278	1.826	1	0.586	0.088	0.674	0.222
3.235	1.705	1.5	0.701	0.094	0.795	0.265
3.202	1.611	2	0.781	0.109	0.889	0.298
3.147	1.494	3	0.874	0.133	1.007	0.353
3.099	1.424	4	0.923	0.153	1.076	0.402
3.065	1.382	5	0.947	0.172	1.119	0.435
3.038	1.343	6	0.970	0.188	1.158	0.462
2.971	1.286	10	0.977	0.238	1.214	0.529
2.890	1.212	20	0.986	0.302	1.288	0.610

Confidential Information:
Privileged and Confidential
Work Product

COLE'S BAYOU
MARSH AREA #3
MARSH CONSTRUCTION GRADE (CG) EL., FT NAVD88: 3.0

Marsh Berm Elevation (ft. NAVD88)	Marsh Area Elevation (ft. NAVD88)	Years Post-Construction	Marsh Fill Settlement (ft)	Marsh Nat. Grd. Settlement (ft)	Marsh Total Settlement (ft)	Total Dike Settlement (ft)
4.000	3.000	0	0	0	0	
	2.797	0.083	0.177	0.027	0.203	
	2.712	0.167	0.251	0.038	0.288	
3.880	2.647	0.25	0.307	0.047	0.353	0.120
	2.595	0.33	0.352	0.053	0.406	
3.800	2.502	0.5	0.432	0.066	0.498	0.200
3.750	2.391	0.75	0.529	0.080	0.609	0.250
3.700	2.297	1	0.611	0.092	0.703	0.300
3.600	2.033	2	0.836	0.131	0.967	0.400
3.530	1.872	3	0.967	0.161	1.128	0.470
3.472	1.774	4	1.041	0.185	1.226	0.528
3.424	1.706	5	1.087	0.207	1.294	0.576
3.350	1.625	7	1.131	0.244	1.375	0.650
3.300	1.558	10	1.154	0.287	1.442	0.700
3.200	1.467	20	1.165	0.368	1.534	0.800

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Work Product

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.071	0.092	0.40	0.3	0.04	0.128
5	7.5	2.5	0.046	0.700	0.055	0.101	0.20	0.3	0.04	0.034
7.5	10	2.5	0.118	0.650	0.051	0.169	0.20	0.3	0.04	0.006
10	12.5	2.5	0.157	0.590	0.047	0.204	3.00	0.3	0.04	0.011
12.5	15	2.5	0.228	0.550	0.043	0.271	3.00	0.3	0.04	0.008
15	20	5	0.335	0.500	0.040	0.375	3.00	0.18	0.03	0.007
20	25	5	0.477	0.400	0.032	0.509	2.80	0.18	0.03	0.004
25	30	5	0.618	0.350	0.028	0.646	2.60	0.18	0.03	0.003
30	35	5	0.761	0.295	0.023	0.784	2.30	0.18	0.03	0.002
35	40	5	0.902	0.260	0.021	0.923	2.20	0.18	0.03	0.001
40	45	5	1.045	0.240	0.019	1.064	2.10	0.18	0.03	0.001
45	50	5	1.187	0.210	0.017	1.204	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				2.49

Total Marsh Creep (in)	3.2
Terrace Berm Fill Shrinkage (in)	1.9
Total Terrace Berm Creep (in)	0.7

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**6.4**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.088	0.109	0.40	0.3	0.04	0.143
5	7.5	2.5	0.046	0.700	0.069	0.115	0.20	0.3	0.04	0.040
7.5	10	2.5	0.118	0.650	0.064	0.182	0.20	0.3	0.04	0.007
10	12.5	2.5	0.157	0.590	0.058	0.215	3.00	0.3	0.04	0.014
12.5	15	2.5	0.228	0.550	0.054	0.282	3.00	0.3	0.04	0.009
15	20	5	0.335	0.500	0.049	0.384	3.00	0.18	0.03	0.009
20	25	5	0.477	0.400	0.039	0.516	2.80	0.18	0.03	0.005
25	30	5	0.618	0.350	0.034	0.652	2.60	0.18	0.03	0.004
30	35	5	0.761	0.295	0.029	0.790	2.30	0.18	0.03	0.002
35	40	5	0.902	0.260	0.025	0.927	2.20	0.18	0.03	0.002
40	45	5	1.045	0.240	0.024	1.069	2.10	0.18	0.03	0.001
45	50	5	1.187	0.210	0.021	1.208	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				2.85

Total Marsh Creep (in)	3.2
Terrace Berm Fill Shrinkage (in)	2.4
Total Terrace Berm Creep (in)	0.8

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**6.9**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.113	0.134	0.40	0.3	0.04	0.161
5	7.5	2.5	0.046	0.700	0.088	0.134	0.20	0.3	0.04	0.046
7.5	10	2.5	0.118	0.650	0.081	0.199	0.20	0.3	0.04	0.009
10	12.5	2.5	0.157	0.590	0.074	0.231	3.00	0.3	0.04	0.017
12.5	15	2.5	0.228	0.550	0.069	0.297	3.00	0.3	0.04	0.011
15	20	5	0.335	0.500	0.063	0.398	3.00	0.18	0.03	0.011
20	25	5	0.477	0.400	0.050	0.527	2.80	0.18	0.03	0.006
25	30	5	0.618	0.350	0.044	0.662	2.60	0.18	0.03	0.004
30	35	5	0.761	0.295	0.037	0.798	2.30	0.18	0.03	0.003
35	40	5	0.902	0.260	0.033	0.935	2.20	0.18	0.03	0.002
40	45	5	1.045	0.240	0.030	1.075	2.10	0.18	0.03	0.002
45	50	5	1.187	0.210	0.026	1.213	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				3.30

Total Marsh Creep (in)	3.2
Terrace Berm Fill Shrinkage (in)	2.9
Total Terrace Berm Creep (in)	1.0

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**7.5**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.074	0.095	0.40	0.3	0.04	0.131
5	7.5	2.5	0.046	0.870	0.069	0.115	0.20	0.3	0.04	0.040
7.5	10	2.5	0.118	0.850	0.067	0.185	0.20	0.3	0.04	0.008
10	12.5	2.5	0.157	0.810	0.064	0.221	3.00	0.3	0.04	0.015
12.5	15	2.5	0.228	0.770	0.061	0.289	3.00	0.3	0.04	0.010
15	20	5	0.335	0.750	0.059	0.394	3.00	0.18	0.03	0.011
20	25	5	0.477	0.710	0.056	0.533	2.80	0.18	0.03	0.007
25	30	5	0.618	0.680	0.054	0.672	2.60	0.18	0.03	0.005
30	35	5	0.761	0.635	0.050	0.811	2.30	0.18	0.03	0.004
35	40	5	0.902	0.620	0.049	0.951	2.20	0.18	0.03	0.003
40	45	5	1.045	0.600	0.047	1.092	2.10	0.18	0.03	0.003
45	50	5	1.187	0.560	0.044	1.231	1.90	0.18	0.03	0.002
						Total Primary Consolidation Settlement (in)				2.88

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	1.9
Total Terrace Berm Creep (in)	0.7

Total 20-year Dike Settlement (in)	**6.5**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.092	0.113	0.40	0.3	0.04	0.146
5	7.5	2.5	0.046	0.870	0.085	0.131	0.20	0.3	0.04	0.046
7.5	10	2.5	0.118	0.850	0.083	0.201	0.20	0.3	0.04	0.009
10	12.5	2.5	0.157	0.810	0.079	0.236	3.00	0.3	0.04	0.018
12.5	15	2.5	0.228	0.770	0.075	0.303	3.00	0.3	0.04	0.012
15	20	5	0.335	0.750	0.074	0.409	3.00	0.18	0.03	0.013
20	25	5	0.477	0.710	0.070	0.547	2.80	0.18	0.03	0.009
25	30	5	0.618	0.680	0.067	0.685	2.60	0.18	0.03	0.007
30	35	5	0.761	0.635	0.062	0.823	2.30	0.18	0.03	0.005
35	40	5	0.902	0.620	0.061	0.963	2.20	0.18	0.03	0.004
40	45	5	1.045	0.600	0.059	1.104	2.10	0.18	0.03	0.004
45	50	5	1.187	0.560	0.055	1.242	1.90	0.18	0.03	0.003
						Total Primary Consolidation Settlement (in)				3.31

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	2.4
Total Terrace Berm Creep (in)	0.8

Total 20-year Dike Settlement (in)	**7.5**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.113	0.134	0.40	0.3	0.04	0.161
5	7.5	2.5	0.046	0.870	0.104	0.150	0.20	0.3	0.04	0.051
7.5	10	2.5	0.118	0.850	0.102	0.220	0.20	0.3	0.04	0.022
10	12.5	2.5	0.157	0.810	0.097	0.254	3.00	0.3	0.04	0.021
12.5	15	2.5	0.228	0.770	0.092	0.320	3.00	0.3	0.04	0.015
15	20	5	0.335	0.750	0.090	0.425	3.00	0.18	0.03	0.016
20	25	5	0.477	0.710	0.085	0.562	2.80	0.18	0.03	0.011
25	30	5	0.618	0.680	0.082	0.700	2.60	0.18	0.03	0.008
30	35	5	0.761	0.635	0.076	0.837	2.30	0.18	0.03	0.006
35	40	5	0.902	0.620	0.074	0.976	2.20	0.18	0.03	0.005
40	45	5	1.045	0.600	0.072	1.117	2.10	0.18	0.03	0.004
45	50	5	1.187	0.560	0.067	1.254	1.90	0.18	0.03	0.004
						Total Primary Consolidation Settlement (in)				3.88

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	2.9
Terrace Berm Fill Creep (in)	1.0

Total 20-year Dike Settlement (in)	**8.8**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.067	0.088	0.40	0.3	0.04	0.124
5	7.5	2.5	0.046	1.000	0.067	0.113	0.20	0.3	0.04	0.039
7.5	10	2.5	0.118	1.000	0.067	0.185	0.20	0.3	0.04	0.008
10	12.5	2.5	0.157	1.000	0.067	0.224	3.00	0.3	0.04	0.015
12.5	15	2.5	0.228	1.000	0.067	0.295	3.00	0.3	0.04	0.011
15	20	5	0.335	1.000	0.067	0.402	3.00	0.18	0.03	0.012
20	25	5	0.477	1.000	0.067	0.544	2.80	0.18	0.03	0.009
25	30	5	0.618	1.000	0.067	0.685	2.60	0.18	0.03	0.007
30	35	5	0.761	1.000	0.067	0.828	2.30	0.18	0.03	0.005
35	40	5	0.902	1.000	0.067	0.969	2.20	0.18	0.03	0.005
40	45	5	1.045	1.000	0.067	1.112	2.10	0.18	0.03	0.004
45	50	5	1.187	1.000	0.067	1.254	1.90	0.18	0.03	0.004
						Total Primary Consolidation Settlement (in)				2.91

Total Marsh Creep (in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**3.9**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.088	0.109	0.40	0.3	0.04	0.143
5	7.5	2.5	0.046	1.000	0.088	0.134	0.20	0.3	0.04	0.046
7.5	10	2.5	0.118	1.000	0.088	0.206	0.20	0.3	0.04	0.013
10	12.5	2.5	0.157	1.000	0.088	0.245	3.00	0.3	0.04	0.019
12.5	15	2.5	0.228	1.000	0.088	0.316	3.00	0.3	0.04	0.014
15	20	5	0.335	1.000	0.088	0.423	3.00	0.18	0.03	0.015
20	25	5	0.477	1.000	0.088	0.565	2.80	0.18	0.03	0.011
25	30	5	0.618	1.000	0.088	0.706	2.60	0.18	0.03	0.009
30	35	5	0.761	1.000	0.088	0.849	2.30	0.18	0.03	0.007
35	40	5	0.902	1.000	0.088	0.990	2.20	0.18	0.03	0.006
40	45	5	1.045	1.000	0.088	1.133	2.10	0.18	0.03	0.005
45	50	5	1.187	1.000	0.088	1.275	1.90	0.18	0.03	0.005
						Total Primary Consolidation Settlement (in)				3.53

Secondary Compression (Creep; in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**4.5**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.108	0.129	0.40	0.3	0.04	0.158
5	7.5	2.5	0.046	1.000	0.108	0.154	0.20	0.3	0.04	0.052
7.5	10	2.5	0.118	1.000	0.108	0.226	0.20	0.3	0.04	0.025
10	12.5	2.5	0.157	1.000	0.108	0.265	3.00	0.3	0.04	0.023
12.5	15	2.5	0.228	1.000	0.108	0.336	3.00	0.3	0.04	0.017
15	20	5	0.335	1.000	0.108	0.443	3.00	0.18	0.03	0.018
20	25	5	0.477	1.000	0.108	0.585	2.80	0.18	0.03	0.013
25	30	5	0.618	1.000	0.108	0.726	2.60	0.18	0.03	0.010
30	35	5	0.761	1.000	0.108	0.869	2.30	0.18	0.03	0.009
35	40	5	0.902	1.000	0.108	1.010	2.20	0.18	0.03	0.007
40	45	5	1.045	1.000	0.108	1.153	2.10	0.18	0.03	0.006
45	50	5	1.187	1.000	0.108	1.295	1.90	0.18	0.03	0.006
						Total Primary Consolidation Settlement (in)				4.14

Secondary Compression (Creep; in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**5.1**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.071	0.092	0.40	0.3	0.04	0.128
5	7.5	2.5	0.046	0.700	0.055	0.101	0.20	0.3	0.04	0.034
7.5	10	2.5	0.118	0.650	0.051	0.169	0.20	0.3	0.04	-0.013
10	12.5	2.5	0.157	0.590	0.047	0.204	0.30	0.3	0.04	0.011
12.5	15	2.5	0.228	0.550	0.043	0.271	0.30	0.3	0.04	-0.021
15	20	5	0.335	0.500	0.040	0.375	3.00	0.18	0.03	0.007
20	25	5	0.477	0.400	0.032	0.509	2.80	0.18	0.03	0.004
25	30	5	0.618	0.350	0.028	0.646	2.60	0.18	0.03	0.003
30	35	5	0.761	0.295	0.023	0.784	2.30	0.18	0.03	0.002
35	40	5	0.902	0.260	0.021	0.923	2.20	0.18	0.03	0.001
40	45	5	1.045	0.240	0.019	1.064	2.10	0.18	0.03	0.001
45	50	5	1.187	0.210	0.017	1.204	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				1.93

Total Marsh Creep (in)	2.5
Terrace Berm Fill Shrinkage (in)	1.9
Total Terrace Berm Creep (in)	1.0

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**7.3**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.088	0.109	0.40	0.3	0.04	0.143
5	7.5	2.5	0.046	0.700	0.069	0.115	0.20	0.3	0.04	0.040
7.5	10	2.5	0.118	0.650	0.064	0.182	0.20	0.3	0.04	-0.003
10	12.5	2.5	0.157	0.590	0.058	0.215	0.30	0.3	0.04	0.014
12.5	15	2.5	0.228	0.550	0.054	0.282	0.30	0.3	0.04	-0.008
15	20	5	0.335	0.500	0.049	0.384	3.00	0.18	0.03	0.009
20	25	5	0.477	0.400	0.039	0.516	2.80	0.18	0.03	0.005
25	30	5	0.618	0.350	0.034	0.652	2.60	0.18	0.03	0.004
30	35	5	0.761	0.295	0.029	0.790	2.30	0.18	0.03	0.002
35	40	5	0.902	0.260	0.025	0.927	2.20	0.18	0.03	0.002
40	45	5	1.045	0.240	0.024	1.069	2.10	0.18	0.03	0.001
45	50	5	1.187	0.210	0.021	1.208	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				2.51

Total Marsh Creep (in)	2.5
Terrace Berm Fill Shrinkage (in)	2.4
Total Terrace Berm Creep (in)	1.1

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**8.5**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.900	0.113	0.134	0.40	0.3	0.04	0.161
5	7.5	2.5	0.046	0.700	0.088	0.134	0.20	0.3	0.04	0.046
7.5	10	2.5	0.118	0.650	0.081	0.199	0.20	0.3	0.04	0.009
10	12.5	2.5	0.157	0.590	0.074	0.231	0.30	0.3	0.04	0.017
12.5	15	2.5	0.228	0.550	0.069	0.297	0.30	0.3	0.04	0.008
15	20	5	0.335	0.500	0.063	0.398	3.00	0.18	0.03	0.011
20	25	5	0.477	0.400	0.050	0.527	2.80	0.18	0.03	0.006
25	30	5	0.618	0.350	0.044	0.662	2.60	0.18	0.03	0.004
30	35	5	0.761	0.295	0.037	0.798	2.30	0.18	0.03	0.003
35	40	5	0.902	0.260	0.033	0.935	2.20	0.18	0.03	0.002
40	45	5	1.045	0.240	0.030	1.075	2.10	0.18	0.03	0.002
45	50	5	1.187	0.210	0.026	1.213	1.90	0.18	0.03	0.001
						Total Primary Consolidation Settlement (in)				3.26

Total Marsh Creep (in)	2.5
Terrace Berm Fill Shrinkage (in)	2.9
Total Terrace Berm Creep (in)	1.2

Total 20-year Settlement (in) (Hypothetical if no Dredge)	**9.9**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.074	0.095	0.40	0.3	0.04	0.131
5	7.5	2.5	0.046	0.870	0.069	0.115	0.20	0.3	0.04	0.040
7.5	10	2.5	0.118	0.850	0.067	0.185	0.20	0.3	0.04	0.008
10	12.5	2.5	0.157	0.810	0.064	0.221	0.30	0.3	0.04	0.015
12.5	15	2.5	0.228	0.770	0.061	0.289	0.30	0.3	0.04	0.010
15	20	5	0.335	0.750	0.059	0.394	3.00	0.18	0.03	0.011
20	25	5	0.477	0.710	0.056	0.533	2.80	0.18	0.03	0.007
25	30	5	0.618	0.680	0.054	0.672	2.60	0.18	0.03	0.005
30	35	5	0.761	0.635	0.050	0.811	2.30	0.18	0.03	0.004
35	40	5	0.902	0.620	0.049	0.951	2.20	0.18	0.03	0.003
40	45	5	1.045	0.600	0.047	1.092	2.10	0.18	0.03	0.003
45	50	5	1.187	0.560	0.044	1.231	1.90	0.18	0.03	0.002
						Total Primary Consolidation Settlement (in)				2.88

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	1.9
Total Terrace Berm Creep (in)	1.0

Total 20-year Dike Settlement (in)	**6.8**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.092	0.113	0.40	0.3	0.04	0.146
5	7.5	2.5	0.046	0.870	0.085	0.131	0.20	0.3	0.04	0.046
7.5	10	2.5	0.118	0.850	0.083	0.201	0.20	0.3	0.04	0.009
10	12.5	2.5	0.157	0.810	0.079	0.236	0.30	0.3	0.04	0.018
12.5	15	2.5	0.228	0.770	0.075	0.303	0.30	0.3	0.04	0.012
15	20	5	0.335	0.750	0.074	0.409	3.00	0.18	0.03	0.013
20	25	5	0.477	0.710	0.070	0.547	2.80	0.18	0.03	0.009
25	30	5	0.618	0.680	0.067	0.685	2.60	0.18	0.03	0.007
30	35	5	0.761	0.635	0.062	0.823	2.30	0.18	0.03	0.005
35	40	5	0.902	0.620	0.061	0.963	2.20	0.18	0.03	0.004
40	45	5	1.045	0.600	0.059	1.104	2.10	0.18	0.03	0.004
45	50	5	1.187	0.560	0.055	1.242	1.90	0.18	0.03	0.003
						Total Primary Consolidation Settlement (in)				3.31

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	2.4
Total Terrace Berm Creep (in)	1.1

Total 20-year Dike Settlement (in)	**7.8**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	0.940	0.113	0.134	0.40	0.3	0.04	0.161
5	7.5	2.5	0.046	0.870	0.104	0.150	0.20	0.3	0.04	0.051
7.5	10	2.5	0.118	0.850	0.102	0.220	0.20	0.3	0.04	0.022
10	12.5	2.5	0.157	0.810	0.097	0.254	0.30	0.3	0.04	0.021
12.5	15	2.5	0.228	0.770	0.092	0.320	0.30	0.3	0.04	0.033
15	20	5	0.335	0.750	0.090	0.425	3.00	0.18	0.03	0.016
20	25	5	0.477	0.710	0.085	0.562	2.80	0.18	0.03	0.011
25	30	5	0.618	0.680	0.082	0.700	2.60	0.18	0.03	0.008
30	35	5	0.761	0.635	0.076	0.837	2.30	0.18	0.03	0.006
35	40	5	0.902	0.620	0.074	0.976	2.20	0.18	0.03	0.005
40	45	5	1.045	0.600	0.072	1.117	2.10	0.18	0.03	0.004
45	50	5	1.187	0.560	0.067	1.254	1.90	0.18	0.03	0.004
						Total Primary Consolidation Settlement (in)				4.10

Total Marsh Creep (in)	1.0
Terrace Berm Fill Shrinkage (in)	2.9
Terrace Berm Fill Creep (in)	1.2

Total 20-year Dike Settlement (in)	**9.2**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.079	0.100	0.40	0.3	0.04	0.136
5	7.5	2.5	0.046	1.000	0.079	0.125	0.20	0.3	0.04	0.043
7.5	10	2.5	0.118	1.000	0.079	0.197	0.20	0.3	0.04	0.009
10	12.5	2.5	0.157	1.000	0.079	0.236	0.30	0.3	0.04	0.018
12.5	15	2.5	0.228	1.000	0.079	0.307	0.30	0.3	0.04	0.019
15	20	5	0.335	1.000	0.079	0.414	3.00	0.18	0.03	0.014
20	25	5	0.477	1.000	0.079	0.556	2.80	0.18	0.03	0.010
25	30	5	0.618	1.000	0.079	0.697	2.60	0.18	0.03	0.008
30	35	5	0.761	1.000	0.079	0.840	2.30	0.18	0.03	0.006
35	40	5	0.902	1.000	0.079	0.981	2.20	0.18	0.03	0.005
40	45	5	1.045	1.000	0.079	1.124	2.10	0.18	0.03	0.005
45	50	5	1.187	1.000	0.079	1.266	1.90	0.18	0.03	0.004
						Total Primary Consolidation Settlement (in)				3.33

Total Marsh Creep (in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**4.3**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.098	0.119	0.40	0.3	0.04	0.151
5	7.5	2.5	0.046	1.000	0.098	0.144	0.20	0.3	0.04	0.050
7.5	10	2.5	0.118	1.000	0.098	0.216	0.20	0.3	0.04	0.019
10	12.5	2.5	0.157	1.000	0.098	0.255	0.30	0.3	0.04	0.021
12.5	15	2.5	0.228	1.000	0.098	0.326	0.30	0.3	0.04	0.039
15	20	5	0.335	1.000	0.098	0.433	3.00	0.18	0.03	0.017
20	25	5	0.477	1.000	0.098	0.575	2.80	0.18	0.03	0.012
25	30	5	0.618	1.000	0.098	0.716	2.60	0.18	0.03	0.010
30	35	5	0.761	1.000	0.098	0.859	2.30	0.18	0.03	0.008
35	40	5	0.902	1.000	0.098	1.000	2.20	0.18	0.03	0.007
40	45	5	1.045	1.000	0.098	1.143	2.10	0.18	0.03	0.006
45	50	5	1.187	1.000	0.098	1.285	1.90	0.18	0.03	0.005
						Total Primary Consolidation Settlement (in)				4.12

Secondary Compression (Creep; in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**5.1**

Depth (feet)		Sub-Layer Thickness (feet)	Vertical Effective Stress (initial), tsf σ'_{v0}	Stress Factor	Vertical Effective Stress Increment $\Delta\ \sigma'_{v0}$	Vertical Effective Stress (final), tsf σ'_{vf}	Preconsolidation Pressure (tsf) σ'_{p}	CR Compression Ratio	RR Recompression Ratio	Consolidation Settlement (in) = [RR * log(σ'p/σ'v0) + CR * log(σ'vf/σ'p)] * Thickness
0	5	5	0.021	1.000	0.120	0.141	0.40	0.3	0.04	0.165
5	7.5	2.5	0.046	1.000	0.120	0.166	0.20	0.3	0.04	0.056
7.5	10	2.5	0.118	1.000	0.120	0.238	0.20	0.3	0.04	0.032
10	12.5	2.5	0.157	1.000	0.120	0.277	0.30	0.3	0.04	0.025
12.5	15	2.5	0.228	1.000	0.120	0.348	0.30	0.3	0.04	0.060
15	20	5	0.335	1.000	0.120	0.455	3.00	0.18	0.03	0.020
20	25	5	0.477	1.000	0.120	0.597	2.80	0.18	0.03	0.015
25	30	5	0.618	1.000	0.120	0.738	2.60	0.18	0.03	0.012
30	35	5	0.761	1.000	0.120	0.881	2.30	0.18	0.03	0.010
35	40	5	0.902	1.000	0.120	1.022	2.20	0.18	0.03	0.008
40	45	5	1.045	1.000	0.120	1.165	2.10	0.18	0.03	0.007
45	50	5	1.187	1.000	0.120	1.307	1.90	0.18	0.03	0.006
						Total Primary Consolidation Settlement (in)				4.98

Secondary Compression (Creep; in) 1.0

Total 20-year Settlement (in) (Consolidation of Natural Ground)	**6.0**

INITIAL UNDRAINED SETTLEMENT CALCULATIONS

ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705

PROJECT COLE'S BAYOU MARSH RESTORATION

SHEET NO OF

CALCULATED BY N DATE 3/10/14

CHECKED BY DATE

SCALE

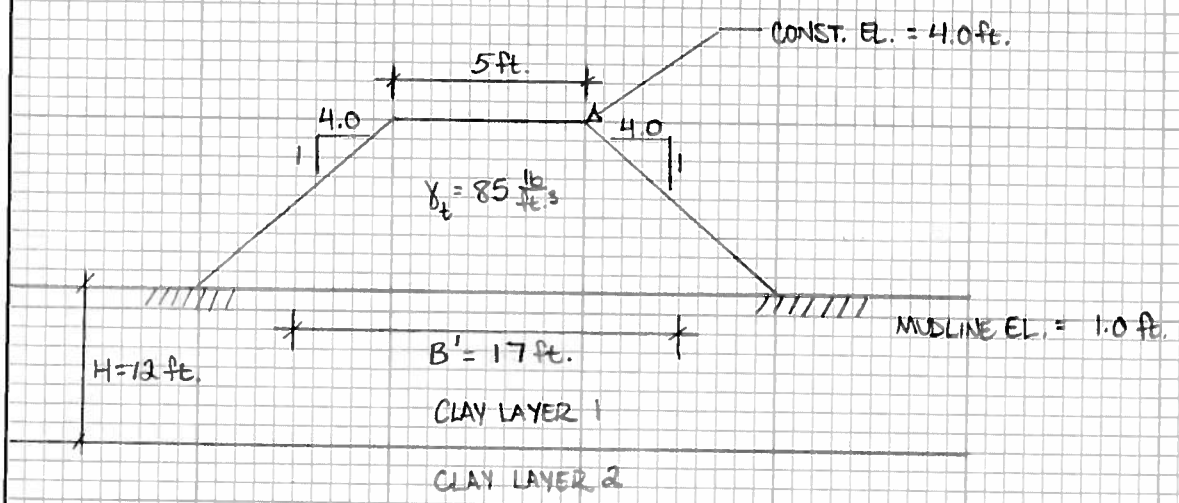
Confidential Information: Privileged & Confidential Work Product

INITIAL UNDRAINED SETTLEMENT

ESTIMATE UNDRAINED SETTLEMENT OF BERM FILL DURING CONSTRUCTION (VOLUME "LOSS" TO SUBSIDENCE BELOW EXISTING MUDLINE)

REF: "UNDRAINED SETTLEMENT OF PLASTIC AND ORGANIC CLAYS", FOOTT & LADD, ASCE JGED, VOL. 107, NO. GT8, AUGUST 1981.

SAMPLE CALCULATION



- BERM APPLIED STRESS: $q = (3.0 ft.)(85 \frac{lb}{ft^3}) = 255 \frac{lb}{ft^2}$

$B' = 2\left[\frac{1}{2}\left[4.0(3.0 ft)\right]\right] + 5 ft.$

$= 12 ft. + 5 ft.$

$B' = 17 ft.$

- FROM STABILITY ANALYSES, THE END OF CONST. FS = 1.3:

SHEAR STRESS RATIO, $\tau_n / C_u = (1/1.3) = 0.77$

- USING DIRECT SIMPLE SHEAR LAB DATA $\longrightarrow E_s/S_u = 23$

AVERAGE S_u @ MID-POINT $= 128.6 \frac{lb}{ft^2} \longrightarrow E_s = 23(128.6 \frac{lb}{ft^2})$ $2{,}957.8 \frac{lb}{ft^2}$

- DETERMINING ELASTIC SETTLEMENT: $\rho_e = \left(\frac{q B'}{E_u}\right) I_p$

FOR $\frac{H}{B'} = \frac{12 ft.}{17 ft.} = 0.71$ ft. $\longrightarrow$ FIG. 6 $\longrightarrow I_p = 0.24$

$$\rho_e = \left[\frac{(255 \frac{lb}{ft^2})(17 ft.)}{2{,}957.8 \frac{lb}{ft^2}}\right](0.24) = 0.352 ft. \approx \underline{0.35 ft.}$$

- INITIAL UNDRAINED SETTLEMENT: $\rho_i = \frac{\rho_e}{S_R}$

AVERAGE OCR 0 TO 12 ft. $\approx 2.0 \rightarrow$ FIG. 7 $\rightarrow f \approx 0.25$

FROM FIG. 8, FOR $H/B = 0.50$ & $q/q_{ULT} = 0.77$, $S_R = 0.80$

FOR $H/B' = 1.00$ & $q/q_{ULT} = 0.77$ $S_R = 0.77$

- FOR $\frac{H}{B'} = 0.71$:

$$\rho_i = \frac{0.35 ft.}{0.787}$$

$$\boxed{\rho_i = 0.45 ft \approx 5.5 in}$$

5126E (01/12)

Confidential Information:
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Work Product

- REPEATING THE CALCULATION WITH ANOTHER PRE-DREDGE ALTERNATIVE OF SIMILAR GEOMETRY:

- BERM APPLIED STRESS $q = 425 \frac{lb}{ft.^2}$
  $B' = 25$ ft.

- FROM STABILITY ANALYSES, THE END OF CONST. FS = 1.63:
  SHEAR STRESS RATIO, $\tau_n / C_u = (1/1.63) = 0.61$

- USING DIRECT SIMPLE SHEAR LAB DATA → $E_s/S_u = 25$
  AVERAGE S_u @ MID-POINT = $112.5 \frac{lb}{ft.^2}$ → $E_s = 25(112.5 \frac{lb}{ft.^2}) = 2{,}812.5 \frac{lb}{ft.^2}$

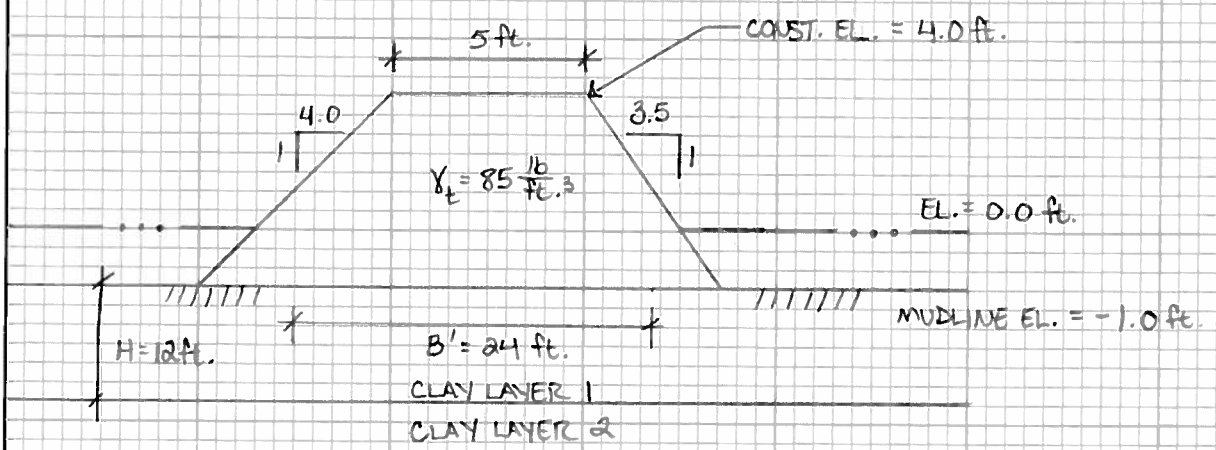
- DETERMINING ELASTIC SETTLEMENT: $\rho_e = \left(\frac{qB'}{E_u}\right) I_p$
  FOR $\frac{H}{B'} = \frac{12 ft.}{25 ft.} = 0.48$ ft. → FIG. 6 → $I_p = 0.14$

$$\rho_e = \left[\frac{(425 \frac{lb}{ft.^2})(25 ft)}{2{,}812.5 \frac{lb}{ft.^2}}\right](0.14) = 0.529 \text{ ft.} \approx \underline{0.53 \text{ ft.}}$$

- INITIAL UNDRAINED SETTLEMENT: $\rho_i = \frac{\rho_e}{S_R}$
  FROM FIG. 8, FOR $H/B' = 0.5$, $q/q_{ULT} = 0$, & $f = 0.25$ $S_R = 0.95$
  USE $S_R \approx 0.96$ FOR $H/B' = 0.48$

$$\rho_i = \frac{0.53 ft.}{0.96} \quad \boxed{\rho_i = 0.55 \text{ ft.} \approx 6.63 \text{ in}}$$

- REPEATING THE CALCULATION ONE LAST TIME FOR A NON-SYMMETRICAL ALTERNATIVE:



- BERM APPLIED STRESS: $q = 425 \frac{lb}{ft.^2}$

$$B' = \frac{1}{2}\left[(4.0)(5 ft.) + (3.5)(5 ft.)\right] + 5 ft.$$
$$= \frac{1}{2}\left[20 ft. + 17.5 ft.\right] + 5 ft.$$
$$= 18.75 ft. + 5 ft.$$
$$\underline{B' = 23.75 ft. \approx 24 ft.}$$

- ELASTIC SETTLEMENT: $\rho_e = \left(\frac{qB'}{E_u}\right) I_p$
  FOR $\frac{H}{B'} = \frac{12 ft.}{24 ft.} = 0.50$ ft. → FIG. 6 → $I_p = 0.15$

$$\rho_e = \left[\frac{(425 \frac{lb}{ft.^2})(24 ft.)}{2{,}812.5 \frac{lb}{ft.^2}}\right](0.15) = 0.54 \text{ ft.} \approx \underline{0.54 \text{ ft.}}$$

- FROM STABILITY ANALYSIS, END OF CONST. F.S. = 1.50:
  SHEAR STRESS RATIO, $\tau_n / C_u = (1/1.50) = 0.67$

- USING DIRECT SIMPLE SHEAR LAB DATA → $E_s/S_u = 25$
  AVERAGE S_u @ MID-POINT = $112.5 \frac{lb}{ft.^2}$ →
  $E_s = 25(112.5 \frac{lb}{ft.^2}) = 2{,}812.5 \frac{lb}{ft.^2}$

INITIAL UNDRAINED SETTLEMENT: $\rho_i = \frac{\rho_e}{S_R}$
$H/B' = 0.50$, $q/q_{ULT} = 0.67$, $f = 0.25$
$S_R = 0.90$

$$\rho_i = \frac{0.54 ft.}{0} \quad \boxed{\rho_i = \quad ft. \approx 7.2 \text{ in}}$$

Undrained Settlement of Plastic and Organic Clays

By Roger Foott,[1] M. ASCE, and Charles C. Ladd,[2] F. ASCE

Introduction

When a load is rapidly applied over a limited area above a clay layer, undrained shear deformations occur, giving rise to *initial* undrained settlements. Techniques for predicting initial settlements based on elastic theory are available. However, these settlements are frequently small compared to consolidation movements, so explicit prediction of initial settlement is often not performed during design. Subsequent undrained creep movements under constant load can cause additional undrained settlement; but routine predictive techniques for these movements are not readily available, and creep settlements are only rarely considered in design practice.

This paper presents performance data on three plastic, organic clays where initial settlements or continuing undrained creep movements, or both, resulted in substantial settlement in addition to that due to consolidation. With these types of soil, such settlements can represent a major design consideration. The paper recommends on approximate method for estimating initial settlement and discusses ways in which the potential for large, undrained initial and creep settlements can be recognized and accounted for in design practice.

Undrained Settlements

When a load is rapidly applied over a limited area above a clay soil deposit, the shear stresses induced in the clay cause lateral deformation of the soil resulting in settlement. This settlement is commonly considered an instantaneous response to the applied loading, therefore, occurring under undrained conditions and known as initial settlement, ρ_i.

Prediction of initial settlement is generally performed using a model derived from elastic theory and having the form

[1]Program Mgr., Soils and Foundation Engrg., Dames and Moore, Burlington, Mass.
[2]Prof. of Civ. Engrg., Massachusetts Inst. of Tech., Cambridge, Mass.

Note.—Discussion open until January 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 4, 1980. This paper is part of the Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. GT8, August, 1981. ISSN 0093-6405/81/0008-1079/$01.00.

$$\rho_i = \frac{q B I_\rho}{E_u} \quad \text{(1)}$$

in which q = the stress applied to the foundation soil; B = the width of the loaded area; I_ρ is an influence factor which depends on the geometry of the problem; and E_u = the undrained Young's modulus for the soil. This relationship assumes that the soil is linearly elastic and of infinite strength. When the clay is saturated, Poisson's ratio for undrained loading is 0.5, and this value is often included in the value of I_ρ.

Application of Eq. 1 requires an evaluation of E_u for the clay. Direct measurement of E_u via laboratory tests is extremely difficult since this parameter is very sensitive to such factors as: the shear stress level and the strain rate; the degree of sample disturbance, especially with unconsolidated-undrained tests; and the consolidation stress and the effects of aging when consolidated-undrained tests are used [e.g., Ladd (11)]. The alternative procedure of selecting values based on empirical correlations is, therefore, widely used, generally in the form of the normalized parameter E_u/c_u, in which c_u = the undrained shear strength of the clay. Bjerrum (1), for example, suggested that E_u/c_u values in the range of 500–1,500 were appropriate, with the lower value applying to very plastic clays subjected to large load increments and the higher limit applying to lean clays with small load increments.

D'Appolonia, et al. (4) presented an improvement in the estimation of ρ_i by including an explicit correction for the effects of local overstressing within the clay deposit. In essence, the *elastic* component computed via Eq. 1 is increased to account for the additional settlement caused by contained plastic flow, wherein a portion of the foundation soil becomes overstressed, resulting in increased lateral stress increments adjacent to the overstressed zone. D'Appolonia, et al. also evaluated field values of E_u/c_u based on 10 case studies: seven yielded E_u/c_u of 1,000–2,500 and the remaining three gave values of 800, 400, and 80–160. Based on these data they concluded that E_u/c_u of 1,000–1,500 was appropriate for "lean inorganic clays of moderate to high sensitivity," but that "considerably lower" values apply to "highly plastic clays and for organic clays." This latter comment is one of the earliest indications that ρ_i may be much more significant with plastic and organic clays than with other cohesive deposits, which have much higher values of E_u/c_u.

The values of ρ_i obtained using the above prediction method tend to be small in comparison to the consolidation settlement ρ_c, at least for *soft ground* conditions wherein the clay is loaded into the virgin compression range. This is particularly so when the loaded width, B, is several times larger than the clay layer thickness, H, for which the values of I_ρ become very small (Fig. 6).

Also, when a ρ_i estimate is made, the conventional one-dimensional ρ_c computation should logically be modified to incorporate a correction as described by Skempton and Bjerrum (19). This correction accounts for the fact that, because of the shear deformations which cause ρ_i, the pore pressure increment immediately after loading, Δu_i, is not equal to the vertical stress increment, $\Delta\sigma_v$, as assumed in the conventional one-dimensional ρ_c computation. In all cases, except perhaps for a very sensitive soft clay loaded to low factors of safety, this correction will result in values of Δu_i that are less than $\Delta\sigma_v$, with a corresponding reduction in ρ_c (1). This reduction in ρ_c can compensate for

most, if not all, of the estimated ρ_i, especially for cases involving highly overconsolidated clay foundations. Indeed, Seed (17) reported that the estimation of ρ_c using the conventional one-dimensional procedure "appeared to give a reasonably correct assessment of the ultimate settlement."

Thus, in many practical situations, there appears to be little merit in extending a settlement prediction beyond evaluation of ρ_c using the conventional one-dimensional model. This conclusion is reinforced by the frequent uncertainties in the computation of ρ_c. These include the distribution of stress increment with depth, idealization of the actual soil profile, evaluation of the amount of prestress (overconsolidation) in the clay, the selection of compressibility parameters, and the applicability of the assumed one-diemsnional consolidation strains under two- or three-dimensional loading conditions. In most cases the range of the ρ_c estimate does not justify modifying the settlement prediction to include the small values of ρ_i typically predicted.

Moreover, Tavenas and Leroueil (21) conclude that the response of most clay foundations is not truly undrained during typical field loading situations, as implied above. They suggest that significant pore pressure dissipation usually occurs during initial loading with all overconsolidated foundation clays, which then rapidly reach a normally consolidated state during continued subsequent loading. In the normally consolidated state, pore pressure dissipation is much slower. Thus, according to these authors, a truly undrained response develops only in those portions of the foundation clay which have become normally consolidated during the latter stages of construction.

In addition to the *instantaneous* undrained settlements, ρ_i, which occur essentially during loading (and perhaps accompanied by some consolidation settlement as per above), settlement can also occur due to undrained creep of the clay, ρ_{cr}. As with ρ_i, ρ_{cr} is due to lateral deformation of the clay resulting from the applied shear stresses in the early stages of consolidation, but it can continue long after the load is applied.

Time-dependent undrained shear deformations have been the subject of considerable research [e.g., Singh and Mitchell (18) and Mitchell (15)], and computer models have been developed [e.g., Edgers et al. (5)] to predict the creep behavior of foundations. Determination of appropriate parameter values for use in these models is extremely difficult, however, and they remain primarily a research tool. As a practical matter, settlement due to undrained creep is rarely evaluated, is probably usually small, and might typically be interpreted as being part of the consolidation settlement. In any case, prediction of settlement due to undrained creep is not part of conventional geotechnical design practice.

Three Field Studies

Having reviewed the general approach used in design regarding undrained settlement, pertinent aspects of the observed field behavior at three sites will not be presented. All three field studies involved loading organic or highly plastic clays, or both, into the virgin compression range.

Atchafalaya Levees.—In 1965 the U.S. Army Corps of Engineers (New Orleans District) constructed three levee test sections in the Atchafalaya Basin to evaluate alternative levee designs. The test sections were built over existing levees on approximately 120 ft (37 m) of soft to medium highly plastic CH clay (plasticity

index, I_p, typically 50%–80%). The undrained shear strength, c_u, profile through the clay foundation soils evaluated using the SHANSEP procedure [Ladd and Foott (13)] is shown in Fig. 1, discontinuities in the strength contours at El.

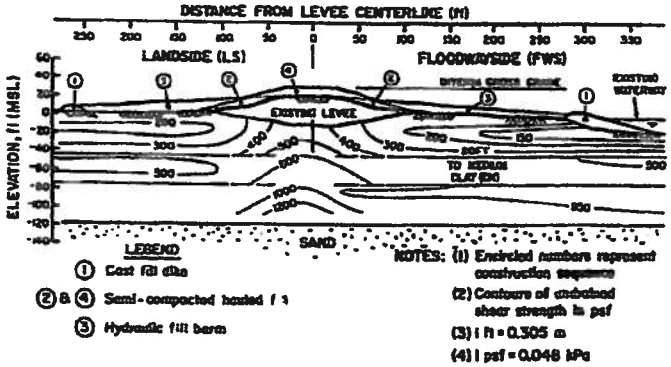


FIG. 1.—Atchafalaya Test Section III: Plan for Raising Levee Grade [after (8,10)]

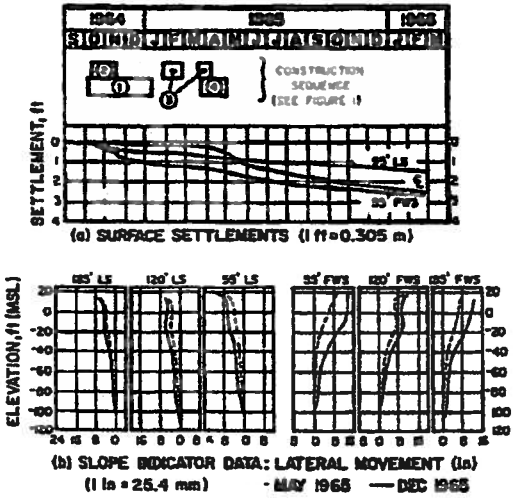


FIG. 2.—Atchafalaya Test Section III: Typical Settlement and Lateral Movement Data [after (10)]

−45 and −75 representing the top of overconsolidated clay layers [Foott and Ladd (7)]. The test section performance has been described and investigated

by Kaufman and Weaver (10), Edgers et al. (5), Fuleihan and Ladd (9), and Foott and Ladd (7,8). Of present interest is the performance of Test Sections II and III, which experienced large initial settlements and ongoing lateral creep movements associated with relatively little dissipation of excess pore pressure.

Fig. 1 shows Test Section III and the sequence used to construct it to an interim gross grade of El. 27. The levee had a nominal design factor of safety of 1.3 and an actual value probably closer to 1.15 [Foott and Ladd. (7)]. Test Section III was extensively instrumented and selected surface settlement and lateral movement data obtained during and for one year after construction are presented in Fig. 2.

The surface settlement data in Fig. 2(*a*) show that substantial settlements occurred at all locations during the construction period and the subsequent year of observation, these settlements ranging up to almost 3 ft (0.9 m) beneath the approximately 10 ft (3 m) of fill that was placed. Of particular interest is the rapid settlement of about 1 ft (0.3 m) which occurred at the center line plate during April and May, 1965, corresponding to the placement of fill at the levee center line and the two or three weeks immediately thereafter. A similar sharp settlement at the 55 ft (17 m) floodwayside (FWS) offset is apparent in October and November, 1964, this corresponding to placement of fill above that settlement plate. Also of interest are the lateral deflections of up to 8 in. (200 mm) recorded by May, 1965, the end of construction, and the continuation of these lateral movements through the following months.

Review of the extensive piezometer data presented in Kaufman and Weaver (10) shows that the excess pore pressures generated by the loading experienced no dissipation immediately after construction. Indeed, in some cases, the pore pressures continued to rise slightly for several weeks after the end of construction, reaching their highest values at approximately the end of June, 1965. Some dissipation did occur subsequently, but more than 80% of the excess pore pressures initially generated remained one year after construction.

It thus appears that the embankment settlements recorded in Fig. 2 contain a large component due to initial settlement and subsequent, largely undrained, creep movements, and further that this component is a very significant design consideration for embankments on these deposits. The writers (8) present comparisons of predicted and measured lateral definitions.

Cross River Embankment.—Between 1975 and 1978 a highway embankment up to 4.7 m high was constructed across the flood plain of the Cross River in Nigeria. The embankment was founded on organic, plastic clay deposits (CH-OH, I_p = 25%–50%) up to 30 m in depth. The lower 10 m approximately of these deposits was medium to stiff and there was a desiccated surface crust of 2 m or more (Fig. 3). However, the intervening clay of up to 18 m thickness was generally soft, particularly for the few meters immediately below the surface crust where strengths of 200 psf (10 kPa) and less were measured in some of the unconfined compression tests. Field vane strengths were typically 275 psf (13 kPa) or higher for these softer deposits.

The design and construction of the Cross River embankment were reported by Foott et al. (6). Present comments are limited to undrained deformation behavior inferred from observed settlement data for the section of embankment built over a buried loose sand lense, which appears to have been the natural levee of a previous river course, as shown in Fig. 3.

The embankment design for this section is illustrated in Fig. 3. It involved an initial construction phase of hydraulically placed sand with three sand drains installed every 10 m along the embankment to ensure that the buried sand lense was free-draining. This first construction phase, considered to have an initial factor of safety in excess of 1.3, was left in place for approximately 1 yr to allow the upper portion of the clay to consolidate, forming a stronger

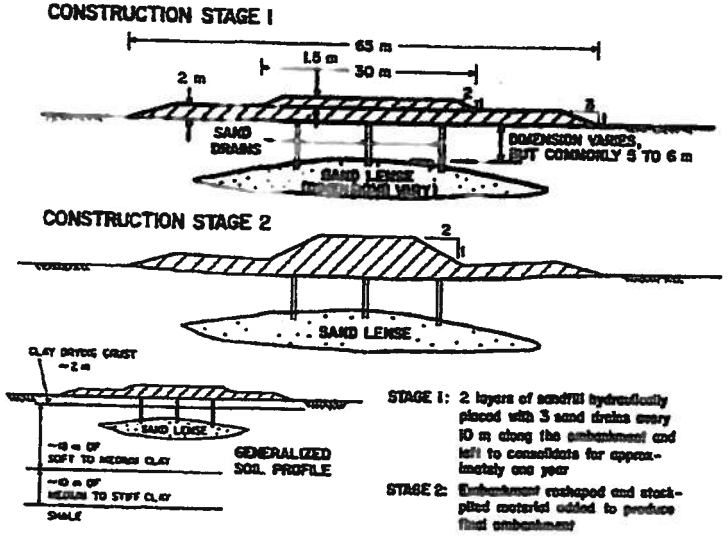


FIG. 3.—Cross River Embankment Design over Buried Sand Lense

layer of clay capable of supporting the final embankment section. The final section was then constructed by reshaping the first embankment stage and adding stockpiled material (construction stage 2 in Fig. 3).

An evaluation of the performance of the first construction phase, undertaken after 1 yr of consolidation, revealed that:

1. The upper levels of the clay, in which consolidation was aided by the buried sand lense and the sand drains, was substantially fully consolidated.
2. Large excess pore pressures still remained in the underlying clay deposits where consolidation was much slower.
3. The total center line settlements measured during the first construction phase were generally in the range 0.75 m–1.25 m, of which 0.50 m–0.75 m typically occurred very quickly during and shortly after load application, with subsequent settlements being much slower and tending to reduce with time in classical consolidation fashion (Fig. 12 of Ref. 6).
4. Based on data obtained during additional field vane testing after the 1-yr consolidation period, it appeared that the thickness of clay overlying the buried sand lense had been reduced on the order of 1 m.

It thus appears that most of the observed settlement probably occurred in the upper clay layer above the sand lense. Since some 2 m of this material is a stiffer surface crust which should not compress significantly under the applied loading, it further appears that much of the observed settlement occurred in the remaining 3 m–4 m of soft clay above the sand lense.

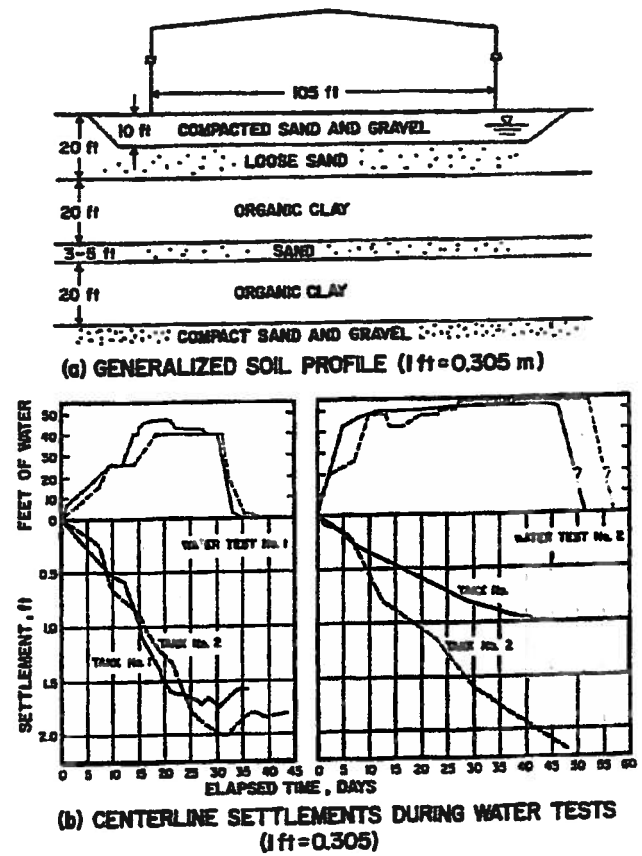


FIG. 4.—Storage Tank Soil Profile and Load-Settlement Data

Such movements are far too large to have resulted from consolidation of this thin layer of soft clay. It might thus be concluded that much of the observed settlement was actually an initial undrained settlement caused by lateral movement in the clay above the sand lense.

Storage Tanks on Organic Clay.—The third field study concerns the settlement of two 105 ft (32 m) diameter flexible bottom steel storage tanks constructed in 1970 on a foundation soil profile that included two layers of organic, plastic clay, CH-OH, $I_p = 45\%$, each about 20 ft (6 m) thick, as shown in generalized fashion in Fig. 4(*a*). There were some differences in the soil profiles at each tank, the Tank No. 2 profile having a somewhat greater thickness of clay and including a thin peat layer under part of the tank. The average undrained strength of the upper clay layer shown in Fig. 4(*a*) was approximately 600 psf (30 kPa) and the strength of the lower layer was approximately 1,000 psf (50 kPa). The design load of the tank was 3,300 psf (158 kPa), corresponding to a water height of 53 ft (16.2 m), and the initial factor of safety of the tanks under full load was estimated at approximately 1.5. With an assumed prestress of 1,200 psf (57.5 kPa) throughout both clay layers, center line consolidation settlements of approximately 2 ft (0.6 m) are calculated for Tank No. 2 using the conventional one-dimensional procedure, while the excessively conservative assumption of zero prestress for the foundation clays increases this settlement to approximately 5 ft (1.5 m).

Fig. 5(*b*) plots the center line settlements recorded for these tanks during two water tests. The tank bottoms were mudjacked back to their initial shape after each water test, and the two tanks were then put into service. However, despite relatively low product loadings, the tanks continued to settle, Tank No. 2 experiencing a further approximately 0.75 ft (0.23 m) center line settlement over a 3-yr period. During this time the average load was about 1,200 psf (57.5 kPa) and peak loads were only rarely and briefly in excess of 1,800 psf (86.2 kPa), with a maximum of about 2,100 psf (100 kPa). When Tank No. 2 was subsequently loaded to about 2,400 psf (115 kPa) for two to three months, it underwent approximately 0.25 ft (0.08 m) additional center line settlement.

These in-service settlements are mainly attributed to consolidation. The fact that consolidation was still occurring under these relatively low loads, plus the large magnitude of the water test settlements, demonstrate clearly that a substantial portion of the observed water test settlements was due to undrained shear deformations, both initial and creep. The amount of undrained settlement cannot be accurately established, but it appears that well in excess of 2 ft (0.6 m) of center line settlement during the water testing of Tank No. 2 could be due to this cause.

Of particular interest are the high factors of safety at which these large initial and creep settlements occurred. With a factor of safety of 1.5 at a 3,300 psf (158 kPa) loading, the 40 ft (12.2 m) maximum water loading [2,500 psf (120 kPa)] during the first test of Tank No. 2 represents a factor of safety of 2.0, while the factor of safety for the intermediate loading of approximately 25 ft (7.6 m) of water [1,550 psf (75 kPa)] during this first test increases to slightly over 3.0. Nevertheless, large initial settlements probably occurred at these relatively low loadings, judging from the large settlements shown in Fig. 4(*b*).

Also worthy of note is the fact that the measured long-term settlement behavior of a nearby, heavily loaded tank demonstrated that when consolidation was finally completed, further in-service tank settlements essentially stopped. It, therefore, appears that large settlements due to undrained shear deformations, both initial and creep, would cease being a problem once the foundation clays become fully consolidated, even with repeated loadings to design capacity.

Discussion of Field Studies

These field studies demonstrate the existence of undrained settlements much greater than would be expected based on past experience. Indeed, in the cases of the Atchafalaya Levee test sections and the storage tanks, initial settlements plus continuing settlement due to undrained lateral creep probably were of the same order of magnitude as the consolidation settlement which would occur under the applied loads. Continued shear deformation due to lateral creep is clearly evident for the Atchafalaya Levees from the inclinometer data and can be presumed, but not positively proven, for the storage tanks. It does not appear to have been a major factor with the Cross River embankment, probably because the upper layer of clay, in which the initial settlement movements mainly occurred, consolidated quite rapidly.

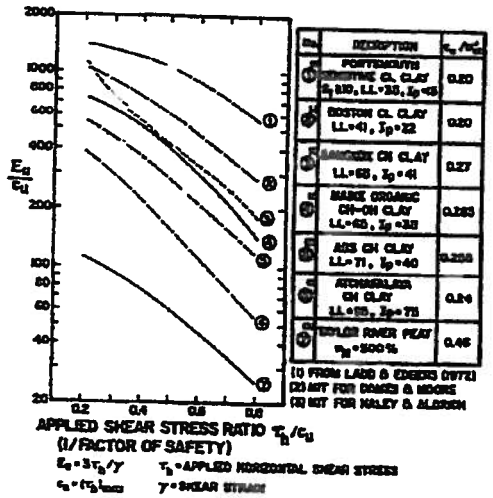
Thus, it is apparent that for the three field cases described, the normal situation where ρ_i is small compared to ρ_c does not apply. In all cases, prediction of ρ_c does not apply. In all cases, prediction of ρ_i using even the lower-bound value of $E_u/c_u = 500$, suggested by Bjerrum (1), would greatly underestimate the initial settlements which occurred. Furthermore, unless the soils consolidated fairly rapidly, it appears that continuing creep deformations can lead to substantial additional settlements.

These three soils therefore seem to present a special design problem which requires considerations beyond the conventional design practice discussed previously. Even allowing for the effects of local overstressing using the D'Appolonia et al. (4) approach, it is necessary for the soils to have extremely low values of E_u/c_u in order to explain the large initial settlements.

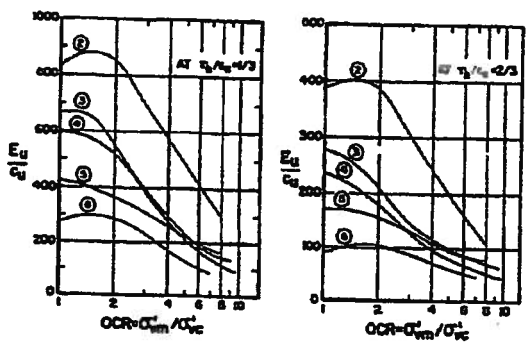
The foundation soils involved in these three cases exhibiting unusually large undrained shear deformations all had a high plasticity index and, to varying degrees, contained organic matter. A soil combining high plasticity and a large organic content might, therefore, be particularly susceptible to such movements. Accordingly, with designs involving loadings on plastic and organic soils, it is appropriate to carefully consider the possibility that large undrained shear deformations may occur in addition to the conventionally estimated consolidation settlement.

Approximate Prediction of Initial Settlement

The method of predicting initial settlements based on elastic theory presented by D'Appolonia et al. (4) is the most rational approach available, accounting explicitly for the effects of initial stress history and shear stress level in the foundation clay. As noted earlier, their 1971 paper suggested that the required value of E_u be selected based on empirical correlation with E_u/c_u. Since that time, further research at MIT [Ladd et al. (14)] has indicated that an approximate value of E_u can be obtained from K_o consolidated-undrained, CK_oU, direct simple shear (DSS) tests run on samples of the soil reconsolidated using the SHANSEP procedure [Ladd and Foott (13)] to reduce sample disturbance effects (excluding highly sensitive and naturally cemented soils that have very high E_u/c_u values and are not ammenable to the SHANSEP approach). The following design method for the approximate prediction of ρ_i applies these CK_oUDSS data to the D'Appolonia et al. (4) procedure for field situations involving loading



(a) NORMALIZED SECANT MODULUS VS. STRESS LEVEL FOR NORMALLY CONSOLIDATED SOILS



(b) NORMALIZED SECANT MODULUS VS. OVERCONSOLIDATION RATIO

FIG. 5.—Normalized Modulus Data from K_o-Consolidated Undrained Direct Simple Shear Tests [from (14)]

the foundation clay beyond its maximum past pressure.

The prediction involves four steps:

1. *Select a value for E_u*, ideally from DSS tests run using the SHANSEP reconsolidation procedure and selected using an applied shear stress ratio equal to the reciprocal of the stability factor of safety for the loading. Alternatively, Fig. 5 gives guidance for selecting a value based on available data for a wide range of clays. The data in Fig. 5(*a*) are for normally consolidated soils, and Fig. 5(*b*) shows the trends expected with increasing over-consolidation ratio, OCR.

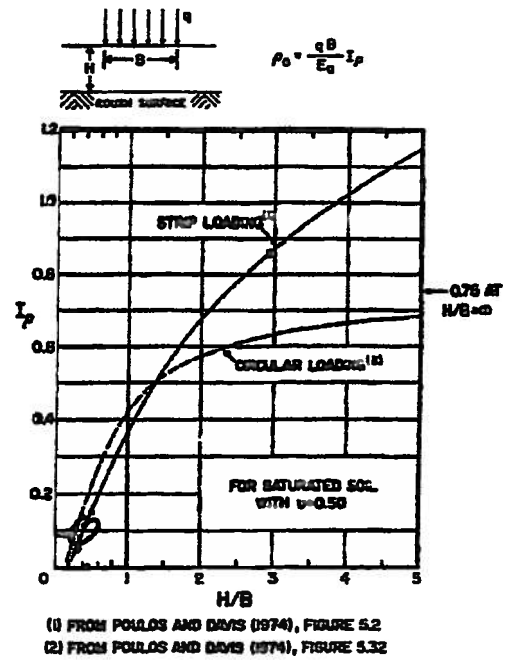


FIG. 6.—Undrained Elastic Settlement Computation for Uniform Loading on Elastic Layer

2. With the value of E_u obtained from Step 1, *compute the elastic settlement*, ρ_o, using the formula and influence factor chart shown in Fig. 6.

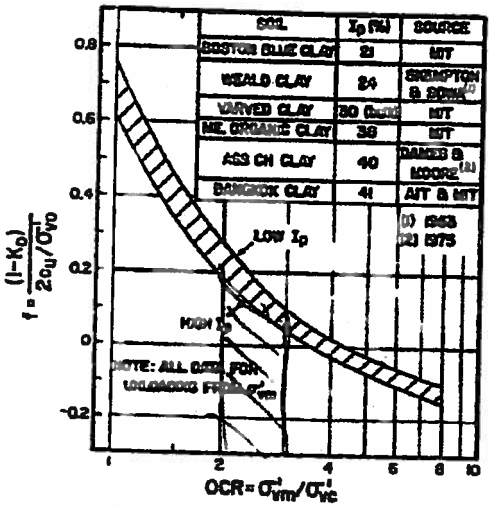
3. *Select a value of the initial shear stress ratio* (f) from Fig. 7 as a function of the OCR of the clay.

4. *Select a value of the settlement ratio*, S_R, from Fig. 8 as a function of the applied stress ratio, the geometry of the loading, H/B, and f. The initial settlement, ρ_i, can then be computed as

$$\rho_i = \frac{\rho_e}{S_R} \qquad (2)$$

Note that Fig. 8 applies to strip loadings, but D'Appolonia et al. (4) indicate that it is also reasonably applicable to circular loadings.

It must be emphasized that the above approach is very approximate, particularly when the value of E_u is selected empirically from Fig. 5 without any DSS tests being performed. The effects of partial consolidation that may occur within overconsolidated deposits during construction are also ignored. Nevertheless, the method should indicate the probable order of magnitude of ρ_i and can serve to give some warning of situations wherein ρ_i may be a significant design



NOTE: $c_u = 0.5(\sigma_1 - \sigma_3)_f$ FROM CK_0U TRIAXIAL OR PLANE STRAIN TESTS. K_0 FROM BROOKER AND IRELAND (1968) FOR ME. ORGANIC CLAY.

FIG. 7.—Initial Shear Stress Ratio Versus OCR [from (14)]

consideration. In the majority of cases, relatively high values of E_u/c_u should apply and ρ_i will be, at most, a few inches. Whenever low values of E_u/c_u are obtained from DSS tests, however, there is a clear warning that significant ρ_i settlements in relation to ρ_c may occur. This is particularly likely when highly plastic soils containing organic materials are encountered. It should be noted that an E_u/c_u value as low as 40 can be backfigured for the plastic organic clay at the storage tank site.

Accounting for Undrained Settlements in Design

Using the above approach, approximate predictions of initial settlement can

be obtained. Additional, creep-induced, undrained settlements cannot be reliably predicted, but a reasonable and probable general concept is: When initial settlements are small, undrained creep deformation will also be small: when initial settlements are significant but consolidation is rapid, creep settlements will tend to be small; and when initial settlements are significant and consolidation is slow, creep settlements will also tend to be significant. With this concept, the engineer can qualitatively consider the potential effect of these settlements on design.

It should again be recognized that, as discussed previously, most field evidence indicates that ρ_i is usually not an important design consideration and that conventional predictions of ρ_c using the one-dimensional model will generally

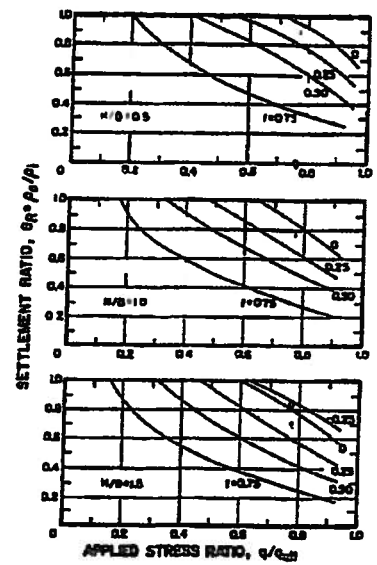


FIG. 8.—Settlement Ratio Versus Applied Stress Ratio for Strip Load on Isotropic Homogeneous Foundation [from (4)]

yield an adequate estimate of the total settlement that will occur. Therefore, design practice does not need to be changed with those situations. The objective is rather to try and detect special cases wherein undrained settlements might become troublesome and to make allowance for potential problems before they occur.

Thus in most cases values of E_u/c_u of 500 or more will apply and ρ_i will be small. Indeed reference to Fig. 5 indicates that even plastic clays nos. 3, 4, and 5 usually have values of E_u/c_u of 200 or higher, for which ρ_i may start to be significant but will rarely be a major design consideration. The low values for E_u/c_u obtained for soils nos. 6 and 7 and backfigured for the storage

tank site, however, are indicative of substantial potential problems and should be considered a distinct warning that large undrained movements may occur and result in major settlements in addition to ρ_c. This possibility should be carefully considered when the soil in question is highly plastic and when it has a significant organic content.

When potentially large ρ_i movements are indicated, the first question must be whether the additional settlement is an important design consideration. With an embankment, wherein additional fill can be added to compensate for ρ_i, it may not be (although this might not be true for continuing undrained creep settlements). With other structures the settlement may not be so easily compensated for, in which case consideration should be given to the use of precompression so that the initial settlement and some consolidation settlement will occur before the final structure is placed on the foundation.

Assuming that the ρ_i settlements can be accommodated in some way, the next step is to consider the possible consequences of undrained creep movements, ρ_{cr}. With large ρ_i, significant ρ_{cr} should be expected unless the foundation soils consolidate fairly rapidly. Creep susceptibility of the soil *might* be estimated to some extent by running laboratory tests and comparing the data obtained to published creep parameter values [e.g., Mitchell (15)] and associated observed soil behavior, although such an approach would still be in the realm of research [Edgers et al. (5)]. Also the behavior of prior structures on the same deposits may give excellent guidance. In general, however, it appears that very low modulus soils tend to be quite highly creep susceptible. The safest way of preventing substantial creep-induced settlements is to ensure that the clay consolidates quite rapidly, thus increasing its stiffness and decreasing the shear stress levels in the foundation clay. If necessary, the installation of vertical drains to accelerate consolidation in portions of the foundation soils, in order to limit lateral movements, should be considered.

Another possible approach is the use of incremental loading via stage construction, allowing stiffening of the clay soils through consolidation before adding the next load increment. This will reduce both the ρ_i and ρ_{cr} components of settlement below the values that would occur with immediate full loading. However, the soil's modulus may be so low that even these reduced undrained settlements are large enough to be a significant problem.

Summary and Conclusions

1. For *soft ground* loading conditions (i.e., clay foundations loaded into the virgin compression range), the conventional practice of computing consolidation settlements using the one-diemsnional model, without explicit computation of initial settlement, will generally yield total settlement estimates of sufficient accuracy for design purposes.

2. Initial settlements due to undrained shear deformations are usually small and for practical purposes can often be assumed to be included in the consolidation settlements computed above. However, the three field studies presented herein show that initial settlements may become very significant with highly plastic or organic foundation soils, or both, especially when loaded to low stability factors of safety, and may even become the predominant design consideration. The occurrence of large initial settlements may also be followed by excessive undrained creep movements if consolidation occurs very slowly.

3. Approximate predictions of initial settlements can be made using the method of D'Appolonia et al. (4), with modulus values selected from direct simple shear tests or estimated using Fig. 5. These predictions, plus consideration of soil type and consolidation time, should indicate cases where initial and creep-induced settlements could be a significant design problem.

4. In such cases, the effect of the initial settlement on the structure should be evaluated and the possible effects of creep-induced settlements considered.

5. The effect of initial settlement on the structure can be reduced by precompression or, to some extent, by a controlled loading program which allows consolidation to increase the soil stiffness and reduce the shear stress level in the foundation. The adverse effects of creep settlements also can be reduced by installing vertical drains to accelerate the rate of consolidation.

Appendix I.—References

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Appendix II.—Notation

The following symbols are used in this paper:

B = width of loaded area;
c_u = undrained shear strength;
CK_oU = K_o consolidated-undrained shear test;
DSS = direct simple shear;
E_u = undrained Young's modulus;
f = initial shear stress ratio;
H = layer thickness;
I_ρ = influence factor;
I_p = plasticity index;
K_o = coefficient of earth pressure at rest;
LL = liquid limit;
OCR = overconsolidation ratio;
q = applied foundation stress;
q_{ult} = ultimate bearing capacity;
S_R = settlement ratio;
S_t = sensitivity;
w_N = natural water content;
γ = shear strain;
Δu_i = pore pressure increment immediately after loading;
$\Delta\sigma_v$ = vertical stress increment;
ν = Poisson's ratio;
ρ_i = initial settlement (due to undrained shear deformation);
ρ_c = consolidated settlement;
ρ_{cr} = undrained creep settlement;
ρ_e = elastic settlement;
σ'_{vc} = vertical consolidation stress;
σ'_{vm} = maximum past pressure;
σ'_{vo} = in-situ vertical consolidation stress; and
τ_h = horizontal shear stress in DSS test.

PRIMARY CONSOLIDATION RATE CALCULATIONS

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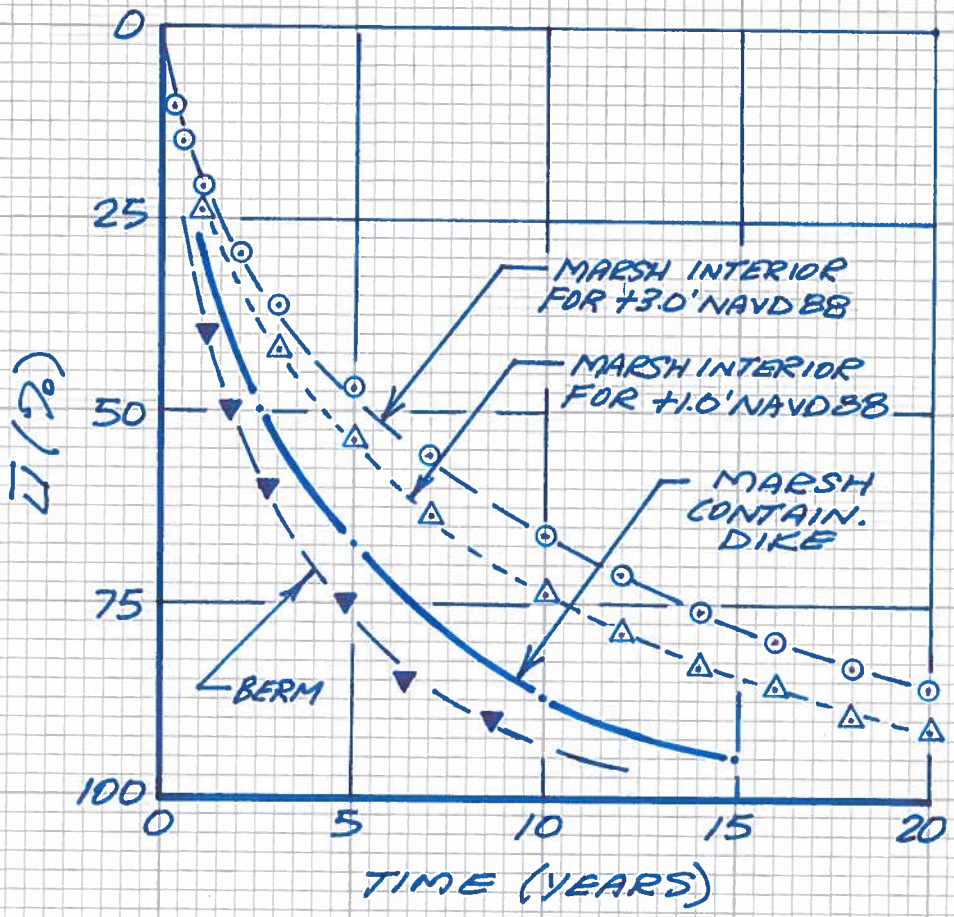
MARSH AREA SETTLEMENT RATES
(FOUNDATION CONSOLIDATION)



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SUMMARY
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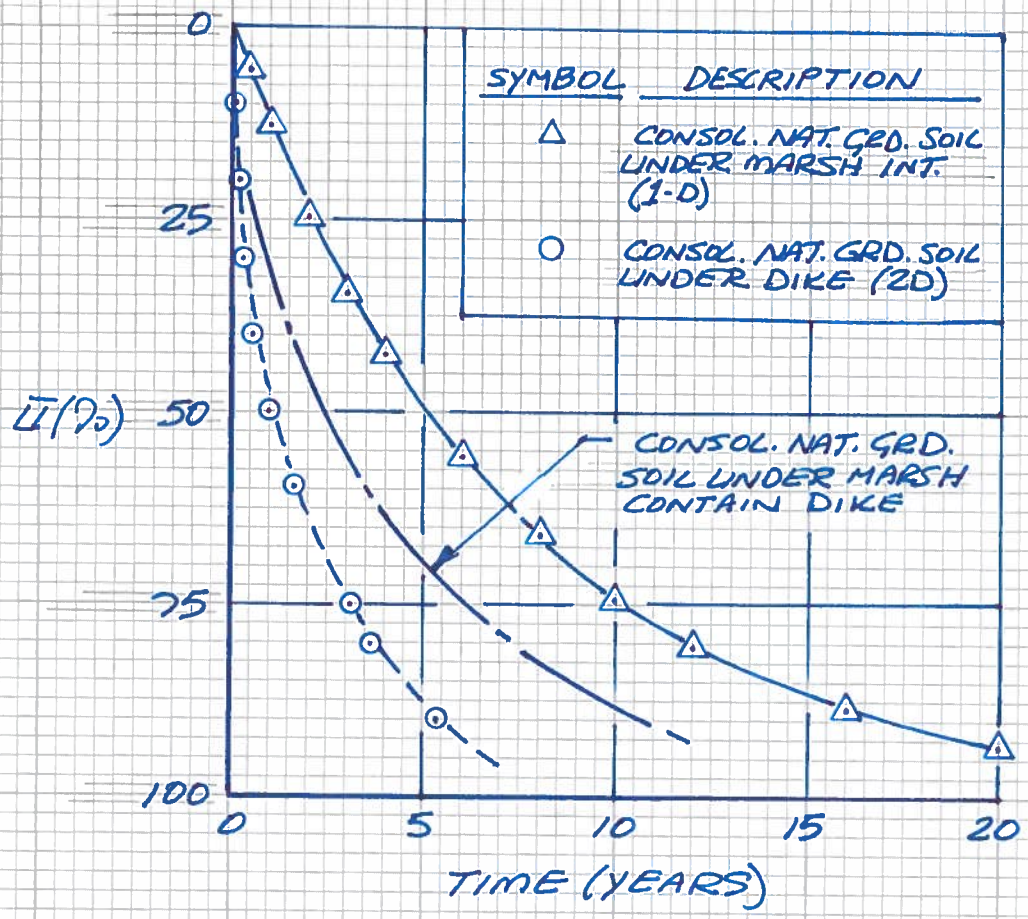

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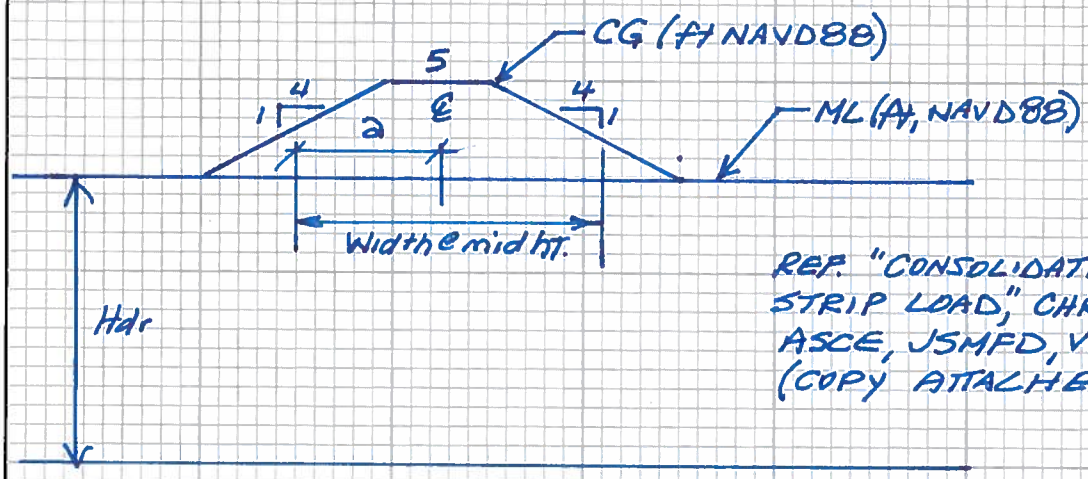
BERM CONSOL. RATE



REF. "CONSOLIDATION UNDER A STRIP LOAD," CHRISTIAN, J.T., ET. AL., ASCE, JSMFD, VOL. SM-7, JULY 1972 (COPY ATTACHED)

MARSH NO.	Hdr (ft)	TYP. ML El	DIKE CG El	a (ft)	a/H
2 (Cv=0.25)	33.5	1.0	3.0	6.5	0.19
			4.0	8.5	0.25
1 (Cv=0.25)	33.5	0.0	3.0	8.5	0.25
			4.0	10.5	0.31
3 (Cv=0.20)	34.8	0.0	3.0	8.5	0.24
			4.0	10.5	0.30

USE a/H=0.25

$\bar{U}$ (%)	TIME FACTOR Kh=Kv	TIME FACTOR Kh=3Kv	Time t, yrs (Kh=3Kv) MARSH 1+2		Time t, yrs (Kh=3Kv) MARSH 3
10	0.007	0.006	0.07	~~0.09~~	0.100
20	0.014	0.013	0.16	~~0.20~~	0.216
30	0.027	0.024	0.30	~~0.37~~	0.40
40	0.055	0.049	0.60	~~0.75~~	0.81
50	0.09	0.078	0.96	~~1.20~~	1.29
60	0.155	0.134	1.65	~~2.06~~	2.22
75	0.30	0.249	3.06	~~3.83~~	4.13
80	0.36	0.290	3.56	~~4.46~~	4.81
85	0.41	0.317	3.90	~~4.87~~	5.26
90	0.60	0.441	5.42	~~6.78~~	7.32

USED WRONG Cv

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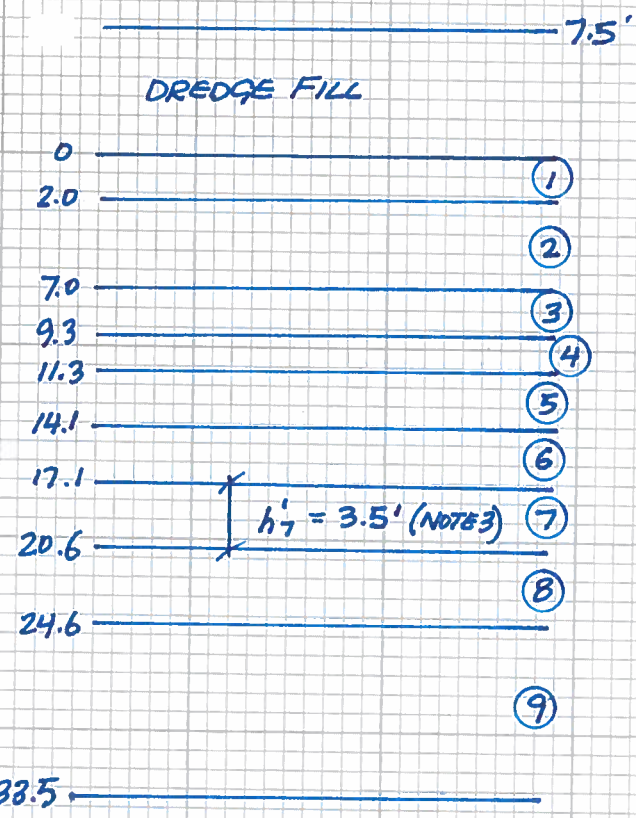
CONSOLIDATION RATE
MARSH #1 + #2
MARSH CONST. GRADE +3.0' NAVD88

TYPICAL PROFILE

		C_v(12b) ft²/d	
0			
	OCR 19 → 3.2 (NOTE 1)	0.10 (NOTE 2)	1
5			
	OCR 2.4 → 1.1	0.025	2
10			
	OCR 15.5 → 10.1	0.12	3
15			
	OCR 9.0 → 6.8	0.10	4
20			
	OCR 5.9 → 4.8	0.08	5
25			
	OCR 4.2 → 3.6	0.07	6
30			
	OCR 3.0 → 2.6	0.05	7
35			
	OCR 2.4 → 2.2	0.04	8
40			
	OCR 1.8 → 1.6	0.025	9
50			

DESIGNATE AS NO-FLOW BOUNDARY @ 50'



NOTE 3: SUBLAYER THICKNESS TRANSFORMED TO EQUIV. h' FOR UNIFORM $C_v = C_{v2} = 0.025$ ft²/d

$$h'_7 = h_7 \left(\sqrt{C_{v2}}/\sqrt{C_{v7}}\right)$$

DREDGE FILL $C_v = 0.004$ ft²/day FROM SLURRY CONSOL. TEST

$$h' = 3.0\left(\sqrt{0.025}/\sqrt{0.004}\right) = 7.5'$$

H_{dr} FOR $C_v = 0.025 \rightarrow 41.0'$

NOTE 1: OCR = OVERCONSOLIDATION RATIO @ $\bar{U} = 0\% \rightarrow \bar{U} = 100\%$ FOR $\Delta\bar{\sigma}_v$ CORRESP. TO 30' DREDGED FILL (MARSH CONST. GRADE +3.0' NAVD88) FIGURE + TABLE

NOTE 2: FROM C_v VS. STRESS RATIO (1/OCR) FIGURE

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0.183

0.0054

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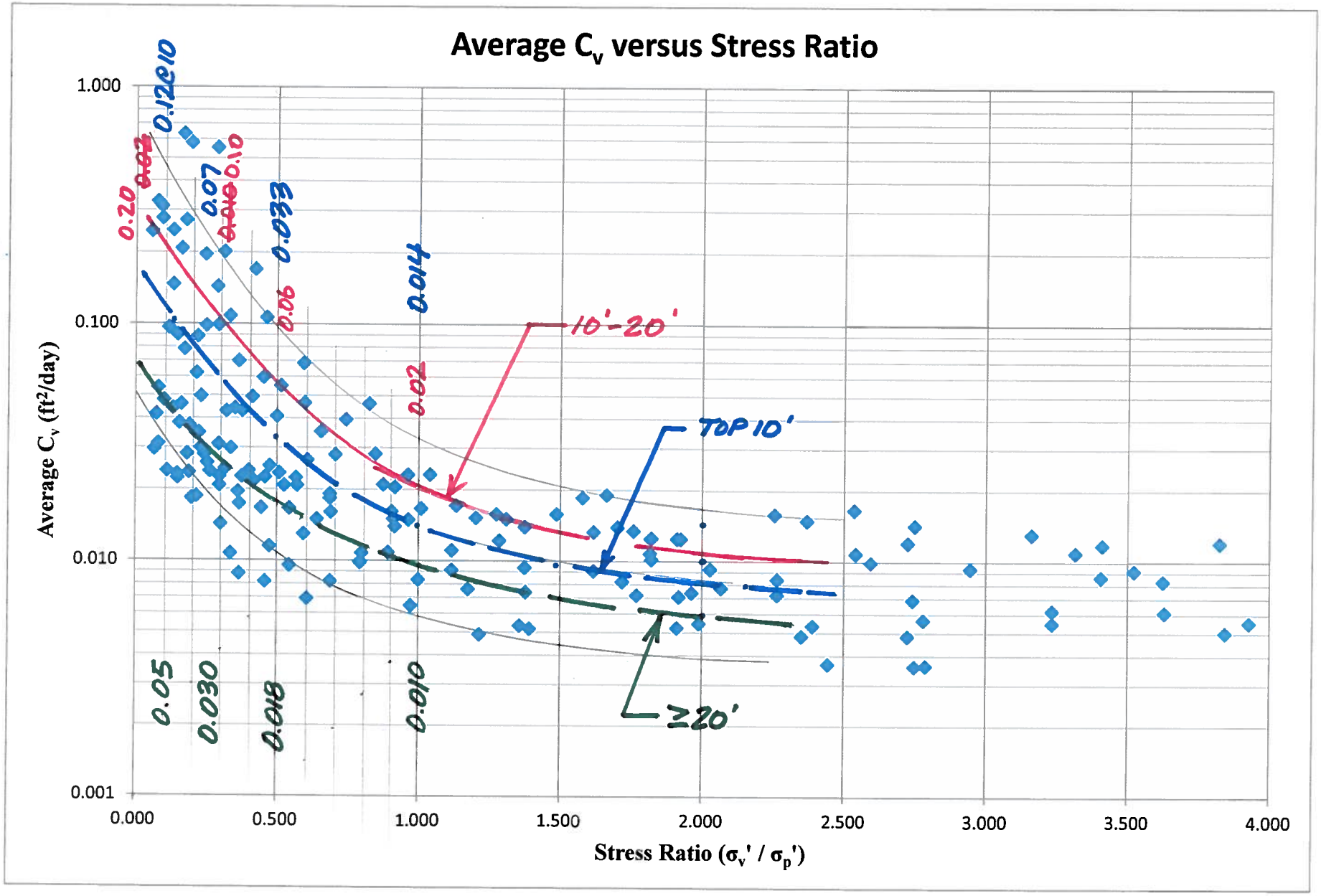

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CONSOLIDATION RATE
MARSH AREAS #1 & 2
MARSH CONST. GRADE +3.0' NAVD88
FOR TRANSFORMED PROFILE
$C_v = 0.025\ ft^2/day$ (LAB)
$H_{dr} = 41.0'$ (one way)

USE FIELD $C_v = 10 LAB\ C_v = 0.25\ ft^2/day$

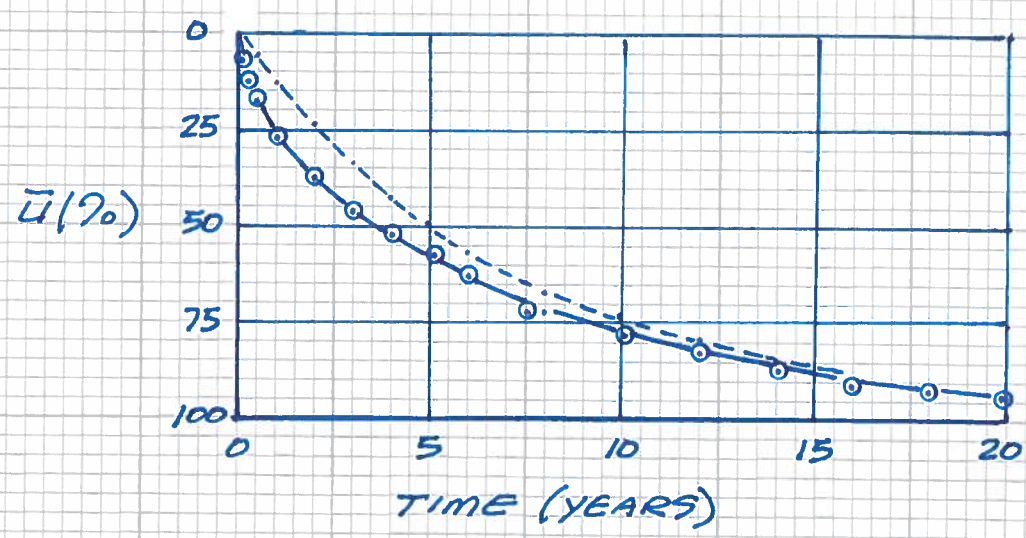
TERZAGHI 1-D SOLUTION

TIME, t (yrs)	T	$\bar{U}$ (%)	$\bar{U}^*$ (%)	$\bar{U}^{**}$ (%)
0.1	0.0054	8.3		
0.25	0.014	13.1		
0.5	0.027	18.6	5.7	
0.75	0.041	22.8	9.5	
1.0	0.054	26.3	12.8	
2	0.109	37.2	24.8	
3	0.163	45.6	34.5	
4	0.217	52.6	42.7	
5	0.271	58.5	49.9	
6	0.326	63.7	56.3	
8	0.434	72.2	66.5	
10	0.543	78.8	74.4	
12	0.651	83.7	80.4	
14	0.760	87.6	85.0	
16	0.868	90.5	88.5	
18	0.977	92.7	91.2	
20	1.086	94.4	93.3	

$T = \frac{C_v t}{41^2}$

FOR $\bar{U} < 53\%$: $T = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

FOR $\bar{U} > 53\%$:
$T = 1.781 - 0.933[\log(100 - \bar{U})]$



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FILE NO. _____
PROJECT _____

CONSOLIDATION RATE
MARSH AREAS 1+2
MARSH CONST. GRADE +2.0' NAVD88

C_v (lab) = 0.030 ft²/day
H_{dr} = 42.9'
FIELD C_v = 0.30 ft²/day

TIME, t (yrs)	T	$\bar{U}_p$ (%)
0.1	0.0059	8.7
0.25	0.0149	13.8
0.5	0.0297	19.5
0.75	0.0446	23.8
1	0.0595	27.5
1.5	0.0892	33.7
2	0.119	38.9
3	0.1785	47.7
4	0.238	54.9
5	0.2975	61.1
6	0.357	66.4
8	0.476	75.0
10	0.595	81.3
12	0.714	86.08
14	0.833	89.6
16	0.952	92.3
18	1.071	94.2
20	1.190	95.7

BY COMPARISON TO CONST. GRADE +3.0 FT NAVD88, RATE FOR +2.0 FT CASE IS ONLY VERY SLIGHTLY FASTER

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PROJECT _____

CONSOLIDATION RATE
MARSH #1 & #2
MARSH CONST. GRADE +2.0' NAVD88

0		
	OCR 19 → 5.2	C_v(lab) = 0.11 ft²/d
5		
	OCR 2.4 → 1.34	0.03
10		
	OCR 15.5 → 11.5	0.12
15		
	OCR 9.0 → 7.5	0.10
20		
	OCR 5.9 → 5.5	0.08
25		
	OCR 4.2 → 3.8	0.07
30		
	OCR 3.0 → 2.8	0.05
35		
	OCR 2.4 → 2.3	0.04
40		
	OCR 1.8 → 1.7	0.03
50		

5.5
0
2.6
7.6
10.1
12.8
15.9
19.2
23.1
27.4
37.4

H_{dr} = 42.9'

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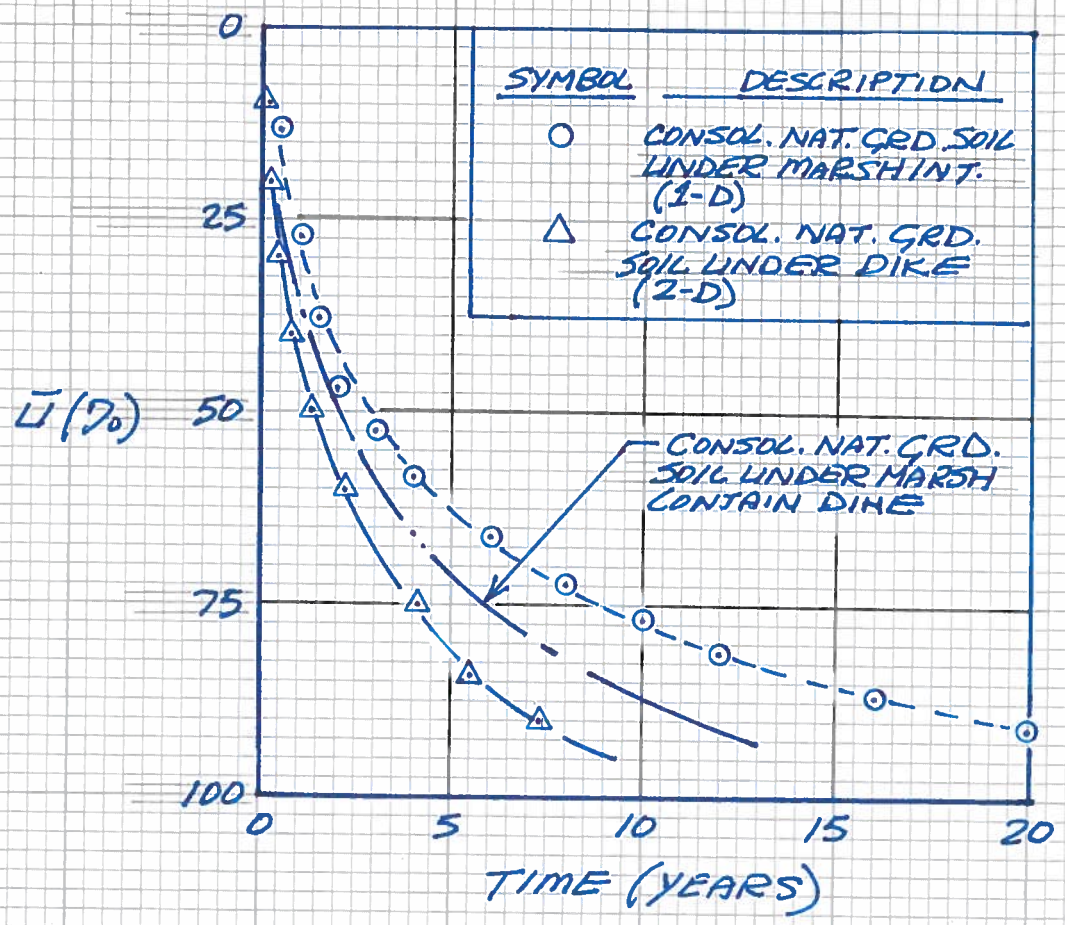
SUMMARY
CONSOL. CURVES - MARSH # 3



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CONSOLIDATION RATE
MARSH #3

TYPICAL PROFILE

	0	EL. -1.0	
1		OCR 5.7	$C_v = 0.10$ (lab) ft^2/d (NOTE 2)
	5		
2		OCR 2.4 → 1.0 (NOTE 1)	0.025
	10		
3		OCR 1.7 → 1.0	0.0.20
	15		
4		OCR 9.0 → 6.6	0.10
	20		
5		OCR 5.9 → 4.7	0.08
	25		
6		OCR 4.2 → 3.5	0.07
	30		
7		OCR 3.0 → 2.6	0.05
	35		
8		OCR 2.4 → 2.2	0.04
	40		
9		OCR 1.8 → 1.6	0.025
	50		

DESIGNATE NO-FLOW BOUNDARY @ 50'

TRANSFORMED PROFILE

8.9		
	DREDGED FILL	
0		
2.25	1	
6.75	2	
11.75	3	
14.0	4	
16.5	5	$h'_5 = 2.5'$ (NOTE 3)
19.2	6	
22.4	7	
25.9	8	
34.8	9	

NOTE 3: SUBLAYER THICKNESS TRANSFORMED TO EQUIV. h' FOR UNIFORM $C_v = C_{v3} = 0.02\ ft^2/day$

$$h'_5 = h_5 \left(\sqrt{C_{v3}} / \sqrt{C_{v5}}\right)$$

DREDGE FILL $C_v = 0.004\ ft^2/day$ FROM SLURRY CONSOL. TEST

$$h' = 4.0 \left(\sqrt{0.02} / \sqrt{0.004}\right) = 8.9'$$

$\Rightarrow$ H_{dr} FOR $C_v = 0.02 \rightarrow 43.7'$

NOTE 1: OVERCONSOLIDATION RATIO @ $\bar{U} = 0\% \rightarrow \bar{U} = 100\%$ FOR $\Delta\bar{\sigma}_v$ CORRESP. TO 4.0' DREDGED FILL (MARSH CONST. GRADE +3.0' NAVD88) SEE FIGURE AND TABLE

NOTE 2: STRESS RATIO = (1/OCR); C_v(lab) FROM FIGURE

0.204

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PROJECT COLE'S BAYOU

SHEET NO. _____ OF _____
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CHECKED BY _____ DATE _____
SCALE _____

CONSOLIDATION RATE
MARSH AREA #3

FOR TRANSFORMED PROFILE
$C_v = 0.02\ ft^2/day$ (LAB)
$H_{dr} = 43.7'$ (ONE WAY)
USE $C_v = 10$ LAB $C_v = 0.20\ ft^2/day$

TERZAGHI 1-D SOLUTION

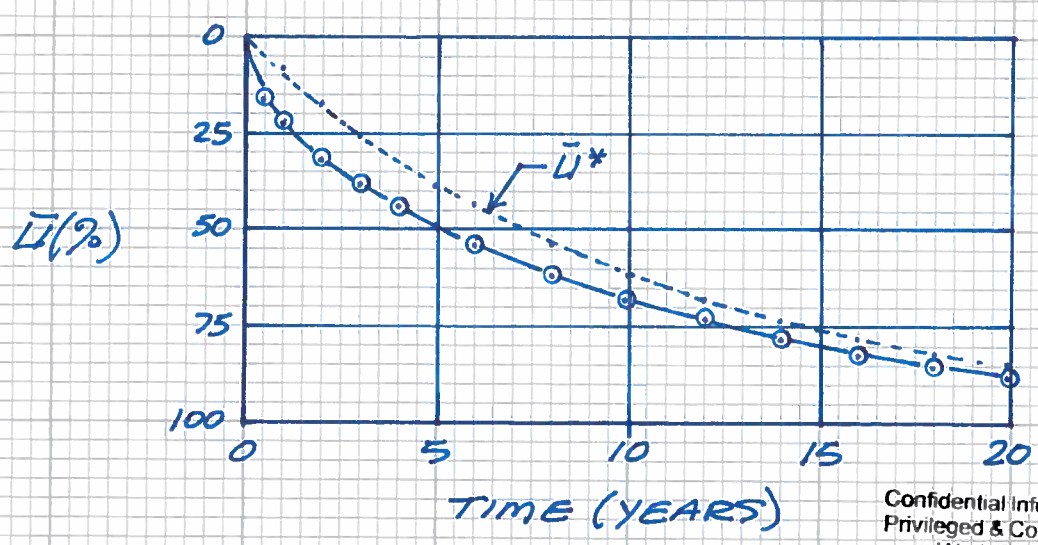
TIME t (yrs)	T	$\bar{U}$ (%)	$\bar{U}^*$ (%)	$\bar{U}^{**}$ (%)
0.1	0.0038	6.9		
0.25	0.0096	11.1		
0.50	0.0191	15.6	3.4	13.3
0.75	0.0287	19.1	6.1	21.5
1.0	0.038	22.0	8.7	27.7
1.5	0.057	26.9	13.6	37.0
2	0.0765	31.2	18.1	43.7
3	0.115	38.3	26.0	52.5
4	0.153	44.1	32.8	58.2
5	0.191	49.3	38.9	62.6
6	0.229	54.0	44.4	66.2
8	0.306	61.9	54.1	72.2
10	0.382	68.4	61.9	77.0
12	0.459	73.9	68.5	81.0
14	0.535	78.3	73.9	84.3
16	0.612	82.1	78.4	87.0
18	0.688	85.2	82.1	89.2
20	0.764	87.7	85.2	91.1

$$T = \frac{C_v t}{43.7^2}$$

FOR $\bar{U} < 53\%$: $T = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

FOR $\bar{U} > 53\%$:
$T = 1.781 - 0.933\left[\log(100 - U\%)\right]$

$\bar{U}^*$ = AVERAGE DEGREE OF CONSOLID. IN THE NATURAL GROUND CLAY UNDER THE DREDGED FILL



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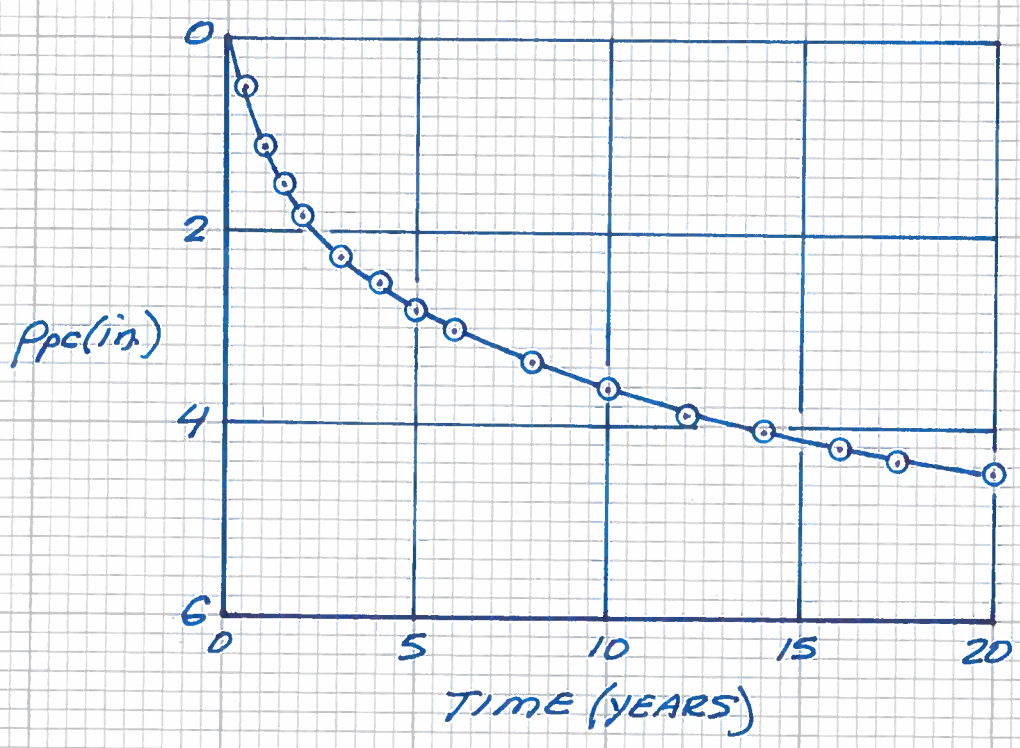
SETTLEMENT RATE
MARSH NO. 3 W/ CONST. GRADE +3.0' NAVD
NAT. GROUND CONSOL. SETTLEMENT

TIME, t (yrs)	U_z (TOP 15') (%)	ρ (TOP 15') (in.)	U_z (15 to 50) (%)	ρ (15 to 50) (in.)	TOTAL ρ (in.)
0.25	2.9	0.12	0	0	0.12 ✓
0.5	13.3	0.53	0	0	0.53 ✓
0.75	21.5	0.86	1.3	0.01	0.87 ✓
1	27.7	1.11	2.5	0.02	1.13 ✓
1.5	37.0	1.49	5.8	0.06	1.55 ✓
2	43.7	1.76	9.5	0.09	1.85 ✓
3	52.5	2.11	17.0	0.16	2.27
4	58.2	2.34	24.1	0.23	2.57 ✓
5	62.6	2.52	30.8	0.30	2.82 ✓
6	66.2	2.66	37.0	0.36	3.02 ✓
8	72.2	2.90	47.8	0.46	3.36
10	77.0	3.10	56.7	0.54	3.64 ✓
12	81.0	3.26	64.2	0.62	3.88
14	84.3	3.39	70.3	0.67	4.06
16	87.0	3.50	75.5	0.72	4.22
18	89.2	3.59	79.7	0.76	4.36
20	91.1	3.66	83.1	0.80	4.45 ✓



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SECONDARY COMPRESSION (CREEP)
MARSH NO. 3
NATURAL GROUND CREEP, ρ_{sc}

$\bar{U} \sim 88\%$ @ 20 YRS (OVERALL AVG.)

SUBLAYER NO.	DEPTH (ft)	T for $U_z = 90\%$	t(yrs) $U_z = 90\%$
1	0-5	0.65	17
2	5-10	0.78	20.5
3	10-15	0.87	~23
4	15-20		

⇒ SMALL CONTRIBUTION TO OVERALL SETTLEMENT

FILL SECONDARY COMPRESSION + SHRINK

$C_{\alpha\varepsilon} \approx 0.046\,CR$

SAY $CR = 0.2 \longrightarrow C_{\alpha\varepsilon} = 0.010\ \%/\log t$

$\log\left(\frac{20\,yrs}{0.2\,yrs}\right) = 2 \Rightarrow 2\%$ (FILL Ht) CREEP

SHRINKAGE USE 8% (FILL Ht) - DIKE

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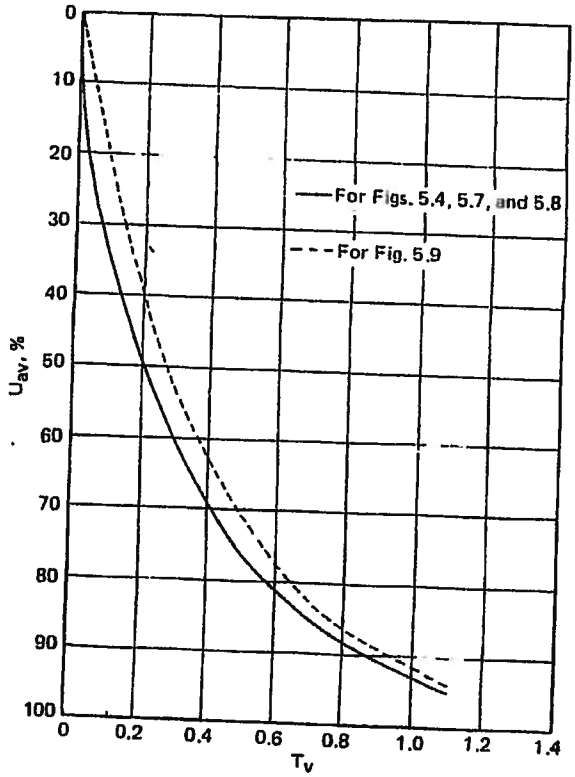

Fig. 5.6 Variation of average degree of consolidation (for conditions given in Figs. 5.4, 5.7, 5.8, and 5.9).

or
$$T_v = \frac{(\pi/4)\,(U_{av}\%/100)^2}{[1-(U_{av}\%/100)^{5.6}]^{0.357}} \tag{5.38}$$

Equations (5.37) and (5.38) give an error in T_v of less than 1% for $0\% < U_{av} < 90\%$ and less than 3% for $90\% < U_{av} < 100\%$.

Table 5.1 Variation of T_v with U_{av} [Eq. (5.34)]

U_{av}, %	T_v	U_{av}, %	T_v
0	0	60	0.287
10	0.008	65	0.342
20	0.031	70	0.403
30	0.071	75	0.478
35	0.096	80	0.567
40	0.126	85	0.684
45	0.159	90	0.848
50	0.197	95	1.127
55	0.238	100	∞

FIGURE 3

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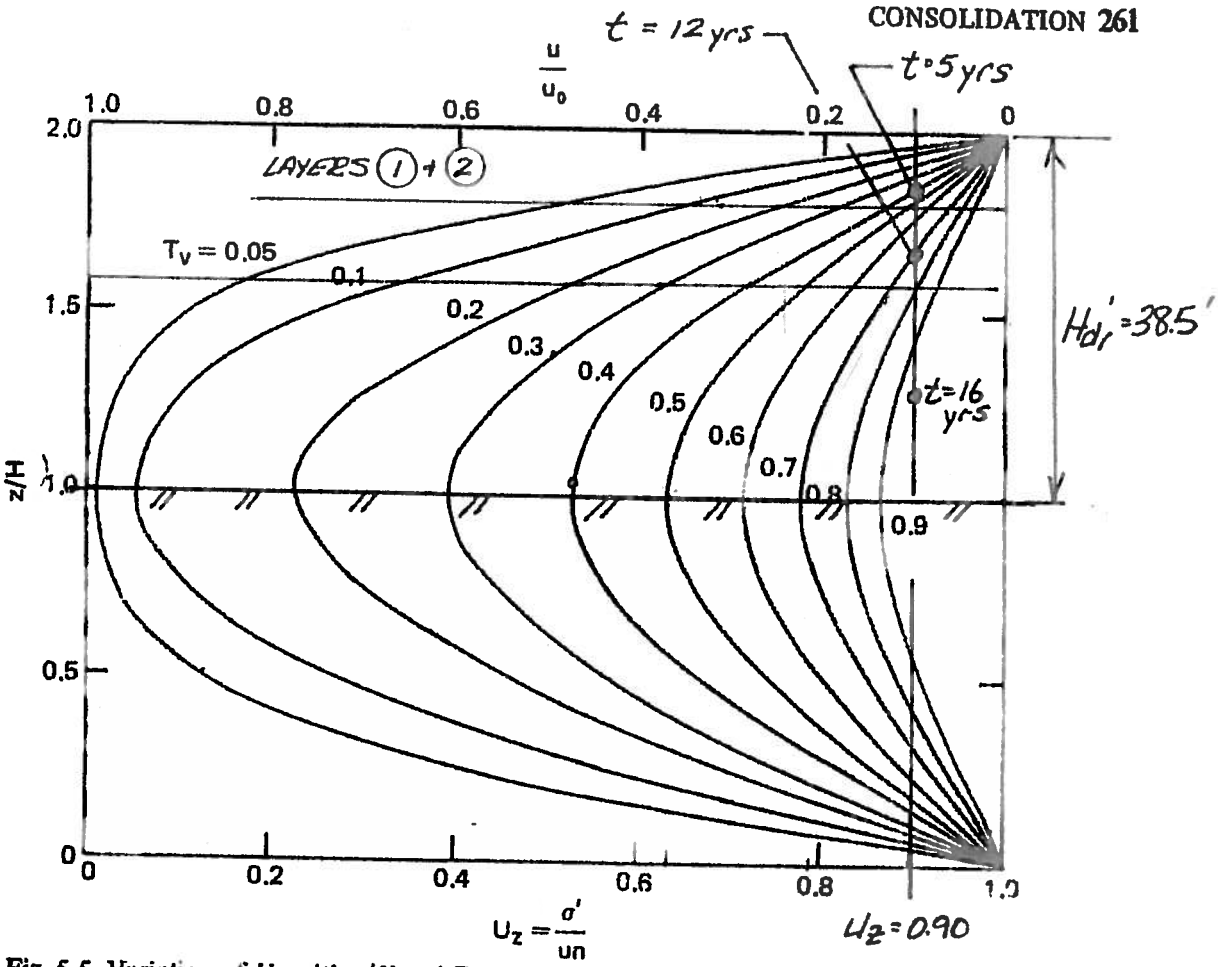

Fig. 5.5 Variation of U_z with z/H and T_v.

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FIGURE 4

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CONSOLIDATION OF A LAYER UNDER A STRIP LOAD

By John T. Christian,[1] M. ASCE, Jan Willem Boehmer,[2] and Philippe P. Martin[3]

INTRODUCTION

In the design of construction on compressible ground it is customary to distinguish three phases of settlement: initial, primary, and secondary. By using elastic theory, oedometer tests, and, recently, finite element methods, engineers have been able to make quite sophisticated analyses of initial settlement and of the magnitude of final consolidation settlement. For example, Hoeg, et al. (6) and D'Appolonia, et al. (3) have presented results showing the effect of inelastic behavior in the undrained case. Duncan and Chang (4) have described reasonably simple methods for including nonlinear stress-strain relations in analyses of final settlements. The process of secondary compression and the rate of primary consolidation, in particular as they are affected by the ratio of load width to depth of consolidating layer, are much less well understood. This paper uses the finite element method to investigate the effect of load width and material properties on the rate of primary consolidation. It makes recommendations as to how these factors can be considered in engineering design.

The particular problem studied is that of a uniform strip load of which $2a$ on the surface of a uniform soil of depth H. The bottom of the soil is fixed to a rough, rigid base so that no vertical or horizontal movement can occur at the bottom. Unless otherwise specified, drainage is allowed at the top surface of the soil but not at the bottom.

The analysis was carried out by the finite element method applied to Biot's (1) theory of the consolidation of an elastic soil. The descriptions of the com-

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[1] Assoc. Prof. of Civ. Engrg., MIT, Cambridge, Mass.
[2] Engr., Rijkswaterstaat, The Hague, Netherlands; formerly Research Engr., MIT, Cambridge, Mass.
[3] Grad. Student, Dept. of Civ. Engrg., MIT, Cambridge, Mass.

puter program CONSOL and of the tests of its accuracy have already been presented by the senior writers (2). In general it was found that the program gave excellent results except when the bottom boundary was supposed to be at infinity, a condition that does not arise here.

The results of the analyses indicate a complicated interaction among three factors: (1) The geometry of the load; (2) the Poisson's ratio of the soil skeleton; and (3) the ratio of horizontal to vertical permeability. The factor having the largest effect is the horizontal to vertical permeability ratio; however, this factor is important primarily at early times in the consolidation process.

BASIC PROBLEM AND BOUNDARY CONDITIONS

The basic problem of the consolidation of a uniform layer under a strip load is shown in Fig. 1. For the initial analyses a/H was equal to 1.0. The

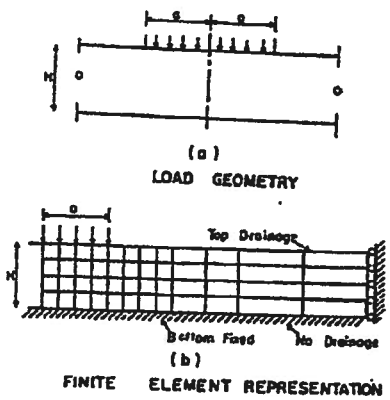

FIG. 1.—PROBLEM GEOMETRY AND FINITE ELEMENT REPRESENTATION

properties of the soil skeleton are described by three material constants: the Young's Modulus, $\bar{E}$, the Poisson's ratio, $\bar{\nu}$, and the permeability, k.

In the Terzaghi one-dimensional theory the development of consolidation is expressed in terms of the dimensionless time factor, T:

$$T = \frac{c_{v1} t}{H^2} \quad \text{(1)}$$

in which c_{v1} = the one-dimensional coefficient of consolidation; t = real time; and H = the drainage path. The coefficient of consolidation is defined by

$$c_{v1} = \frac{k}{\gamma_w} \frac{1}{m_v} \quad \text{(2)}$$

In this equation γ_w = the unit weight of water and m_v = the coefficient of volume compressibility.

As the Terzaghi theory assumes m_v is a constant and as m_v is measured in the oedometer test, which prevents lateral motion, it follows that m_v is simply the inverse of the constrained elastic modulus, $\bar{D}$, of the soil skeleton:

$$\frac{1}{m_v} = \bar{D} = \frac{\bar{E}(1 - \bar{\nu})}{(1 + \bar{\nu})(1 - 2\bar{\nu})} \quad \text{(3)}$$

In practice, of course, m_v is chosen to reflect the range of stresses expected in the field, but the fundamental assumption remains in the Terzaghi theory that the skeleton is elastic.

Use of several finite element arrays revealed that when the distance from the center line to the smooth, rigid lateral boundary was changed from $2H$ to $5H$, there was very little effect on the computed rate of settlement. With the lateral boundary at $5H$, changing the drainage condition there from total impermeability to free drainage had no noticable effect on the rate of settlement. Therefore, a rigid, smooth, impervious boundary at $5H$ from the center line was regarded as an adequate representation of a layer of infinite lateral extent.

The results of the previous paragraph do not apply when the bottom is smooth. Then the lateral boundary is the main component preventing lateral deformation, while in the case with the rough bottom the bottom restrains lateral movement. The nature and location of the lateral boundary are always important in the case of the smooth bottom. Fortunately, the rough bottom, which is most likely to represent soil conditions, allows the lateral boundary to be placed reasonably near and still have a negligible effect. Gibson, et al. (3) have also observed that analytical solutions for a smooth bottom give anomalous results.

GENERAL NATURE OF RESPONSE

In order to illustrate the general nature of the response, material constants were chosen as follows: $\bar{E}$ = 1,000 kg/cm²; $\bar{\nu}$ = 0.0; and $k = 10^{-9}$ cm/sec. These give c_{v1} = 1.0 cm²/sec. Poisson's ratio, $\bar{\nu}$, was made 0.0 to maximize the effect of changes in compressibility during consolidation. The effect of using different values of $\bar{\nu}$ is investigated in the following.

The displacements during consolidation under a strip load of 1 kg/cm² are shown in Fig. 2, exaggerated by a factor of 50 and plotted with the original finite element array. The initial displacements involve downward motion under the load and a general horizontal displacement away from the loaded area. The upward motion at the surface just outside the loaded area is increased somewhat by the rigid lateral boundaries. During consolidation the material settles further under the load and moves horizontally toward the load. At all times the settlements are bowl-shaped.

If the same load were applied over the entire surface, the problem would be identical to Terzaghi's one-dimensional consolidation. The final settlements would then be $H \times 10^{-3}$. In the present strip loading case the final settlement under the center line of the load is $1.0H \times 10^{-3}$ because conditions at the center of the load are very similar to those in the one-dimensional problem.

The development of the settlements, ρ, at the center of the load and at the edge is plotted in Fig. 3. The actual displacements are divided by the final

The second cause of change in σ_{vol} is that for any elastic element in plane strain:

$$\sigma_{vol} = \frac{(\sigma_x + \sigma_y)(1 + \nu)}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which ν = the Poisson's ratio for total stresses. Thus, even if the in-plane

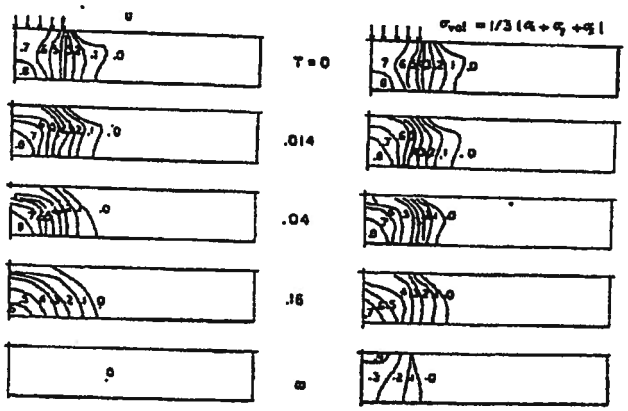

FIG. 5.—STRESSES FOR STANDARD PROBLEM

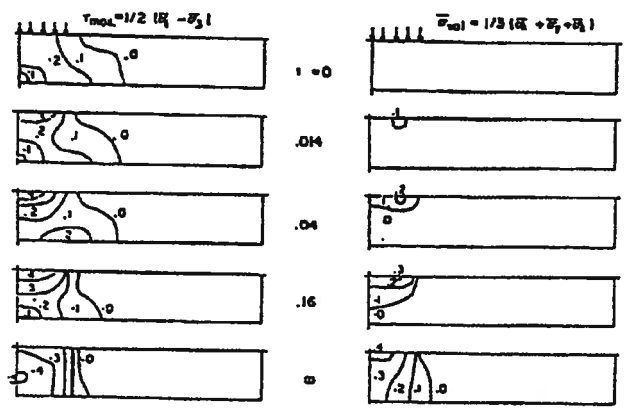

FIG. 6.—STRESSES FOR STANDARD PROBLEM

stresses remain unchanged, the σ_{vol} must change as ν changes from 0.5 to 0.0.

The $\bar{\sigma}_{vol}$ in Fig. 6 starts to develop near the load edge and spreads out from there. It can also be seen in Fig. 5 that both σ_{vol} and u increase directly under the center of the load at early times even though no significant change in $\bar{\sigma}_{vol}$ has yet occurred there. This is called the Mandel-Cryer effect. Maximum shear stresses increase during consolidation and develop a final maximum under the center line near the bottom. This means there is an increase in shear deformation and in shear strain during consolidation. The final settlement thus includes both volumetric and deformation effects.

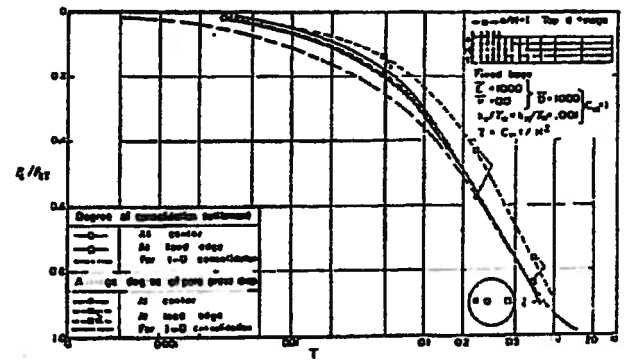

FIG. 7.—CONSOLIDATION SETTLEMENT AND PORE PRESSURE DISSIPATION AT CENTER LINE AND LOAD EDGE OF STANDARD PROBLEM

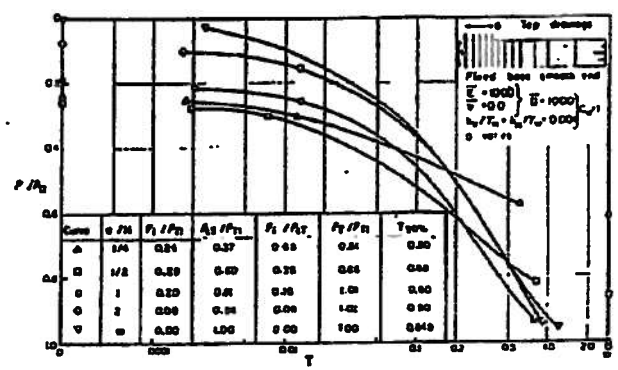

FIG. 8.—SETTLEMENT AT CENTER LINE FOR DIFFERENT LOAD WIDTHS

Rates of consolidation settlement (ρ_c/ρ_{cT}) and rates of pore pressure dissipation averaged over a vertical section are compared in Fig. 7 for the

total settlement in the one-dimensional case (ρ_{T_1}) to obtain a comparison with that case. [This is a semilogarithmic plot, but in order to show initial and final conditions additional points have been plotted at the left and right ($t = 0$ and ∞).] Although the final settlement of the center of the strip load is virtually identical to that for the one-dimensional case, only 81 % of that settlement is caused by consolidation under the strip load. The rest is initial settlement due to deformation without change of volume. The final settlement at the load edge is 50 % of the one-dimensional settlement. During consolidation, the differential settlements increase from less than 20 % of the final settlement to 50 % of it. The rates of settlement during the middle and final stages of consolidation are rather close for the one-dimensional case and for the center of the load.

The development of settlement under the strip load is further shown in Fig. 4. The initial outward movement is followed by inward consolidation

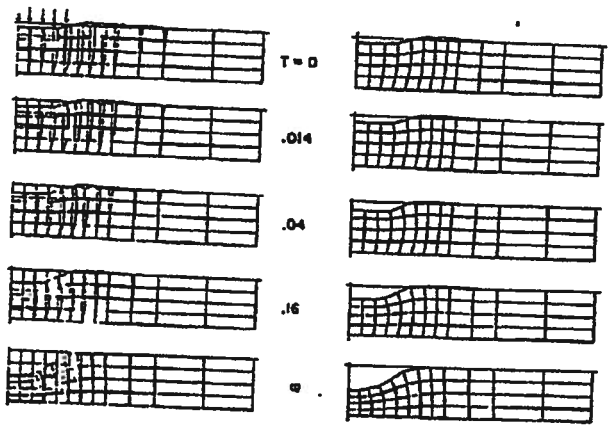

FIG. 2.—DISPLACEMENTS OF STANDARD PROBLEM

movement, and it can be seen that differential settlements increase gradually during consolidation. The inward motion during consolidation could be counteracted by outward motion due to plastic flow. Then an inclinometer would be motionless even if plastic failure were about to occur. Field control of construction is greatly complicated by this fact.

Figs. 5 and 6 present contours of significant stress parameters as they change during consolidation. As the initial stresses (before loading) have no effect on the solution, only the increments are considered in the following. As would be expected, the initial $\bar{\sigma}_{vol}$ is zero everywhere, the final σ_{vol} and $\bar{\sigma}_{vol}$ are equal, and the final u is zero. The process involves a transfer of stress from pore pressure, u, to effective stress $\bar{\sigma}_{vol}$. However, the initial and final σ_{vol} are not the same.

The explanation of the change in σ_{vol} during consolidation requires an understanding of the elastic behavior with respect to total stress. For the drained final condition the effective stresses and the total stresses minus

static pore pressures are identical, so the elastic constants are $\bar{E}$ and $\bar{\nu}$. Before consolidation starts, the soil is incompressible, but it does react elastically to total stresses. In particular, the Poisson's ratio for undrained

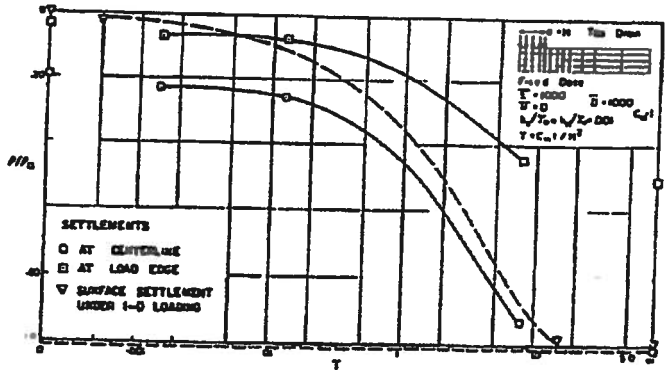

FIG. 3.—SETTLEMENTS OF A FINITE LAYER—STRIP LOAD AND UNIFORM LOADING

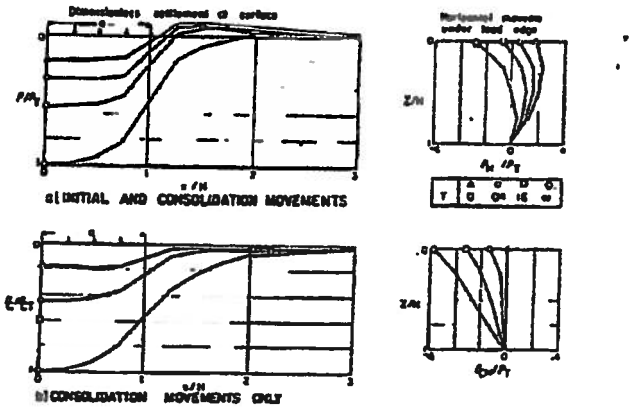

FIG. 4.—TOTAL MOVEMENTS AND CONSOLIDATION MOVEMENTS FOR STANDARD PROBLEM

behavior is 0.5. Consolidation can therefore be considered a process in which the elastic constants with respect to total stress change during dissipation of excess pore pressures. The rigid bottom boundary requires changes in in-plane stresses when Poisson's ratio changes, as is shown for τ_{max} in Fig. 6.

load center and the load edge. In the one-dimensional case of uniform loading over the entire surface, these rates would be identical. In the case of the strip load with $\bar{\nu} = 0$ the differences are quite small. Two adjacent columns of elements were used for the load edge as there are no element centers directly under it. Their average would indicate there is some lag in pore pressure dissipation at the load edge.

EFFECT OF LOAD WIDTH

The effect of the width of the loaded strip on the magnitudes of the initial and final settlement involves several interacting factors. As the load width becomes very large ($a/H \to \infty$), the initial settlement goes to zero, and the final settlement is proportional to H. The other extreme ($a/H = 0$) can be in-

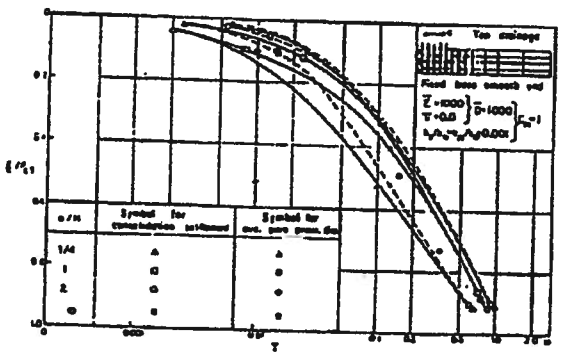

FIG. 9.—TIME LAGS IN CONSOLIDATION AND PORE PRESSURE DISSIPATION UNDER CENTER LINE

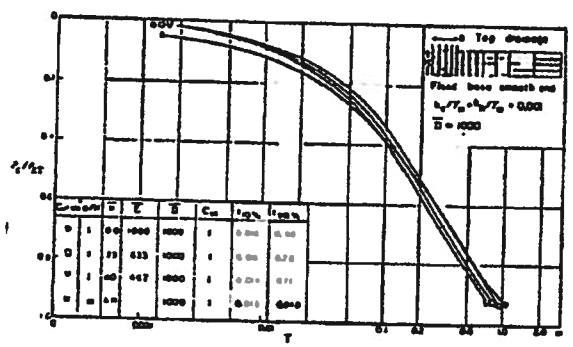

FIG. 10.—CONSOLIDATION SETTLEMENT AT CENTER LINE FOR VARIOUS $\bar{\nu}$ AND CONSTANT $\bar{D}$

terpreted as the case of a strip load on an infinitely thick layer ($H = \infty$). Then the settlements must be infinite. A more useful approach is to consider the thickness of the layer as fixed and to vary the width for a uniform strip loading. It would then be expected that the initial and final settlement for very small a/H would be approximately proportional to a, reaching a limit of zero when $a = 0$. At an intermediate value of a/H both the initial and final settlements would be finite.

The result of this interaction can be seen in Fig. 8, where settlements have been normalized by dividing them by the one-dimensional total settlement (ρ_{T_1}). The ratio a/H has a major effect on ρ_i and on ρ_T which in this plot masks the effect on rate of consolidation. Fig. 9 shows the consolidation effects alone, and it can be seen that as a/H becomes smaller, the rate of consolidation becomes faster. This is because the initial excess pore pressures are distributed in a smaller area and have more space into which to dissipate. There is one apparent anomaly in these two figures: the time for 90 % consolidation is slower for a/H of 2 than for a/H of ∞. Even for a/H of 2 there is some coupling between dissipation and total stress leading to a small Mandel-Cryer effect and a slowing of consolidation. When the initial settle-

TABLE 1.—EFFECT OF a/H ON TIME FACTOR

a/H (1)	T_{10} (2)[a]	T_{50} (3)	T_{75} (4)	T_{90} (5)
1/4	0.007	0.09	0.30	0.60
1/2	0.011	0.14	0.35	0.68
1	0.016	0.20	0.45	0.80
2	0.016	0.24	0.53	0.90
∞	0.008	0.197	0.48	0.818

[a] T_{10} = time factor at 10 % of consolidation under center line.

ments are added in as in Fig. 8, the settlements for $a/H = 2$ and $a/H = \infty$ are nearly identical throughout most of the consolidation history.

The average degree of dissipation of excess pore pressure near the vertical center line is compared to the degree of consolidation settlement in Fig. 9. Clearly there is almost no lag for $a/H = 2$ through $a/H = \infty$. However, there is a sizeable lag in pore pressure dissipation when $a/H = 0.25$.

Table 1 summarizes the effect of a/H on the rate of primary consolidation. When $a/H = 0.5$ the time factor for 90 % of consolidation settlement (T_{90}) at the center of the strip load is reduced to about 85 % of its value at $a/H = 1$. When $a/H = 0.25$, T_{90} is reduced to 75 % of its value at $a/H = 1$. Similar analyses with $\bar{\nu} = 0.25$ and 0.40 revealed that the percentage reductions are largely independent of $\bar{\nu}$.

EFFECT OF ELASTIC CONSTANTS

Fig. 10 shows the rate of development of consolidation settlement for various values of $\bar{\nu}$ while $\bar{D}$ (or m_p) is kept constant. It can be seen that $\bar{\nu}$ has very

little effect on the results. Varying $\bar{\nu}$ with a constant $\bar{D}$ means that the shear modulus, $\bar{G}$, and the Young's modulus, $\bar{E}$, must change. However, $\bar{D}$ or m_v is the most easily evaluated modulus, and it is of practical significance that uncertainties in $\bar{\nu}$ do not make much difference. Poisson's ratio, $\bar{\nu}$, does have an effect on ρ_f and therefore on ρ_T.

The time factor T might be defined in other ways. It can be defined in plane strain as

$$c_{v2} = \frac{k}{\gamma_w} \bar{M} \quad \text{(5)}$$

in which $\bar{M}$ = the modulus relating the average in-plane stress $(\sigma_1 + \sigma_2)/2$ to the volumetric strain:

$$\bar{M} = \frac{\bar{E}}{2(1 + \bar{\nu})(1 - 2\bar{\nu})} \quad \text{(6)}$$

It could be defined for a general three-dimensional case as

$$c_{v3} = \frac{k}{\gamma_w} \bar{K} \quad \text{(7)}$$

in which $\bar{K}$ = the bulk modulus:

$$\bar{K} = \frac{\bar{E}}{3(1 - 2\bar{\nu})} \quad \text{(8)}$$

As any of these three definitions of c_v could be used, the effect of varying the elastic constants enters into the time factor in a way that is not immediately obvious. All the moduli are proportional to $\bar{E}$, so regardless of which c_v is used to evaluate T, T will be proportional to $\bar{E}$ and so will the rate of consolidation.

The effects of varying $\bar{\nu}$ must depend on which modulus is kept constant or which definition of c_v is used. Its influence was determined by a parametric study, in which runs were made with different values of a/H and $\bar{\nu}$. The use of c_{v2} or c_{v3} only led to greater scatter in the ρ_c/ρ_{cT} curves. This indicates that the uncertainty in evaluating $\bar{\nu}$ can be overcome by using c_{v1} in computing the time factor T. In the practical range of $\bar{\nu}$ = 0.25 to 0.40 there is almost no difference between the consolidation rates when c_{v1} is used. When a/H is less than 0.25, another method of evaluating c_v may be appropriate, but there are no data on this at present.

EFFECTS OF ANISOTROPIC PERMEABILITY

If the permeability is anisotropic, there must be an effect of the anisotropy on the rate of consolidation. This was investigated by making computer runs with the ratio of horizontal to vertical permeability (k_H/k_V) equal to 4 and 16. Also Ladd and Wissa (4) have used the same computer program CONSOL to investigate the behavior of an embankment on soft clay with k_H/k_V = 1, 10, and 100. Their results are also used herein. Both single drainage at the top and double drainage at top and bottom were used.

Obviously, the magnitude of initial and final deformations is independent of permeability. The only question is how does the ratio of k_H/k_V affect the rate

of consolidation, and this can be expressed by defining an effective permeability k_{eff} as that isotropic permeability which would cause the same percentage of consolidation settlement at the same time as is observed in the anisotropic case.

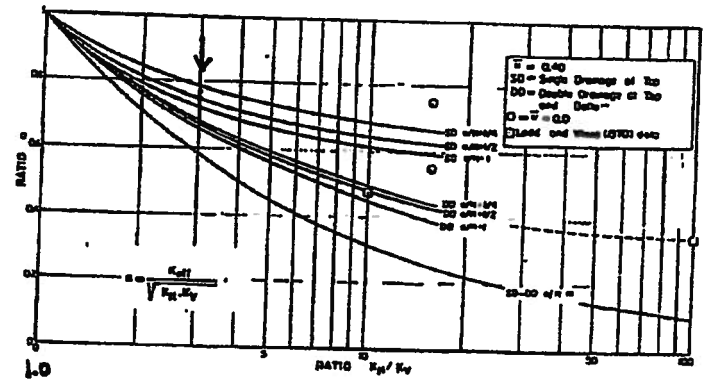

FIG. 11.—α VERSUS k_H/k_V AT 85 % CONSOLIDATION

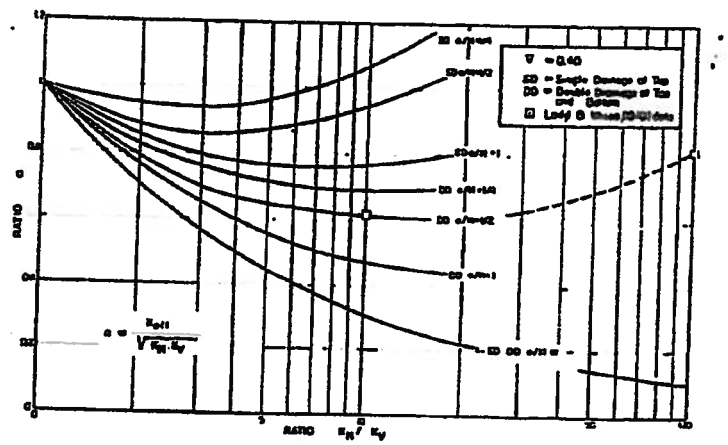

FIG. 12.—α VERSUS k_H/k_V AT 50 % CONSOLIDATION

From the theory of steady state flow through porous media an initial candidate for k_{eff} is $\sqrt{k_H k_V}$. Thus the ratio of the actual k_{eff} to $\sqrt{k_H k_V}$ is a useful measure of the effectiveness of the anisotropic permeability. This ratio is

defined as α:

$$\alpha = \frac{k_{eff}}{\sqrt{k_H k_V}} \quad \text{(9)}$$

For a uniform load over the entire surface ($a/H = \infty$) the effective permeability at any percentage of consolidation is clearly simply k_V, so

$$\alpha = \sqrt{\frac{k_V}{k_H}} \quad \text{(10)}$$

This is true for single drainage at the top and for double drainage at the top and bottom.

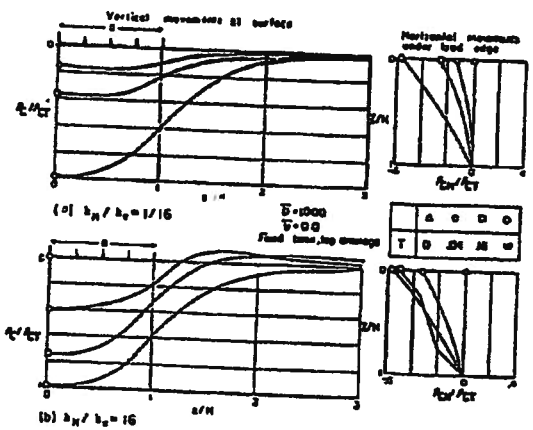

FIG. 13.—EFFECT OF k_H/k_V ON CONSOLIDATION MOVEMENTS

The results of evaluating α for 85 % of consolidation are plotted in Fig. 11. Ladd and Wissa's computations were for an embankment with a/H nearly = 0.5. All the results are for $\bar{\nu} = 0.4$; when $\bar{\nu} = 0.0$ there is somewhat more range of results as indicated by the circles. In all cases α is less than one, indicating that horizontal drainage is less effective than one would expect in accelerating the time for 85 % consolidation.

If the same plot is made for α at 50 % consolidation, the different picture of Fig. 12 appears. Especially for single drainage, high horizontal permeability is quite effective in the early stages of consolidation.

The reason for this is that early in the consolidation history the excess pore pressures are concentrated directly under the load. High horizontal permeability allows them to drain laterally and thus to accelerate consolidation. Once they have drained laterally, they must dissipate vertically, and the

rate of consolidation decreases at later times. Indeed, in runs with $\bar{\nu} = 0$, there was significant surface heave at early times with $k_H/k_V = 10$ (Fig. 13).

CONCLUSIONS

The finite element analysis of the consolidation of an elastic layer with a rough base yields the following conclusions:

1. The time for consolidation settlement is virtually the same for $a/H = 1$ or greater.
2. When a/H is smaller than 1 there is some acceleration of the rate of consolidation, but at $a/H = 0.25$ the time for 90 % of consolidation is still about 75 % of what it was at $a/H = 1$.
3. Poisson's ratio has relatively little effect on the rate of consolidation if the conventional c_{v1} is used in evaluating T.
4. The time factor for 90 % consolidation is about .75 for $a/H = 1$ and $\bar{\nu}$ between 0.25 and 0.40. This is not very far from 0.848 for the one-dimensional case.
5. Because of the preceding the engineer can use the results of one-dimensional theory to predict rates of consolidation settlement of strip loads on uniform soil with a rough base. He should use the conventional one-dimensional theory when $a/H > 1$. When $a/H \leq 1$, he should reduce the time for consolidation according to Table 1.
6. The effect of anisotropic permeability can be accounted for from the curves of Figs. 1 and 2. Especially at late times, high horizontal permeability has less effect than one would expect from $\sqrt{k_H/k_V}$.

The preceding results can be summarized in an example or two. Suppose $a/H = 1$, $k_H/k_V = 10$, and there is double drainage. The time for 90 % consolidation can be found by first evaluating t_∞ from one-dimensional theory (i.e. from $T_\infty = 0.848$). If this is t_{1D}, the correct t_{2D} can be found from Table 1 and Fig. 11:

$$\left.\begin{aligned} t_{2D} &= t_{1D} \frac{T_\infty \text{ for } \frac{a}{H} = 1}{T_\infty \text{ for } \frac{a}{H} = \infty} \frac{1}{\alpha \sqrt{\frac{k_H}{k_V}}} \\ &= t_{1D} = \frac{0.80}{0.848} \frac{1}{0.45\sqrt{10}} = 0.663\, t_{1D} \end{aligned}\right\} \quad \text{(11)}$$

A second example is a case with $a/H = 1/2$, $k_H/k_V = 5$, and double drainage. Then at 50 % consolidation:

$$t_{2D} = t_{1D} \frac{0.14}{0.197} \frac{1}{0.85\sqrt{5}} = 0.49\, t_{1D} \quad \text{(12)}$$

The numerous finite element programs now in existence can be used to extend this work for other geometries. It is to be hoped that this will be done and published. In specific instances it may be possible for the engineer to use such programs for actual design analyses, but the running times are so large

that it would be most helpful to have available the results of further parametric studies.

Finally, it should be pointed out that these results may not function well if there is considerable plastic or viscous flow. To the writers' knowledge, solutions in such cases are not now available.

ACKNOWLEDGMENT

A portion of the work was performed under research contract C-85-65 with the Office of High Speed Ground Transportation, U.S. Department of Transportation, and this support is gratefully acknowledged. The writers are also thankful to A. E. Z. Wissa and C. C. Ladd for the use of their data and R. V. Whitman for his criticism of the drafts of this paper.

APPENDIX I.—REFERENCES

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APPENDIX II.—NOTATION

The following symbols are used in this paper:

a = half width of uniform strip load;
c_v, c_{v1} = coefficient of consolidation in Terzaghi Theory;
c_{v2}, c_{v3} = coefficient of consolidation based on plane strain and bulk moduli, respectively;

$\overline{D}$ = constrained modulus of soil skeleton;
$\overline{E}$ = Young's modulus of soil skeleton;
H = thickness of layer;
$\overline{K}$ = bulk modulus of soil skeleton;
k = permeability of soil;
k_H, k_V = permeability in horizontal and vertical directions, respectively;
k_{eff} = effective permeability in anisotropic soil;
$\overline{M}$ = plane strain modulus of soil skeleton;
T = dimensionless time factor;
t = real time;
u = pore pressure;
α = ratio between k_{eff} and $\sqrt{k_H k_V}$;
γ_w = unit weight of water;
ν = Poisson's ratio defined for total stresses;
$\overline{\nu}$ = Poisson's ratio defined for effective stresses, i.e., soil skeleton;
ρ = settlement;
ρ_c = consolidation settlement at any time;
ρ_{cT} = total or final consolidation settlement;
ρ_i = initial undrained settlement;
$\rho_T = \rho_{cT} + \rho_i$ = total settlement;
ρ_{T_1} = total settlement in one-dimensional case;
σ_{vol} = volumetric total stress;
$\overline{\sigma}_{vol}$ = volumetric effective stress;
σ_x, σ_y = normal stresses in x and y directions; and
τ_{max} = maximum shear stress.

DREDGED FILL SETTLEMENT CALCULATIONS

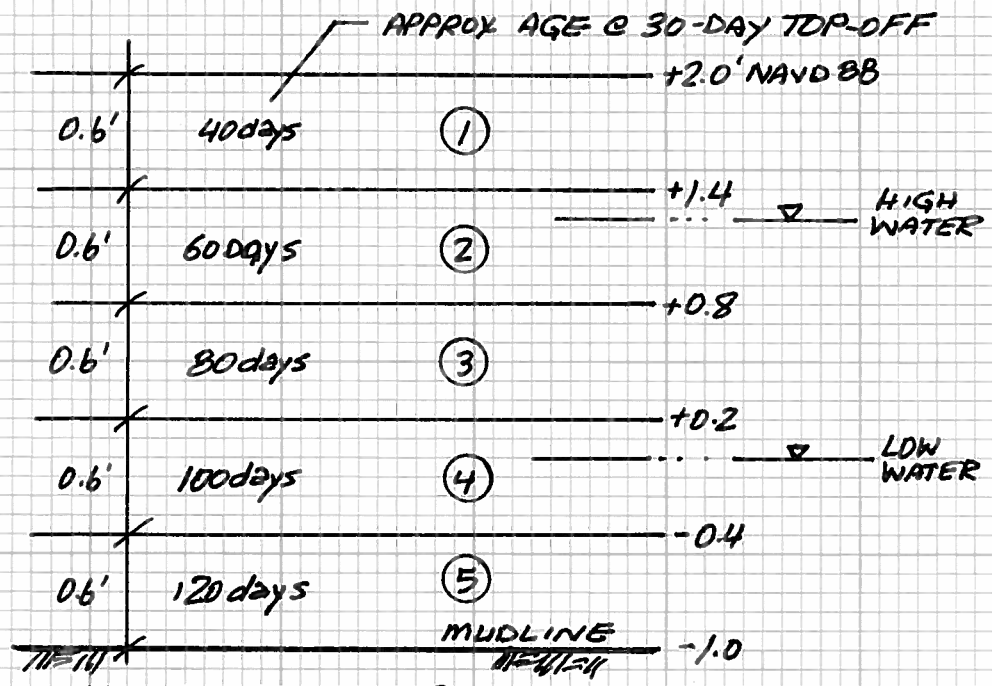
ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705
PROJECT COLE'S BAYOU

SHEET NO. 1 OF
CALCULATED BY RW DATE 4 2 1
CHECKED BY AT
SCALE

MARSH CREATION AREA
DREDGE FILL CONSOLIDATION
AREA #3

AVG. MUDLINE EL. -1.0'
CONST. GRD. EL. +2.0'
"LOW WATER" 2014 = -0.01
"HIGH WATER" 2014 = +1.26
20-YR RSLR = 0.6902 ft.



FROM SLURRY CONSOL. DATA (APPX. C)
C_v (field) ~ C_v (lab) ~ 0.015 ft²/day
H_{dr} = 3'

$C_{\alpha E}/C_R$ = 0.036
C_R ~ 0.25
$C_{\alpha E}$ = 0.009

TIME POST-DREDGE (DAYS)	T	U %
(30)	0.05	25 (OCCURS PRIOR TO 30-DAY TOP-OFF)
40	0.066	29
90	0.15	43.7
120	0.20	50.5
180	0.30	61.3
275	0.46	73.9
365	0.61	82.0
550	0.917	91.6
730	1.217	96.0

ARDAMAN & ASSOCIATES, INC.

SHEET NO. 2 OF

CALCULATED BY RW DATE 4/24/14

FILE NO. 13-80-3705

CHECKED BY DATE

PROJECT COLE'S BAYOU

SCALE

SUBLAYER	e_o*	e_f**	γ_{df} (pcf)	γ_{tf} (pcf)	$\bar{\sigma}_v$ (psf)	e_{pc}	ρ_{pc} (ft)
1	10.0	9.0	17.0	73.2	35	3.5	0.33
2	8.0	7.0	21.2	75.8	44	3.2	0.285
3	6.5	5.5	26.1	78.9	52	3.0	0.17
4	5.0	4.2	32.7	83.2	62	2.75	0.17
5	4.25	3.5	37.7	86.2	74	2.5	0.13
							1.085

* SETTLED ** END COMPRESSION SETTLING

CREEP @ YEAR 5 = 0.009 log (5/1.5) 3.0 = 0.014'

@ YEAR 20 = 0.009 log (20/1.5) 3.0 = 0.030

POST CONST. SETTLEMENT = (1 - 0.29) 1.085 + 0.030

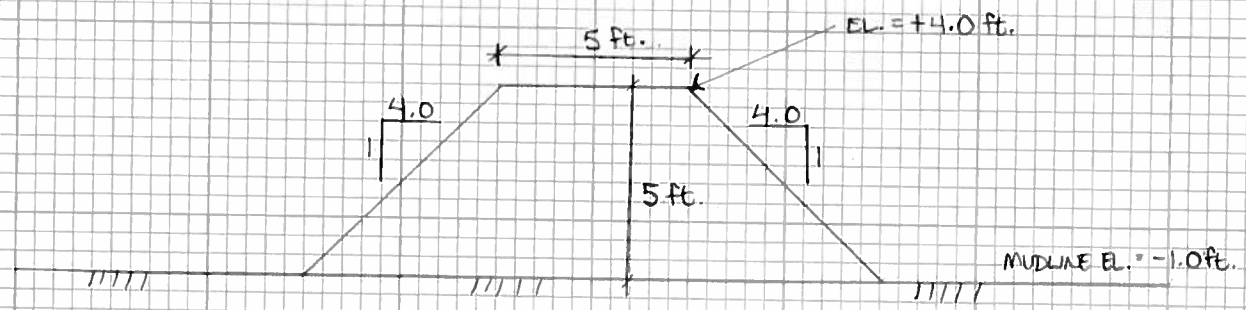
= 0.801

45126E (01/12)

CUT / FILL VOLUME CALCUATIONS

MARSH 1 (CONST. EL. = +4.0 ft.)

• COMPUTING THE DESIGN FILL VOLUME OF THE DIKE:



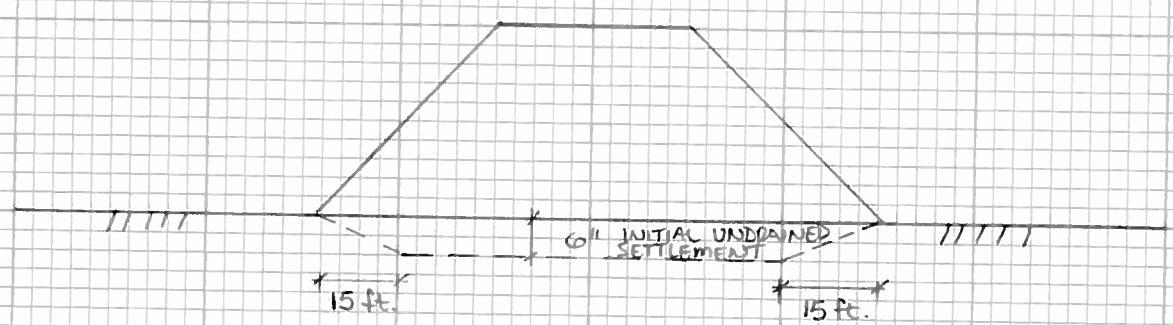
$A_N = h[b_1 + b_2]$
$= (5\text{ ft.})[20\text{ ft.} + 5\text{ ft.}]$
$= (5\text{ ft.})(25\text{ ft.})$
$A_N = 125\text{ ft.}^2$

$V_N = A_N L$
$= (125\text{ ft.}^2)(6{,}148\text{ ft.})$
$V_N = 768{,}500\text{ ft.}^3$

$\left(\frac{768{,}500\text{ ft.}^3}{1}\right)\left(\frac{1\text{ yd.}^3}{27\text{ ft.}^3}\right)$

$V_N = 28{,}463\text{ yd.}^3$

• COMPUTING THE REQUIRED CUT VOLUME FOR THE DIKE:



$A_R = h[b_1 + b_2]$
$= (0.5\text{ ft.})[15\text{ ft.} + 15\text{ ft.}]$
$= (0.5\text{ ft.})(30\text{ ft.})$
$A_R = 15\text{ ft.}^2$

$V_R = A_R L$
$= (15\text{ ft.}^2)(6{,}148\text{ ft.})$
$V_R = 92{,}220\text{ ft.}^3$

$\left(\frac{92{,}220\text{ ft.}^3}{1}\right)\left(\frac{1\text{ yd.}^3}{27\text{ ft.}^3}\right)$

$V_R = 3{,}416\text{ yd.}^3$

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• COMBINING THE LOSS DUE TO SETTLEMENT WITH THE DESIGN FILL:

$$V_T = V_N + V_R + 0.10[V_N + V_R]$$
$$= 31{,}879\ yd.^3 + 0.10[31{,}879\ yd.^3]$$
$$= 31{,}879\ yd.^3 + 3{,}187.9\ yd.^3$$

$$\boxed{V_T = 35{,}067\ yd.^3}$$

• COMPUTING THE ASSOCIATED CUT/FILL RATIO & OVERBUILD:

$$\frac{CUT}{FILL} = \frac{35{,}067\ yd.^3}{28{,}463\ yd.^3} = \boxed{1.23}$$

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Work Product

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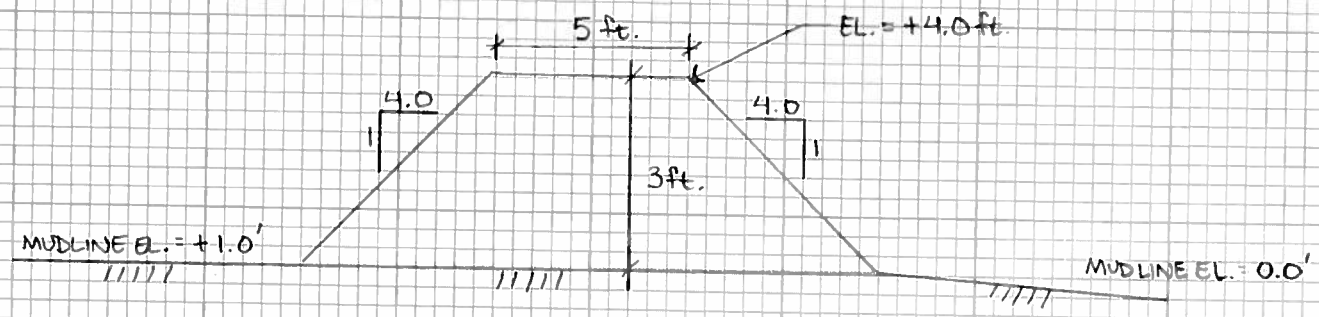
MARSH 2 (CONST. EL. = +4.0 ft.)

- COMPUTING THE DESIGN FILL VOLUME OF THE DIKE:



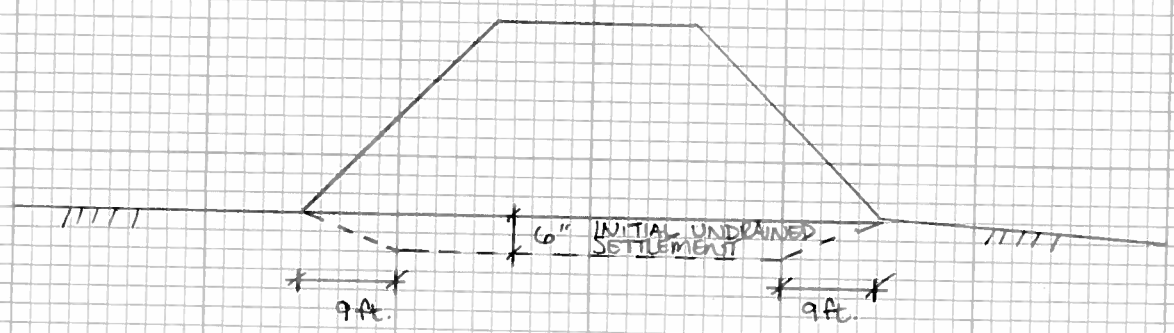
$A_N = h[b_1 + b_2]$
$= (3\text{ ft.})[12\text{ ft.} + 5\text{ ft.}]$
$= (3\text{ ft.})(17\text{ ft.})$
$A_N = 51\text{ ft.}^2$

$V_N = A_N L$
$= (51\text{ ft.}^2)(11{,}278\text{ ft.})$
$V_N = 575{,}178\text{ ft.}^3$

$\left(\frac{575{,}178\text{ ft.}^3}{1}\right)\left(\frac{1\text{ yd.}^3}{27\text{ ft.}^3}\right)$

$V_N = 21{,}303\text{ yd.}^3$

- COMPUTING THE REQUIRED CUT VOLUME FOR THE DIKE:



$A_R = h[b_1 + b_2]$
$= (0.5\text{ ft.})[9\text{ ft.} + 11\text{ ft.}]$
$= (0.5\text{ ft.})(20\text{ ft.})$
$A_R = 10\text{ ft.}^2$

$V_R = A_R L$
$= (10\text{ ft.}^2)(11{,}278\text{ ft.})$
$V_R = 112{,}780\text{ ft.}^3$

$\left(\frac{112{,}780\text{ ft.}^3}{1}\right)\left(\frac{1\text{ yd.}^3}{27\text{ ft.}^3}\right)$

$V_R = 4{,}177\text{ yd.}^3$

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- COMBINING THE LOSS DUE TO SETTLEMENT WITH THE DESIGN FILL:

$$V_T = V_N + V_R + 0.10\,[V_N + V_R]$$

$$= 25{,}480\ yd.^3 + 0.10\,[25{,}480\ yd.^3]$$

$$= 25{,}480\ yd.^3 + 2{,}548\ yd.^3$$

$$\boxed{V_T = 28{,}028\ yd.^3}$$

- COMPUTING THE ASSOCIATED CUT / FILL RATIO & OVERBUILD:

$$\frac{CUT}{FILL} = \frac{28{,}028\ yd.^3}{21{,}303\ yd.^3} = \boxed{1.32}$$

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ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705

PROJECT COLE'S BAYOU MARSH RESTORATION

SHEET NO. 5 OF 6

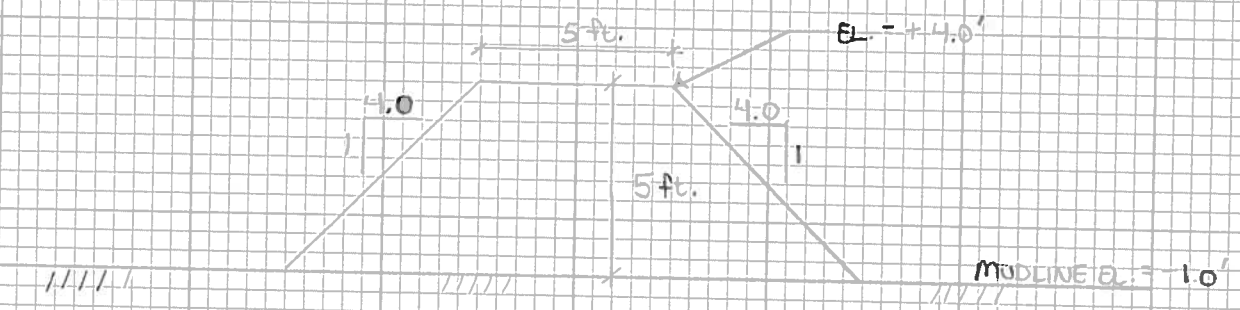
CALCULATED BY NMA DATE 3/28/14

CHECKED BY DATE

SCALE

MARSH 3 (CONST. EL. = +4.0 ft.)

• COMPUTING THE DESIGN FILL VOLUME OF THE DIKE:



$A_N = h[b_1 + b_2]$

$= (5\ ft.)[20\ ft. + 5\ ft.]$

$= (5\ ft.)[25\ ft.]$

$A_N = 125\ ft.^2$

$V_N = A_N L$

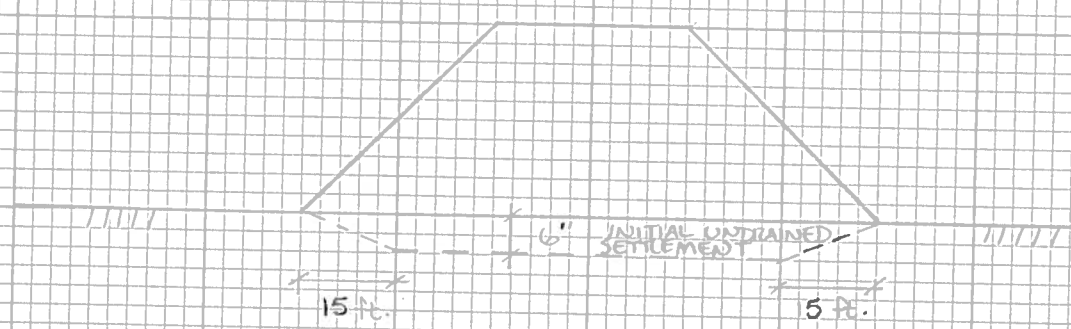
$= (125\ ft.^2)(25{,}432\ ft.)$

$V_N = 3{,}179{,}000\ ft.^3$

$\left(\frac{3{,}179{,}000\ ft.^3}{1}\right)\left(\frac{1\ yd.^3}{27\ ft.^3}\right)$

$V_N = 117{,}741\ yd.^3$

• COMPUTING THE REQUIRED CUT VOLUME FOR THE DIKE:



$A_R = h[b_1 + b_2]$

$= (0.5\ ft.)[15\ ft. + 15\ ft.]$

$= (0.5\ ft.)(30\ ft.)$

$A_R = 15\ ft.^2$

$V_R = A_R L$

$= (15\ ft.^2)(25{,}432\ ft.)$

$V_R = 381{,}480\ ft.^3$

$\left(\frac{381{,}480\ ft.^3}{1}\right)\left(\frac{1\ yd.^3}{27\ ft.^3}\right)$

$V_R = 14{,}129\ yd.^3$

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SHEET NO. 6 OF 6

FILE NO. 13-80-3705

CALCULATED BY NMA DATE 3/28/14

PROJECT COLE'S BAYOU MARSH RESTORATION

CHECKED BY DATE

SCALE

- COMBINING THE LOSS DUE TO SETTLEMENT WITH THE DESIGN FILL:

$$V_T = V_N + V_R + 0.10\,[V_N + V_R]$$

$$= 131{,}870\ yd.^3 + 0.10\,[131{,}870\ yd.^3]$$

$$= 131{,}870\ yd.^3 + 13{,}187\ yd.^3$$

$$\boxed{V_T = 145{,}057\ yd.^3}$$

- COMPUTING THE ASSOCIATED CUT/FILL RATIO & OVERBUILD:

$$\frac{CUT}{FILL} = \frac{145{,}057\ yd.^3}{117{,}741\ yd.^3} = \boxed{1.23}$$

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Work Product

ARDAMAN & ASSOCIATES, INC.

SHEET NO. _____ OF _____

FILE NO. 13-80-3705

CALCULATED BY _____ DATE _____

PROJECT COLE'S BAYOU MARSH RESTORATION

CHECKED BY _____ DATE _____

SCALE _____

MARSH 1

CUT VOLUME = 35,067 yd.^{3}
NEAT FILL VOLUME = 28,463 yd.^{3}
CUT / FILL = 1.23

CREST EL. = +4.0'

CUT VOLUME = 24,170 yd.^{3}
NEAT FILL VOLUME = 19,127 yd.^{3}
CUT/FILL = 1.26

CREST EL. = +3.0'

H = 4.0 ft.

$A_N = h[b_1 + b_2]$
$= (4\text{ ft.})(16\text{ ft.} + 5\text{ ft.})$
$= (4\text{ ft.})(21\text{ ft.})$
$= 84\text{ ft.}^2$

$V_N = A_N L$
$= (84\text{ ft.}^2)(6{,}148\text{ ft.})$
$= \frac{516{,}432\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$\boxed{V_N = 19{,}127\text{ yd.}^3}$

$A_R = h[b_1 + b_2]$
$= (0.5\text{ ft.})[2\text{ ft.} + 13\text{ ft.}]$
$= (0.5\text{ ft.})(15\text{ ft.})$
$= 12.5\text{ ft.}$

$V_R = (12.5\text{ ft.}^2)(6{,}148\text{ ft.})$
$= \frac{76{,}850\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_R = 2{,}846\text{ yd.}^3$

37 TOTAL
−24
13

$V_T = 19{,}127\text{ yd.}^3 + 2{,}846\text{ yd.}^3 + 0.16[19{,}127\text{ yd.}^3 + 2846\text{ yd.}^3]$
$= 21{,}973\text{ yd.}^3 + 2{,}197.3\text{ yd.}^3$

$\boxed{V_T = 24{,}170\text{ yd.}^3}$

$\frac{\text{CUT}}{\text{FILL}} = \frac{24{,}170\text{ yd.}^3}{19{,}127\text{ yd.}^3} = 1.26$

Confidential Information
Privileged & Confidential
Work Product

5126E (01/12)

ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705

PROJECT COLE'S BAYOU MARS RESTORATION

SHEET NO OF

CALCULATED BY DATE

CHECKED BY DATE

SCALE

MARSH 2

CUT VOLUME = 28,028 yd.^{3}
NEAT FILL VOLUME 21,303 yd.^{3}
CUT / FILL = 1.32

CONST. EL. = +4.0'

CUT VOLUME = 15,392 yd.^{3}
NEAT FILL VOLUME = 10,860 yd.^{3}
CUT / FILL = 1.42

CONST. EL. = +3.0'

H = 2.0 ft.

$A_N = h[b_1 + b_2]$
$= (2\text{ ft.})(8\text{ ft.} + 5\text{ ft.})$
$= (2\text{ ft.})(13\text{ ft.})$
$= 26\text{ ft.}^2$

$V_N = (26\text{ ft.}^2)(11{,}278\text{ ft.})$
$= \dfrac{293{,}228\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$\boxed{V_N = 10{,}860\text{ yd.}^3}$

$A_R = h[b_1 + b_2]$
$= (0.5\text{ ft.})(6\text{ ft.} + 9\text{ ft.})$
$= (0.5\text{ ft.})(15\text{ ft.})$
$= 7.5\text{ ft.}^2$

$V_R = (7.5\text{ ft.}^2)(11{,}278\text{ ft.})$
$= \dfrac{84{,}585\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_R = 3{,}133\text{ yd.}^3$

21 TOTAL
− 12
9

$V_T = 10{,}860\text{ yd.}^3 + 3{,}133\text{ yd.}^3 + 0.10[10{,}860\text{ yd.}^3 + 3{,}133\text{ yd.}^3]$
$= 13{,}993\text{ yd.}^3 + 1{,}399.3\text{ yd.}^3$

$\boxed{V_T = 15{,}392\text{ yd.}^3}$

$\dfrac{\text{CUT}}{\text{FILL}} = \dfrac{15{,}392\text{ yd.}^3}{10{,}860\text{ yd.}^3} = 1.42$

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ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705

PROJECT COLE'S BAYOU MARSH RES ORA ON

SHEET NO OF

CALCULATED BY DATE

CHECKED BY DATE

SCALE

MARSH 3

CUT VOLUME = 145,057 yd.^{3}
NEAT FILL VOLUME = 117,741 yd.^{3}
CUT / FILL = 1.23

CONST. EL. = +4.0'

CUT VOLUME = 99,986 yd.^{3}
NEAT FILL VOLUME = 79,122 yd.^{3}
CUT / FILL = 1.26

CONST. EL. = +3.0'

H = 4.0 ft.

$A_N = h[b_1 + b_2]$
$= (4\text{ ft.})[16\text{ ft.} + 5\text{ ft.}]$
$= (4\text{ ft.})(21\text{ ft.})$
$= 84\text{ ft.}^2$

$V_N = (84\text{ ft.}^2)(25{,}432\text{ ft.})$
$= \dfrac{2{,}136{,}288\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_N = 79{,}122\text{ yd.}^3$

$A_R = h[b_1 + b_2]$
$= (0.5\text{ ft.})[12\text{ ft.} + 13\text{ ft.}]$
$= (0.5\text{ ft.})(25\text{ ft.})$
$= 12.5\text{ ft.}^2$

$V_R = (12.5\text{ ft.}^2)(25{,}432\text{ ft.})$
$= \dfrac{317{,}900\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_R = 11{,}774\text{ yd.}^3$

$V_T = 79{,}122\text{ yd.}^3 + 11{,}774\text{ yd.}^3 + 0.10[79{,}122\text{ yd.}^3 + 11{,}774\text{ yd.}^3]$
$= 90{,}896\text{ yd.}^3 + [9{,}089.6\text{ yd.}^3]$

$V_T = 99{,}986\text{ yd.}^3$

$\dfrac{\text{CUT}}{\text{FILL}} = \dfrac{99{,}986\text{ yd.}^3}{79{,}122\text{ yd.}^3} = 1.26$

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ARDAMAN & ASSOCIATES, INC.

FILE NO. 13-80-3705

PROJECT COLE'S BAYOU MARSH RESTORATION

SHEET NO. _____ OF _____

CALCULATED BY NMA DATE _____

CHECKED BY _____ DATE _____

SCALE _____

MARSH 3

CUT VOLUME = 121,485 yd.^{3}
NEAT FILL VOLUME: 97,489 yd.^{3}
CUT / FILL = 1.25

CONST. EL. = +3.5'

H = 4.5 ft. $A_N = h[b_1 + b_2]$

$= (4.5\text{ ft.})[18\text{ ft.} + 5\text{ ft.}]$

$= (4.5\text{ ft.})[23\text{ ft.}]$

$A_N = 103.5\text{ ft.}^2$

$V_N = (103.5\text{ ft.}^2)(25{,}432\text{ ft.})$

$= \dfrac{2{,}632{,}212\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_N = 97{,}489\text{ yd.}^3$

$A_R = h[b_1 + b_2]$

$= (0.5\text{ ft.})[13.5\text{ ft.} + 14\text{ ft.}]$

$= (0.5\text{ ft.})(27.5\text{ ft.})$

$= 13.75\text{ ft.}^2$

41 TOTAL
−27
14

$V_R = (13.75\text{ ft.}^2)(25{,}432\text{ ft.})$

$= \dfrac{349{,}690\text{ ft.}^3}{27\ \frac{\text{ft.}^3}{\text{yd.}^3}}$

$V_R = 12{,}952\text{ yd.}^3$

$V_T = 97{,}489\text{ yd.}^3 + 12{,}952\text{ yd.}^3 + 0.10[97{,}489\text{ yd.}^3 + 12{,}952\text{ yd.}^3]$

$= 110{,}441\text{ yd.}^3 + 11{,}044.1\text{ yd.}^3$

$V_T = 121{,}485\text{ yd.}^3$

$\dfrac{\text{CUT}}{\text{FILL}} = \dfrac{121{,}485\text{ yd.}^3}{97{,}489\text{ yd.}^3} = 1.25$

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APPENDIX C. FINAL DATA REPORT

This Appendix contains the following:

- Final Data Report (excluding Appendices C, D, E and G)

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

VERMILLION PARISH, LOUISIANA

Ardaman & Associates, Inc.

OFFICES

Orlando - 8008 S. Orange Avenue, Orlando, Florida 32809 - Phone (407) 855-3860
Alexandria - 3609 Mac Lee Drive, Alexandria, Louisiana 71302 - Phone (318) 443-2888
Bartow - 1525 Centennial Drive, Bartow, Florida 33830 - Phone (863) 533-0858
Baton Rouge - 316 Highlandia Drive, Baton Rouge, Louisiana 70884 - Phone (225) 752-4790
Cocoa - 1300 N. Cocoa Blvd., Cocoa, Florida 32922 - Phone (321) 632-2503
Fort Myers - 9970 Bavaria Road, Fort Myers, Florida 33913 - Phone (239) 768-6600
Miami - 2608 W. 84th Street, Hialeah, Florida 33016 - Phone (305) 825-2683
Monroe - 1122 Hayes Street, West Monroe, Louisiana 71292 - Phone (318) 387-4103
New Orleans - 1305 Distributors Row, Suite I, Jefferson, Louisiana 70123 - Phone (504) 835-2593
Port St. Lucie - 460 Concourse Place NW, Unit 1, Port St. Lucie, Florida 34986 - Phone (772) 878-0072
Sarasota - 78 Sarasota Center Blvd., Sarasota, Florida 34240 - Phone (941) 922-3526
Shreveport - 7222 Greenwood Road, Shreveport, Louisiana 71119 - Phone (318) 636-3673
Tallahassee - 3175 West Tharpe Street, Tallahassee, Florida 32303 - Phone (850) 576-6131
Tampa - 3925 Coconut Palm Drive, Suite 115, Tampa, Florida 33619 - Phone (813) 620-3389
West Palm Beach - 2200 North Florida Mango Road, Suite 101, West Palm Beach, Florida 33409 - Phone (561) 687-8200

MEMBERS:
A.S.F.E.
American Concrete Institute
ASTM International
Florida Institute of Consulting Engineers

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Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

March 28, 2014
AAI File: 13-80-3705

Coastal Protection and Restoration Authority
P.O. Box 44027
Baton Rouge, LA 70804

Attention: Ms. Amanda Taylor, P.E.
Amanda.Taylor@la.gov

Re: Final 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 Final 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.

STATE OF LOUISIANA
ROBERT J. WERNER
License No. 36633
PROFESSIONAL ENGINEER IN CIVIL ENGINEERING
3/28/14

MEGAN G. BOURGEOIS, P. E.
PROJECT ENGINEER

ROBERT J. WERNER, P.E.
BRANCH MANAGER/PRINCIPAL ENGINEER

MGB/RJW/rjw

(6) bound originals

316 Highlandia Drive 70810, Post Office Box 83710, Baton Rouge, Louisiana 70884 Phone (225) 752-4790 Fax (225) 752-4878
Louisiana: Alexandria, Baton Rouge, Monroe, New Orleans, Shreveport
Florida: Bartow, Cocoa, Fort Myers, Miami, Orlando, Port Charlotte, Port St. Lucie, Sarasota, Tallahassee, Tampa, West Palm Beach

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

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	191
Consolidation Test	ASTM-D2435	23
Atterberg Limit Determination	ASTM-D4318	199
Organic Content	ASTM-D2974	39
Moisture Content	ASTM-D2216	573
Grain Size Analysis	ASTM-(C136,D1140,D422)	62
Unit Weight Determination	ASTM-D2937	573
Specific Gravity Determination	ASTM-D854	23

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 2.78, with an average value of 2.70. 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 39 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 199 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 G.

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.

3.3.4 Settling Column Test

One representative composite sample was generated using samples obtained from 4 soil borings performed within the proposed borrow area. After mixing, the composite sample was split into three replicates (one for testing, one for archive storage, and one for CPRA use if requested).

A laboratory settling test was performed in a 10.8-cm diameter graduated plexi-glass settling column. The test sample was mixed with synthesized marsh water having a salinity, in terms of total dissolved solids, of 10.5 g/l. Initial solids content of the settling test sample was 13.4 percent, with a corresponding initial total suspended solids concentration of 146 g/l. The slurry was mixed with a hand-held stirrer to provide a homogeneous sample, and remove any segregation of particles which occurred during placement of the slurry into the column. The columns were covered with clear plastic wrap to prevent evaporation of the supernatant fluid during the test period.

The settling test consisted of visually monitoring the height of the sediment-water interface with time over a period of 14 days. The test results, in terms of interface height versus time are included in Appendix F. The initial settling velocity (i.e., the initial slope of the settled height versus time curve) was 0.032 cm/minute. This more rapid "zone settling" behavior, prevailed during the first 24 hours of the test. Significant slower "compression settling" then commenced and continued until the end of the 14-day test.

The final settled sediment height was 10 cm, with corresponding final solids content of 31 percent (total suspended solids concentration of 394 g/l).

3.3.5 Slurry Consolidation Tests

A one-dimensional incremental slurry consolidation test was performed on a sample prepared from the dredge borrow area composite. The test was performed using specially designed and fabricated equipment with a counterbalance pulley system (to counteract a portion of the normal load associated with the top loading piston) that is similar to a conventional one-dimensional consolidation device except that the equipment used allows for evaluation of the consolidation behavior at stresses as low as 0.001 kg/cm^2. For this test, the specimen is created and then incrementally loaded within a 10.2-cm diameter by 60-cm tall settling column to produce a specimen height at the end of settling of approximately 9 cm. After gravity settling is substantially complete, the specimen is loaded one-dimensionally under stresses of 0.001 to 2.0 kg/cm^2 using a load increment ratio of 1.

The change in specimen height with time under each load was monitored and evaluated to characterize the one-dimensional compressibility, consolidation and drained creep properties of the sediment. Results of the slurry consolidation test are included in Appendix F.

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 191 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

One consolidated-undrained (CK_oU) direct simple shear (CKoUDSS) strength test was performed on a selected sample. The test was performed at a vertical effective stress greater than the *in-situ* preconsolidation pressure in order to determine normally consolidated undrained strength ratio, s_u/ σ'_{vc}. This test was used, along with the UU, TV, and LV strength data discussed above, to select final undrained shear strength profiles for use in design stability analyses, and to develop shear modulus values for use in undrained settlement calculations.

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

Results of the CKoUDSS tests indicated SHANSEP strength parameters C=0.26. This value is higher than normally expected for inorganic highly plastic clays (where C=0.24 more typical). An undrained shear strength profile computed according to the SHANSEP methodology and using somewhat more conservative parameters C=0.24 and m=0.70 is plotted alongside the current UU and TV strength test data on Figure B-17. As can be seen, the computed strength profile based on normalized clay behavior is consistent with the lower bound of the measured data. Considering the observed variability in measured shear strength data above the lower bound strength envelope, a somewhat higher design shear strength profile was developed for use in stability analysis.

SECTION 4. REFERENCES

Becker, D. E., Crooks, J. H. A., Been, K., and Jefferies, M. B. (1987), "Work as a Criterion for Determining In Situ and Yield Stresses in Clays", Canadian Geotechnical Journal, Vol. 24, No. 4, pp 549-564.

Casagrande, A. (1936), "The Determination of the Preconsolidation Load and its Practical Significance", Proceedings, First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Vol. 3, pp 60-64.

Germaine, J., and Germaine, A. (2009). "Geotechnical Laboratory Measurements for Engineers." Hoboken, New Jersey: John Wiley and Sons, Inc.

Ladd, C. C. and Foott, R. (1974), "New Design Procedure for Stability of Soft Clays," ASCE, JGGE, Vol. 100, No. G57, pp 763-786.

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APPENDIX A. BORING LOGS AND PROFILES

This Appendix contains the following:

- Figure A-1 – Boring Location Plan
- Description of Terms & Symbols Used on Soil Boring Logs
- Soil Boring Logs
- Soil Boring Profiles

VICINITY MAP
NOT TO SCALE

REFERENCE:
SITE IMAGE BY GOOGLE.

SCALE
1" = 2500'

Confidential Information:
Privileged & Confidential
Work Product

REFERENCE:
PLAN BY GOOGLE.

NOTE:
BORING LOCATIONS ARE APPROXIMATE.

COLE'S BAYOU MARSH CREATION AND HYDROLOGIC RESTORATION
VERMILION PARISH, LOUISIANA

for

COASTAL PROTECTION & RESTORATION AUTHORITY
BATON ROUGE, LOUISIANA

AAI
Ardaman & Associates, Inc.
Baton Rouge, LA Jefferson, LA

Project Engineer:	Drawn by:	Checked by:
R. ROUSSET	RJB	RER
File No.: 13-80-3705	Date: 12-17-13	Figure No.: A-1

Title:
BORING PLAN

Mar 24, 2014 - 3:33pm

P:\2013\13-80-3705 Cole's Bayou\AutoCAD\13-80-3705 Boring Plan.dwg

DESCRIPTION OF TERMS AND SYMBOLS USED ON SOIL BORING LOG

FIELD DATA				LABORATORY DATA								
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Other	Soil Type	DESCRIPTION

5

10

15

20

25

30

35

40

Ground Water Levels

Long-Term Depth
Depth to water after boring is completed (time noted).

Short-Term Depth
Depth to water after initial water encountered prior to proceeding with boring (time noted).

Initially Encountered
Depth where free water was initially encountered during augering.

Sampling/Field Data

3.5 (P) — **Undisturbed**
3" dia. Tube sample

Pocket Penetrometer (P)
Penetration resistance (tons/sq. ft.).

Torvane (T)
Shearing resistance (tons/sq. ft.)

13 b/f (3-7-6) — **Split Spoon**
Std. penetration test

Std. Penetration
No. of blows per foot (blows per each six inch increments).

Auger
Disturbed (auger) collected in accordance with ASTM D-1452.

No Recovery
Sampling attempted but no sample retrieved.

DESCRIPTION

Description
Classifications are based on visual observations by field & lab representatives as well as results of laboratory data (when available).

Laboratory Data

Compressive Strength
Value based on peak compressive strength. Determined by unconfined compression test unless otherwise noted.

Dry Unit Weight
As determined by method similar to ASTM D-2937.

Water Content
As determined by pertinent portions of ASTM D-2216.

Atterberg Limits
LL : Liquid Limit
PL : Plastic Limit
PI : Plasticity Index
(= Liquid Limit - Plastic Limit)

Other
Results of other tests such as consolidation, permeability, grain size or notes associated with testing program.

Soil Type
Graphical representation of soil type. In accordance with USCS Symbols.

Ground Water Level Data	Boring Advancement Method	Notes
	Boring Abandonment Method	

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Form LOGTERMS

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-01

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/14/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 20.4"
Long. 92° 13' 37.2"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.12 t=1.0	77 73	56 38	87	22	65	98	ORG_1		Very soft gray CLAY (CH) w/ organics
			0.0 (P)	0.09 t=2.0	596 333	10 15	465	134	331		ORG_2		Very soft dark brown ORGANIC CLAY (OH)
	5		0.0 (P)	0.24 t=2.0	63 72	63 50							Very soft gray CLAY (CH) w/ trace organics
			0.0 (P)	0.18 t=3.0	84 120	53 37	90	23	67	98	ORG_3		
			1.0 (P)	0.25 t=4.0	37 34	87 65	47	17	30		ORG_4		Soft gray SILTY CLAY (CL) w/ organics
	10		1.5 (P)	0.74 t=5.0	33 33	90 79							
			2.5 (P)	1.37 t=6.0	28 25	97 86	57	17	40				Stiff gray and tan CLAY (CH)
	15		2.5 (P)	1.94 t=6.0	24 26	102 96							
			2.0 (P)	1.17 t=7.0	28	97	54	17	37				
	20		2.5 (P)	1.31 t=8.0	26 23	97 98							
	25		2.5 (P)	1.74 t=10.0	24 28	101 100	47	13	34	97	GSH		Stiff gray and tan SILTY CLAY (CL)
	30		3.0 (P)	2.03 t=12.0	22 23	103 107	55	15	40				Very stiff gray and tan CLAY (CH)
	35												Boring completed at 30 ft.
	40												

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 30 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis
ORG_1= 66.9%
ORG_2=47.2%
ORG_3=14.5%
ORG_4=22.6%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

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Vermillion Parish, LA

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LOG OF SOIL BORING B-02

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Sheet 1 of 2

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USACE Validated

File: 13-80-3705
Date: 10/15/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 18.3"
Long. 92° 13' 31.4"
Surface Elevation: 0 (ft., NAVD)

FIELD DATA				LABORATORY DATA										
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.12 t=1.0	525	11	695	322	373		ORG,[1]		Brown ORGANIC CLAY (OH)
					597	9							
			0.0 (P)	0.14 t=2.0	163	32							
					678	9							
	5		0.0 (P)	0.10 t=2.0	76	57	86	25	61	94			Very soft gray CLAY (CH) w/ organics
					132	37							
			1.0 (P)	0.18 t=3.0	45	77							
			1.5 (P)	0.32 t=4.0	37	85	47	16	31				Soft gray SILTY CLAY (CL)
	10				35	87							
			2.0 (P)	1.02 t=5.0	30	95	61	17	44	99			Medium to stiff gray and tan CLAY (CH)
					27	92							
			2.0 (P)	1.33 t=5.0	27	99	56	18	38				
					26	90							
	15		3.5 (P)	1.85 t=6.0	27	96							
					22	84							
			3.0 (P)	0.80 t=6.0	27	98	56	17	39				
					24	100							
			2.0 (P)	2.50 t=8.0	21	108							
	20				29	95							
			3.0 (P)	0.57 t=10.0	32	92	55	17	38				
	25				26	93							
			3.0 (P)										Very stiff gray and tan SILTY CLAY (CL) w/ sand
	30				22	103							
			3.0 (P)	2.50 t=14.0	22	99	37	19	18	83	GSH		
	35				25	92							
			2.0 (P)							100	GSH		Stiff gray and tan CLAY (CH)
	40				33	91							

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 80 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG,=75.7%
*Boring Elevations are approximate
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

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Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-02

Ardaman & Associates, Inc.

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/15/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 18.3"
Long. 92° 13' 31.4"
Surface Elevation: 0 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
													Stiff gray and tan CLAY (CH)
	45		3.0 (P)	1.07 t=18.0	42 48	78 77	83	26	57				Stiff gray and tan CLAY (CH)
	50		0.5 (P)		38	81				65	GSH		Soft gray and tan SANDY CLAY (CL)
	55		2.5 (P)										Stiff gray and tan CLAY (CH)
	60		2.0 (P)	0.60 t=24.0	32 29	93 92	46	17	29				Medium to stiff brown and gray SILTY CLAY (CL)
	65		2.0 (P)										
	70		1.0 (P)										
	75		0.5 (P)	1.20 t=29.0	27 30	99 92	31	20	11	94	GSH		--w/ trace sand, 73 to 80 ft.
	80		0.5 (P)	1.04 t=32.0	27 26	99 100	31	18	13	89	GSH		

Boring completed at 80 ft

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 80 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

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Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-03

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Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/17/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 22.1"
Long. 92° 13' 23.1"

Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		547	9							Very soft dark brown ORGANIC CLAY (OH)
			0.0 (P)		528	11	381	94	287	100	ORG_1		
	5		0.0 (P)	0.10 t=2.0	97	47	104	24	80	96	ORG_2		Very soft gray CLAY (CH) w/ organics
					109	41							
			0.5 (P)	0.17 t=3.0	84	52							
					328	19							
			1.0 (P)	0.15 t=4.0	55	69	71	18	53				
	10				40	74							
			1.5 (P)	0.75 t=5.0	31	93							Medium to stiff gray and tan CLAY (CH)
					31	87							
			2.0 (P)	1.39 t=6.0	26	101	55	13	42				
					26	96							
	15		1.5 (P)	1.80 t=6.0	25	102							
					28	87							
			2.0 (P)	1.57 t=7.0	26	99							
					26	94							
			2.5 (P)	1.32 t=8.0	26	99	53	13	40				
	20				26	99							
			1.5 (P)										
	25				29	94							
			2.5 (P)	1.79 t=12.0	27	96	67	15	52				
	30				27	94							Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 30 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG_1=64.4%
ORG_2=36.6%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705 GPJ LOG01R GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-04

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/21/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 5.4"
Long. 92° 13' 54.3"

Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.11	808	7	645	199	446		ORG$_1$		Very soft brown ORGANIC CLAY (OH)
			0.0 (P)		553		375	120	255	99			
					237	18				99			
	5		0.0 (P)	0.12 t=2.0	75	56							
					75	51							Soft to medium gray and tan CLAY (CH) w/ organics
			0.5 (P)	0.47 t=2.0	32	92	50	15	35				
					32	82							
			1.5 (P)	1.32 t=4.0	27	97							
	10				36	79							
			2.5 (P)	0.75 t=4.0	31	93	61	13	48				
					31	86							
			3.0 (P)	2.12 t=6.0	24	103							Stiff to very stiff gray and tan CLAY (CH)
	15		3.0 (P)	1.03 t=6.0	28	97	58	17	41				
					31	85							
			2.5 (P)	1.99 t=7.0	25	101							
			2.5 (P)	1.12 t=7.0	28	96	66	20	46				
	20				30	97							
			2.0 (P)	2.04 t=9.0	26	97				79	GSH		
	25			1.64 t=10.0	27	97							--w/ sand, 23 to 25 ft.
					26	96							
			2.5 (P)	1.54 t=11.0	28	96	65	15	50				
	30				31	88							
													Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 30 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG$_1$=74.3%
GSH: Hydrometer Particle Size Analysis
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-05

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/10/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 59.5"
Long. 92° 13' 44.1"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)				173	35	138				Very soft gray ORGANIC CLAY (OH)
					319	18							
			0.0 (P)	0.14 t=2.0	91	50							
					95	39							Very soft dark gray CLAY (CH) w/ organics
	5		0.0 (P)	0.12 t=2.0	81	53	95	25	70				
					90	48							
			1.0 (P)	0.14 t=3.0	247	21	266	99	167		ORG_1		Very soft dark gray ORGANIC CLAY (OH) w/ roots
					134	34							
			0.5 (P)	0.14 t=4.0	170	29	175	58	117	99	ORG_2		
	10		1.5 (P)	0.70 t=5.0	32	90							
			1.5 (P)	1.03 t=6.0	26	99							
					30	89							
	15		2.0 (P)	0.93 t=6.0	29	95	58	15	43				Medium to stiff gray CLAY (CH)
					26	95							
			2.5 (P)	1.12 t=7.0	26	100	53	14	39				
					25	98							
			4.0 (P)	0.54 t=8.0	28	94							
	20				32	88							
			3.0 (P)				46	16	30	79	GSH		Medium gray and tan SILTY CLAY (CL) w/ sand
	25				25	101							
			1.0 (P)	0.41 t=12.0	42	79	67	19	48				Soft gray and tan CLAY (CH)
	30				41	78							
			1.5 (P)	1.69 t=14.0	26	99	46	15	31	95	GSH		Stiff gray and tan SILTY CLAY (CL) w/ trace sand
	35				28	95							
			2.0 (P)										
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis
ORG_1=30.2%
ORG_2=45.2%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-05

Ardaman & Associates, Inc.

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/10/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 59.5"
Long. 92° 13' 44.1"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					33	91							Stiff gray and tan SILTY CLAY (CL) w/ trace sand
			2.5 (P)		44								Soft gray CLAY (CH)
	45												
			2.0 (P)				59	16	43	99	GSH		
	50				44	78							
			2.0 (P)	0.43 t=22.0	62	64	88	21	67				
	55				58	66							
			1.5 (P)										
	60												Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-06

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/8/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 58"
Long. 92° 13' 34.9"

Surface Elevation: 0 (ft., NAVD)

FIELD DATA				LABORATORY DATA										
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.06 t=1.0	153	33	135	29	106		ORG_1		Very soft gray ORGANIC CLAY (OH)
					193								
			0.0 (P)	0.04 t=2.0	163	32	126	27	99	97	GSH		
					142	35							
	5		0.0 (P)	0.24 t=2.0	63	62							
					115	41							
			0.0 (P)	0.25 t=3.0	70	60	83	19	64	99	GSH		Soft gray CLAY (CH) w/ organics
					72	53							
			0.0 (P)	0.22 t=4.0	212	24							
	10				169	30							Very soft gray ORGANIC CLAY (OH)
			1.0 (P)	0.40 t=5.0	34	89	44	15	29				Soft gray SILTY CLAY (CL) w/ organics
			1.5 (P)	0.88 t=6.0	31	91							
					36	83							
	15		2.0 (P)	0.73 t=6.0	33	91	61	15	46				Medium to stiff gray and tan CLAY (CH)
					28	93							
			2.0 (P)	1.81 t=7.0	24	103							
					25	97							
			1.5 (P)	1.08 t=8.0	25	100	51	12	39				
	20				27	91							
			1.0 (P)	2.60 t=10.0	26	97	58	19	39	97	GSH		Very stiff gray and tan CLAY (CH)
	25				32	91							
			3.5 (P)	0.82 t=12.0	43	78	87	25	62	100	GSH		Soft to medium gray and tan CLAY (CH)
	30				46	78							
			2.0 (P)										
	35				31	81							
			3.5 (P)	0.44 t=16.0	43	80	62	18	44				
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis
ORG_1=11.4%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-06

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/8/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 58"
Long. 92° 13' 34.9"
Surface Elevation: 0 (ft., NAVD)

FIELD DATA				LABORATORY DATA										
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					31	84							Soft to medium gray and tan CLAY (CH)
	45		2.0 (P)	0.85 t=18.0	57 36	66 79	76	20	56				
	50		0.0 (P)	2.10 t=20.0 1.28 t=20.0	31 34 65	92 88 70	32 48	21 17	11 31	97	GSH		Stiff to very stiff gray SILTY CLAY (CL)
	55		0.0 (P)		31	89							Medium brown and gray SILTY CLAY (CL)
	60		0.0 (P)	0.66 t=24.0	31	95	34	15	19	99	GSH		
	65												Boring completed at 60 ft.
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 60 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi GSH: Hydrometer Particle Size Analysis
Confidential Information: Privileged & Confidential Work Product	Boring Abandonment Method: Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-07

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/11/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 57.1"
Long. 92° 13' 35.5"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.22 t=1.0	88	50							Very soft to soft gray CLAY (CH) w/ organics
					94	43							
			0.0 (P)	0.10 t=2.0	77	56	76	20	56				
					100	47							
	5		0.0 (P)	0.16 t=2.0	88	50							
					87	46							
			0.0 (P)	0.19 t=3.0	141	35							
			1.0 (P)	0.21 t=4.0	68	61	81	19	62		ORG_1		
	10				43	65							
			1.0 (P)	0.46 t=5.0	36	87							
					35	77							
			1.5 (P)	1.90 t=6.0	25	101							
					31	88							
	15		2.0 (P)	0.86 t=6.0	26	99	63	15	48				
					33	83							
			2.0 (P)	2.08 t=7.0	22	106							Stiff gray and tan CLAY (CH)
					23	109							
			3.0 (P)	1.30 t=8.0	24	103	55	15	40				
	20				25	106							
			3.5 (P)	1.32 t=10.0	34	88							
	25				34	88							
			2.5 (P)	1.39 t=12.0	27	97	52	16	36				
	30				33	89							
			3.5 (P)										
	35				31	88							
													Medium gray and tan SANDY CLAY (CL)
			2.5 (P)	0.80 t=16.0	29	96	46	15	31	66			
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG_1=28.6%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-07

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/11/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 57.1"
Long. 92° 13' 35.5"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					28	94							Medium gray and tan SANDY CLAY (CL)
	45		1.0 (P)		59	65							Medium gray and tan SILTY CLAY (CL)
	50		1.5 (P)	0.83 t=20.0	31 29	94 86	32	23	9	95	GSH		
	55		1.5 (P)		32	95							Medium gray and tan CLAY (CH) w/ organics
	60		1.5 (P)	0.86 t=24.0	32 28	93 88	59	15	44	99	ORG_2 GSH		
													Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 60 ft.	ORG_2=31.7%
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-08

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/22/13
Logged by: R. Brunet
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 54.7"
Long. 92° 14' 0"

Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA										
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		177	22	347	99	248	97	ORG_1		Very soft dark brown ORGANIC CLAY (OH)
			0.0 (P)	0.20 t=2.0	61	64	65	16	49				Very soft gray CLAY (CH)
					54	62							
	5		0.0 (P)	0.19 t=2.0	96	47							
					80	57							
			0.0 (P)	0.14 t=3.0	98	48	104	24	80	98	ORG_2		
					117	40							
			1.0 (P)	0.37 t=4.0	33	90							Medium to stiff gray and tan CLAY (CH)
					34	78							
	10		0.5 (P)	0.77 t=5.0	29	96	52	14	38				
					27	93							
			2.0 (P)	1.20 t=6.0	26	100							
					28	97							
	15		2.0 (P)	1.76 t=6.0	25	102							
					26	93							
			1.5 (P)	1.57 t=7.0	25	101							
					26	90							
			2.0 (P)	1.31 t=8.0	26	99	52	15	37	97			
	20				27	97							
			0.75 (P)										
	25				27	95							
			3.0 (P)	0.87 t=12.0	43	79							
	30				39	80							
			1.25 (P)										
	35				23	96							Stiff to very stiff gray and tan CLAY (CH)
			4.25 (P)	2.01 t=16.0	28	94	51	16	35				
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 80 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG_1=73.4%
ORG_2=6.7%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-08

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/22/13
Logged by: R. Brunet
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 54.7"
Long. 92° 14' 0"

Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					34	90							Stiff to very stiff gray and tan CLAY (CH)
	45		1.5 (P)	1.54 t=18.0	34	87	61	17	44				
	50		1.0 (P)										
	55		0.75 (P)		39	74							
													Stiff brown and gray SILTY CLAY (CL)
	60		1.0 (P)	1.30 t=24.0	28 30	96 91	31	17	14	97	GSH		
	65		0.0 (P)										
	70		1.0 (P)	1.35 t=28.0	30 30	92 94	45	16	29				
	75		1.0 (P)		29	91							
	80		0.25 (P)										

Boring completed at 80 ft.

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 80 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-09

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/13/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 55.1"
Long. 92° 12' 39.4"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA										
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.19 t=2.0	127 118	38 42	120	34	86		ORG$_1$		Very soft brown ORGANIC CLAY (OH)
	5		0.0 (P) 0.0 (P) 0.0 (P)	0.15 t=2.0	64 86	62 35	73	18	55				Very soft gray and tan CLAY (CH)
	10		0.0 (P)	0.15 t=4.0	186 318	29 18	403	118	285				Very soft gray ORGANIC CLAY (OH)
	15		0.75 (P) 1.0 (P) 2.0 (P)	1.38 t=6.0	27 25 24	96 100 88	49	12	37				Stiff gray and tan SILTY CLAY (CL)
	20		2.25 (P) 3.0 (P)	1.53 t=7.0	22 21 21 19	98 106 109 110	42	12	30				
	25		0.75 (P) 1.25 (P)	1.06 t=10.0	24 24 37	101 93 86	52	13	39				Stiff gray and tan CLAY (CH)
	30 35		1.75 (P)	2.22 t=14.0	22 22	103 99	36	16	20	86	GSH		Very stiff gray and tan SILTY CLAY (CL) w/ trace sand
	40		1.0 (P)	2.22 t=16.0	26	95	50	17	33				Stiff to very stiff gray CLAY (CH)

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 30 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis
ORG$_1$=15.4%
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-09

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/13/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 55.1"
Long. 92° 12' 39.4"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					27	92							Stiff to very stiff gray CLAY (CH)
	45		2.25 (P)		27	93							
	50		0.5 (P)	1.93 t=20.0	38	84	65	19	46				
					35	81							
	55		1.5 (P)										
	60		2.0 (P)	2.01 t=24.0	28	97	69	25	44				
					24	84							Boring completed at 30 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 30 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-10

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/14/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 46.4"
Long. 92° 13' 0.5"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.15 t=1.0	91	49	95	24	71		ORG_1		Very soft gray CLAY (CH) w/ trace organics
					86	48							
			0.0 (P)										
					78	47							
	5		0.0 (P)	0.20 t=2.0	70	59	84	21	63				
					84	45							
			0.0 (P)										
					99	48							
			0.0 (P)	0.14 t=4.0	80	54	77	20	57		ORG_2		
	10				74	55							
			0.0 (P)		170						ORG_3		
					155	34							
			0.5 (P)	0.87 t=6.0	29	95	51	15	36				Medium to stiff gray and tan CLAY (CH)
					31	89							
	15		1.25 (P)										
			2.0 (P)										
					25	94							
			2.0 (P)	1.40 t=8.0	24	102	53	16	37				
	20				25	92							
			0.25 (P)	1.63 t=10.0	25	100							
	25												
			2.25 (P)	1.49 t=12.0	41	81	93	27	66	98			
	30				46	75							
			2.25 (P)	2.29 t=14.0	24	101	48	15	33	85	GSH		Very stiff gray and tan SILTY CLAY (CL) w/ sand
	35				23	103							
			2.0 (P)		24		51	14	37	30			Loose tan CLAYEY SAND (SC)
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
ORG_1=4.9%
ORG_2=4.6%
ORG_3=24.6%
GSH: Hydrometer Particle Size Analysis
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-10

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/14/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 46.4"
Long. 92° 13' 0.5"
Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					27	97							Loose tan CLAYEY SAND (SC)
	45		3.5 (P)		35 36	80	68	19	49	100	GSH		Soft gray and tan CLAY (CH)
	50		1.0 (P)										
	55		3.0 (P)										
	60		1.5 (P)	0.49 t=24.0	48 42	75 79	71	21	50	97			Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-11

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/21/13
Logged by: D. Thibodaux
Driller: D. Anthony
Rig: Airboat

Location: Lat. 29° 43' 41.75"
Long. 92° 13' 51.92"
Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.12 t=1.0	424	13	421	119	302				Very soft dark brown ORGANIC CLAY (OH)
					481	11							
			0.0 (P)	0.19 t=2.0	54	71							
					87	38							Very soft gray CLAY (CH) w/ organics
	5		0.0 (P)	0.21 t=2.0	85	52	98	23	75	100			
					94	48							
			0.5 (P)	0.11 t=3.0	109	43							
					98	48							
			1.5 (P)	0.14 t=4.0	87	51	80	19	61				
	10				59	53							
			2.25 (P)	0.73 t=5.0	36	86							
					40	81							Stiff gray and tan CLAY (CH)
			3.0 (P)	0.98 t=6.0	30	93							
					35	81							
	15		3.0 (P)	1.01 t=6.0	30	95							
					27	92							
			2.75 (P)	1.21 t=7.0	27	98	53	14	39				
					25	94							
			2.5 (P)	1.35 t=8.0	25	100							
	20				27	95							
			2.25 (P)		28		63	17	46				
	25				35	91							
			2.5 (P)		34		73	21	52				
	30				36	80							Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 30 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information. Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-12

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/12/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 28.6"
Long. 92° 13' 14"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			No (P)										Very soft dark gray ORGANIC CLAY (OH)
			0.0 (P)	0.15 t=2.0	114 95	41 39	130	37	93				
	5		0.0 (P)	0.13 t=2.0	72 64	59 61	78	21	57				Very soft gray CLAY (CH) w/ organics
			0.0 (P)										
			0.0 (P)	0.13 t=4.0	97 75	49 53	99	27	72				
	10		1.5 (P)										Medium gray and tan CLAY (CH)
			1.5 (P)	0.59 t=6.0	44 33 31	78 73 84	55	16	39				
	15		1.5 (P)	0.87 t=6.0	27	101	48	16	32				Medium to stiff gray and tan SILTY CLAY (CL)
			1.5 (P)	1.37 t=7.0	22 25	106 100	46	13	33				
			2.5 (P)										
	20												
			2.5 (P)										
	25												Soft to medium gray and tan CLAY (CH)
			1.5 (P)		27	108	52	16	36				
	30												
			1.5 (P)										
	35												
			0.75 (P)	0.45 t=16.0	36	85	69	17	52				
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-12

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/12/13
Logged by: D. Wyble
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 28.6"
Long. 92° 13' 14"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					35	75							Soft to medium gray and tan CLAY (CH)
			1.0 (P)	0.85 t=18.0	31	90	54	19	35	100	GSH		
	45				34	88							
			0.5 (P)		32		52	17	35				
	50				30	91							
			0.5 (P)										
	55				49	68							
			0.5 (P)										
	60												Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data

GW @ Ground Surface

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis

Strata Boundaries May Not Be Exact

Confidential Information:
Privileged & Confidential
Work Product

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-13

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 0.9"
Long. 92° 13' 4.1"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		93	49	91	24	67				Very soft dark gray CLAY (CH) w/ organics
			0.0 (P)	0.10 t=2.0	425	13	299	81	218	99			Very soft gray ORGANIC CLAY (OH)
					105	34							
	5		0.0 (P)										
			0.5 (P)	0.17 t=3.0	51	74	57	16	41				Very soft gray CLAY (CH) w/ silt pockets and organics
					89	48							
			0.5 (P)										
	10		1.0 (P)	0.21 t=5.0	41	80							
					32	59							
			2.0 (P)	0.76 t=6.0	30	93							Stiff gray and tan CLAY (CH) w/ trace organics
					32	86							
	15		2.0 (P)	1.09 t=6.0	28	97	53	14	39	100			
					27	90							
			3.0 (P)	1.40 t=7.0	25	102							
					25	99							
			3.0 (P)	1.02 t=8.0	27	99	54	16	38	98			
	20				24	99							
			3.5 (P)	1.63 t=10.0	30	95	65	19	46				
	25				29	96							
			3.5 (P)										
	30												Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 30 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-14

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 42' 55.76"
Long. 92° 12' 52.06"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		626	9	594	179	415				Very soft dark brown and gray ORGANICS (PT) w/ trace clay and roots
			0.0 (P)										
	5		0.0 (P)	0.17 t=2.0	71	58	57	17	40				Very soft dark gray CLAY (CH) w/ trace organics
			0.0 (P)	0.10 t=3.0	99	46							
			0.0 (P)										
	10		0.0 (P)	0.15 t=5.0	217	24	312	89	223				Very soft dark brown and gray ORGANICS (PT) w/ trace clay
			0.0 (P)	0.39 t=6.0	33	89							
	15		1.0 (P)	1.12 t=6.0	29	95	44	14	30				Very soft gray SILTY CLAY (CL) w/ trace organics and fenods
			1.0 (P)	1.79 t=7.0	26	100	61	14	47				Very soft gray, tan, and brown CLAY (CH) w/ trace organics and fenods
			1.75 (P)	1.78 t=8.0	25	102							
	20												
			2.25 (P)										
	25												
			2.25 (P)	1.84 t=12.0	28	96	62	19	43				
	30												
	35												
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 40 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-14

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File:	13-80-3705
Date:	10/23/13
Logged by:	J. Garner
Driller:	D. Anthony
Rig:	Ardco K-1000

Location: Lat. 29° 42' 55.76"
Long. 92° 12' 52.06"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
													Very soft gray, tan, and brown CLAY (CH) w/ trace organics and fenods
			3.0 (P)										
	45												Boring completed at 40 ft.
	50												
	55												
	60												
	65												
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 40 ft.	
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method**: Borehole grouted upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-15

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Hamilton
Driller: D. Anthony
Rig: Airboat

Location: Lat. 29° 42' 55.7"
Long. 92° 14' 6.9"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)										Very soft brown ORGANIC CLAY (OH)
			0.0 (P)										Very soft to soft gray CLAY (CH)
	5		0.0 (P)	0.13 t=2.0	68	61	78	19	59	100			
					68	58							
			0.0 (P)	0.13 t=3.0	70	59							
					81	49							
			0.0 (P)				103	24	79	100			
	10				100	45							
			0.0 (P)	0.15 t=5.0	105	45	99	23	76				
					81	45							
			1.0 (P)	0.47 t=6.0	41	81							
					40	76							
	15		1.25 (P)	0.26 t=6.0	42	81	62	16	46				
					42	75							
			1.5 (P)	0.74 t=7.0	32	92							Medium to stiff bluish gray and tan CLAY (CH)
					29	86							
			1.75 (P)	1.63 t=8.0	24	101							
	20												
			2.0 (P)	1.86 t=10.0	23	102	57	15	42				
	25				24	98							
			2.75 (P)										Very stiff light gray and tan CLAY (CH)
	30				30	96							
			4.5 (P)	2.33 t=14.0	22	104	55	14	41				
	35				25	92							
			1.25 (P)										
	40				28	97							

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Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 80 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-15

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Hamilton
Driller: D. Anthony
Rig: Airboat

Location: Lat. 29° 42' 55.7"
Long. 92° 14' 6.9"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
													Very stiff light gray and tan CLAY (CH)
	45		3.25 (P)	1.58 t=18.0	30 31	93 88	46	17	29				Medium to stiff tan, gray, and brown SILTY CLAY (CL)
	50		1.5 (P)		28	93							
	55		1.5 (P)	1.66 t=22.0	44 39	79 82							
	60		1.0 (P)		33	88							
	65		1.75 (P)	1.37 t=26.0	29	95	34	18	16				
	70		0.0 (P)										
	75		0.0 (P)										
	80		0.0 (P)		27 26	93	31	18	13				

Boring completed at 80 ft.

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 80 ft.	
	Boring Abandonment Method: Borehole grouted upon completion	

Confidential Information:
Privileged & Confidential
Work Product

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-16

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/24/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 5.6"
Long. 92° 14' 17.5"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)				86	22	64	99			Very soft gray CLAY (CH) w/ organics and shells
					116	33							
			0.5 (P)	0.09 t=2.0	91	50	96	21	75				
					72	51							
	5		0.0 (P)	0.13 t=2.0	72	58	77	19	58				
					86	51							
			0.0 (P)	0.15 t=3.0	92	49							
					103	41							
			0.0 (P)	0.08 t=4.0	104	45	99	21	78	98			
	10				100	46							
			0.5 (P)	0.20 t=5.0	71	58							
			1.5 (P)	0.66 t=6.0	36	85							Medium to stiff gray and tan CLAY (CH)
					39	82							
	15		0.5 (P)	0.59 t=6.0	37	85	57	15	42				
					33	86							
			2.5 (P)	1.49 t=7.0	25	100							
					29	84							
			2.5 (P)	1.39 t=8.0	24	102	64	14	50				
	20				24	98							
			2.5 (P)										
	25												
			2.5 (P)				57	17	40				
	30				30	83							Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 30 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-17

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/26/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 42' 40.3"
Long. 92° 12' 4.5"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)										Very soft dark gray ORGANIC CLAY (OH)
			0.0 (P)	0.11 t=2.0	105 135 122	33 36 31	136	34	102				
	5		0.0 (P)										
			0.5 (P)	0.11 t=3.0	173 149 149	22 34 26	149	36	113				
			0.5 (P)	0.16 t=4.0	220	24							
	10		0.5 (P)										
			0.5 (P)	0.21 t=6.0	447 42 36	11 79 87	48	15	33				Very soft gray SILTY CLAY (CL) w/ organics
	15		1.0 (P)	0.31 t=6.0	37 34	65 88	61	16	45				Medium to stiff dark gray CLAY (CH)
			2.5 (P)	0.53 t=6.0 0.81 t=7.0	32 30 27	80 94 89							
			2.5 (P)	1.19 t=8.0	29 26	95 84	60	15	45				
	20												
			2.5 (P)										
	25												
			3.0 (P)										
	30												
			1.5 (P)		21					30			Medium dense gray and tan SILTY SAND (SM)
	35												
			1.5 (P)	0.73 t=16.0	34	89	66	17	49				Medium gray CLAY (CH) w/ trace organics
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-17

Ardaman & Associates, Inc.

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/26/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 42' 40.3"
Long. 92° 12' 4.5"
Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					31	84							Medium gray CLAY (CH) w/ trace organics
			2.0 (P)										Very stiff tan and gray SILTY CLAY (CL) w/ silt pockets
	45												
			2.0 (P)	2.00 t=20.0	40	89							
					41	74							
	50												
			2.0 (P)	2.05 t=22.0	28	95	39	15	24				
					33	83							
	55												
			2.0 (P)										
	60												Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 60 ft.	
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-18

Ardaman & Associates, Inc.

Sheet 1 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/24/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 42' 2.5"
Long. 92° 13' 23.1"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.5 (P)		70	38							Very soft gray and dark gray CLAY (CH) w/ organics
			0.5 (P)	0.21 t=2.0	51	70	72	16	56				
					165	27							
	5		0.5 (P)		59	53							
			0.5 (P)	0.21 t=3.0	57	68	74	17	57				
					61	48							
			0.5 (P)	0.17 t=4.0	63	64	80	18	62				
	10				53	60							
			0.0 (P)										
					117	37							
			0.5 (P)										
					89	44							
			0.5 (P)	0.18 t=6.0	84	52	82	20	62				
	15				181	28							
				0.20 t=6.0	193	28							
			0.5 (P)										
					84	45							
			1.0 (P)	0.75 t=8.0	29	94	72	19	53				
	20				45	64							
			2.5 (P)	1.31 t=10.0	26	99							Medium to stiff tan, brown, and gray CLAY (CH)
	25				27	92							
			2.0 (P)	0.98 t=12.0	39	83	80	20	60				
	30				37	79							
			2.5 (P)										
	35				32	88							
			3.0 (P)	1.16 t=16.0	32	89	68	16	52	99			
	40												

Continued Next Page

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.
4" Dia. Rotary Wash:
20 to 60 ft.

Boring Abandonment Method

Borehole grouted upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-18

Ardaman & Associates, Inc.

Sheet 2 of 2

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/24/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 42' 2.5"
Long. 92° 13' 23.1"

Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
					32	86							Medium to stiff tan, brown, and gray CLAY (CH)
			2.5 (P)	1.48 t=18.0	33	90							
	45				38	80							
			3.0 (P)										
	50												
			1.0 (P)										
	55												
			1.5 (P)										
	60												Boring completed at 60 ft.
	65												
	70												
	75												
	80												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 60 ft.	
Confidential Information: Privileged & Confidential Work Product	Boring Abandonment Method: Borehole grouted upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705 GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-19

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/21/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 6.2"
Long. 92° 10' 25"

Surface Elevation: -3 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			1.5 (P)	0.14 t=1.0	45	78	48	13	35	99			Very soft gray SILTY CLAY (CL)
					39	72							
			1.5 (P)										
					28	92							
	5		3.0 (P)	1.22 t=2.0	29	94	46	14	32				Stiff to very stiff gray, tan, and brown SILTY CLAY (CL) w/ trace organics
					30	90							
			2.5 (P)	2.40 t=3.0	22	105	48	13	35				
					22	103							
			1.5 (P)										
	10				23	102							
			2.0 (P)										
					24	101							
			2.0 (P)	1.46 t=6.0	26	101	41	14	27				
					26	89							
	15		1.5 (P)										
					25	97							
			1.0 (P)	0.85 t=7.0	27	98	38	12	26				Medium tan SILTY CLAY (CL) w/ trace sand
					26	90							
			1.0 (P)		26					90			
	20				25	92							Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information. Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole backfilled with soil upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-20

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/22/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 6.4"
Long. 92° 10' 14.7"
Surface Elevation: -3 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.5 (P)		36	66							Very soft gray CLAY (CH) w/ trace organics
			0.5 (P)	0.14 t=2.0	105	46	87	18	69				
					84	82							
	5		2.5 (P)	1.36 t=2.0	24	104	46	15	31				Stiff bluish gray and tan SILTY CLAY (CL)
					24	95							
			3.5 (P)		22	102							Stiff bluish gray and tan CLAY (CH) w/ trace organics
			3.5 (P)	1.46 t=4.0	23	104	50	15	35				
	10				24	102							
			4.5 (P)	1.85 t=5.0	26	99	58	14	44				
					27	96							
			4.0 (P)		26	89							
	15		2.0 (P)		27	96							Stiff tan and gray SILTY CLAY (CL) w/ trace organics
			1.0 (P)	1.42 t=7.0	27	95	38	15	23	98	GSH		
					26	93							
			1.5 (P)		27	91							
	20												Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data

GW @ Ground Surface

Confidential Information:
Privileged & Confidential
Work Product

Boring Advancement Method

4" Nom. Dia. Short Flight Auger:
0 to 20 ft.

Boring Abandonment Method

Borehole backfilled with soil upon completion

Notes

t: Unconsolidated, Undrained Triaxial Compression Test
Lateral Pressure = psi
GSH: Hydrometer Particle Size Analysis
*Boring Elevations are approximate

Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-21

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/22/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 6.6"
Long. 92° 10' 4.5"

Surface Elevation: -3 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.14 t=1.0	175	30	175	36	139				Very soft gray and brown CLAY (CH) w/ organics
					158	32							
			0.0 (P)	0.08 t=2.0	105	45	94	20	74				
					99	43							
	5		0.0 (P)										
					100	39							
			0.5 (P)	0.12 t=3.0	133	38	143	32	111				
					103	40							
			0.5 (P)										
	10				87	48							
			0.5 (P)	0.16 t=5.0	87	51	97	23	74				
					84	47							
			2.0 (P)										Medium bluish gray SILTY CLAY (CL) w/ trace fine sand and trace organics
					26	85							
	15		3.0 (P)	0.82 t=6.0	26	102	28	20	8	86			
					24	100							
			1.0 (P)										Hard gray SANDY CLAY (SC)
					23	98							
			0.5 (P)	4.91 t=8.0	22	107				52			
	20				28	82							
													Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole backfilled with soil upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-22

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 0.7"
Long. 92° 10' 24.7"

Surface Elevation: -4 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		184	28							Very soft dark gray and brown ORGANIC CLAY (OH)
			2.0 (P)										Stiff gray and tan SILTY CLAY (CL) w/ trace organics
	5		4.0 (P)	1.70 t=2.0	24 23	103 100	42	16	26				
			4.0 (P)	3.11 t=3.0	20 20	109 103	54	14	40				Very stiff brown, gray, and tan CLAY (CH)
			3.0 (P)										
	10		2.0 (P)	2.17 t=5.0	27 27	98 97	56	17	39				
			4.0 (P)		28	98							Stiff gray and tan SILTY CLAY (CL)
	15		3.5 (P)	1.05 t=6.0	28 26	99 99	42	15	27				
			2.0 (P)		42	91							
			2.0 (P)	1.53 t=8.0	27 28	99 83	47	16	31				
	20												Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole backfilled with soil upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

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LOG OF SOIL BORING B-23

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 1.3"
Long. 92° 10' 14.7"

Surface Elevation: -4 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)				106	20	86				Very soft gray CLAY (CH) w/ organics
					112	41							
			0.0 (P)	0.13 t=2.0	78	55	81	16	65				
					40	64							
	5		2.0 (P)	0.55 t=2.0	32	92	45	14	31				Medium to stiff gray SILTY CLAY (CL) w/ trace organics
					34	84							
			3.0 (P)	1.41 t=3.0	25	103	45	16	29				
					25	93							
			1.5 (P)										
	10				25	94							
			2.5 (P)	1.42 t=5.0	25	102	50	15	35				Medium to stiff gray and tan CLAY (CH) w/ trace organics
					26	97							
			3.0 (P)										
					28	83							
	15		2.0 (P)										
			1.5 (P)	0.81 t=7.0	28	95	50	16	34				
					29	93							
			2.0 (P)	0.96 t=8.0	28	96	37	16	21	97	GSH		Medium light gray and tan SILTY CLAY (CL)
	20				29	87							
													Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi GSH: Hydrometer Particle Size Analysis *Boring Elevations are approximate
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole backfilled with soil upon completion	Strata Boundaries May Not Be Exact

ARD LOG01 01R 13-80-3705.GPJ LOG01R.GDT 3/24/14

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-24

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/23/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 43' 1.2"
Long. 92° 10' 4.3"

Surface Elevation: -4 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)		182	29							Very soft gray and brown CLAY (CH) w/ organics
			0.0 (P)	0.08 t=2.0	114	43	90	19	71				
					102	43							
	5		0.0 (P)	0.07 t=2.0	106	44	93	21	72				
					98	51							
			0.0 (P)										
					276	20							
			0.0 (P)	0.13 t=4.0	98	47	140	29	111				
	10				180	31							
			0.5 (P)										
					138	32							
			2.0 (P)										Medium to stiff bluish gray SILTY CLAY (CL) w/ trace organics
					171	28							
	15		2.5 (P)	0.97 t=6.0	27	98	45	13	32				
					27	98							
			4.5+ (P)										
					25	92							
			3.5 (P)	1.38 t=8.0	22	106	44	13	31				
	20				22	96							Boring completed at 20 ft.
	25												
	30												
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi *Boring Elevations are approximate
Confidential Information Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole backfilled with soil upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-25

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/16/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 27.2"
Long. 92° 13' 45.7"
Surface Elevation: 1 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	PL	PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)				110	26	84	100			Very soft dark brown and gray CLAY (CH) w/ organics
					352	12							
			0.0 (P)	0.08 t=2.0	155	33	128	29	99	99			
					63	45							
	5		0.0 (P)	0.13 t=2.0	111	43	107	23	84				
					113	44							
			0.5 (P)										
					118	45							
			1.5 (P)	0.51 t=4.0	34	88	59	16	43				Medium to stiff gray and tan CLAY (CH) w/ trace organics
					36	83							
	10		2.5 (P)	1.15 t=5.0	28	96	59	12	47				
					28	91							
			2.5 (P)	2.65 t=6.0	22	106							
					29	90							
	15		3.5 (P)	2.53 t=6.0	22	106							
			2.0 (P)										
			1.5 (P)	0.98 t=8.0	27	97	59	14	45				
	20												
			2.5 (P)	2.16 t=10.0	27	96	63	16	47	89	GSH		Very stiff tan and gray CLAY (CH) w/ trace sand and organics
	25				41	90							
			2.0 (P)										
	30												Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 30 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi GSH: Hydrometer Particle Size Analysis *Boring Elevations are approximate
Confidential Information: Privileged & Confidential Work Product	**Boring Abandonment Method** Borehole grouted upon completion	Strata Boundaries May Not Be Exact

Cole's Bayou Marsh Restoration
(TV-63)
Vermillion Parish, LA

CPRA

LOG OF SOIL BORING B-26

Ardaman & Associates, Inc.

Sheet 1 of 1

Baton Rouge Geotechnical Laboratory
AASHTO Accredited Laboratory , LELAP Certificate No. 02052,
USACE Validated

File: 13-80-3705
Date: 10/16/13
Logged by: J. Garner
Driller: D. Anthony
Rig: Ardco K-1000

Location: Lat. 29° 44' 24.7"
Long. 92° 13' 26.9"

Surface Elevation: 2 (ft., NAVD)

FIELD DATA				LABORATORY DATA									
Ground Water Level	Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Percent Passing #200 Sieve	Other	Soil Type	Description
			0.0 (P)	0.10 t=1.0	81	55							Very soft dark brown ORGANIC CLAY (OH)
					119	32							
			0.0 (P)	0.08 t=2.0	159	31	145	34	111		ORG$_1$		
					71	47							
	5		0.0 (P)	0.15 t=2.0	84	53							Very soft gray CLAY (CH) w/ organics
					91	53							
			0.0 (P)	0.14 t=5.0	96	48	104	24	80	98			
					100	38							
			0.5 (P)	0.34 t=4.0	39	83							
	10				37	79							Medium gray and tan CLAY (CH)
			0.5 (P)	0.73 t=5.0	30	94	55	16	39				
					26	94							
			2.0 (P)	2.43 t=6.0	22	106							Very stiff gray and tan CLAY (CH)
					20	94							
	15		2.5 (P)	2.78 t=6.0	21	106							
					24	96							
			3.5 (P)	2.26 t=7.0	21	107	50	15	35				
					24	108							
			4.5 (P)										
	20				24	100							
			2.0 (P)	0.65 t=10.0	31	93	54	16	38	91	GSH		Medium to stiff gray and tan CLAY (CH)
	25			1.87 t=10.0	29	92							
					25	101							
			2.5 (P)	1.33 t=12.0	32	90	72	22	50				
	30				28	95							Boring completed at 30 ft.
	35												
	40												

Ground Water Level Data	Boring Advancement Method	Notes
GW @ Ground Surface	4" Nom. Dia. Short Flight Auger: 0 to 20 ft. 4" Dia. Rotary Wash: 20 to 30 ft.	t: Unconsolidated, Undrained Triaxial Compression Test Lateral Pressure = psi ORG$_1$=67.7% GSH: Hydrometer Particle Size Analysis *Boring Elevations are approximate
	Boring Abandonment Method	
Confidential Information. Privileged & Confidential Work Product	Borehole grouted upon completion	Strata Boundaries May Not Be Exact

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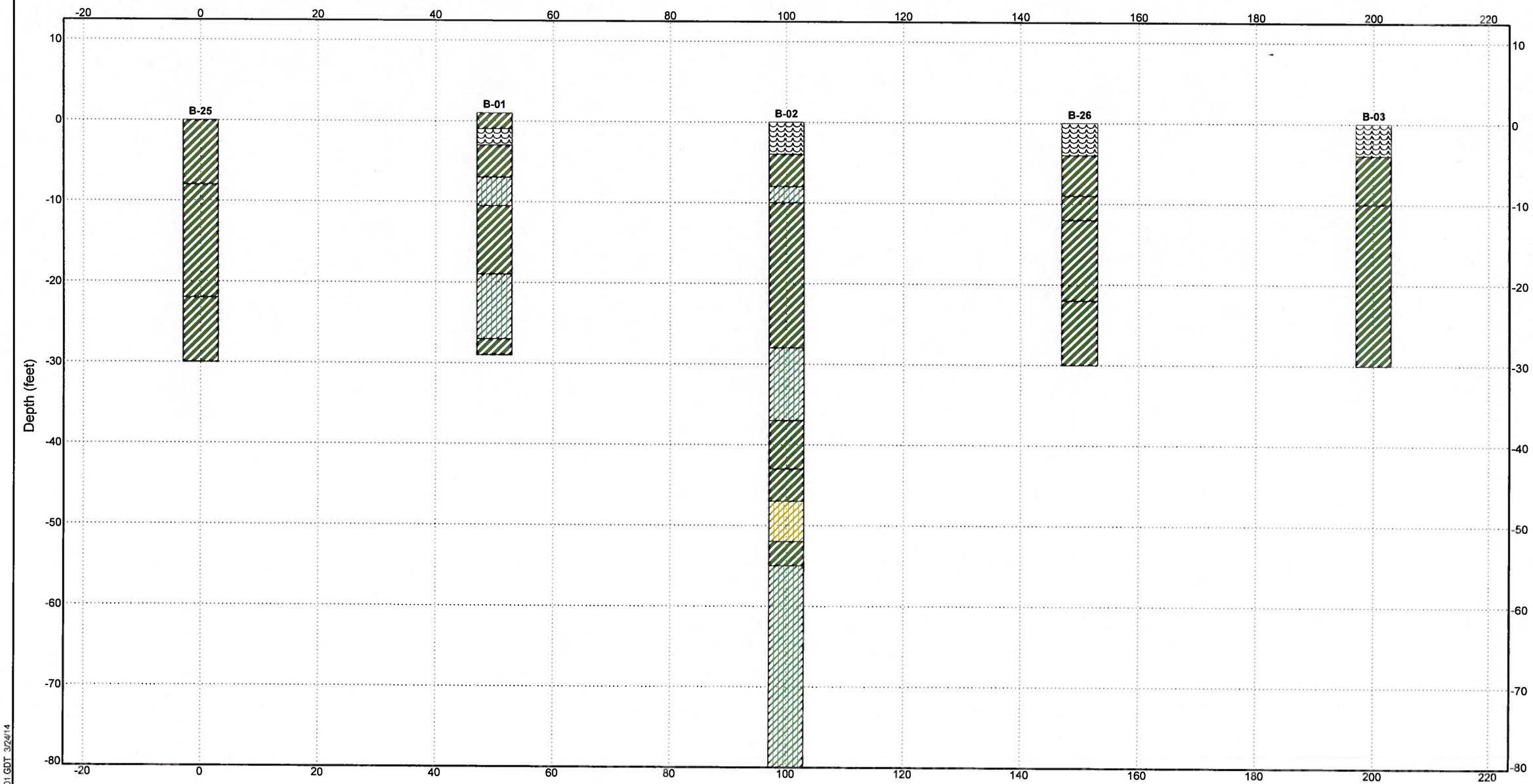

ARD CROSS SECTION 13-80-3705.GPJ LOG01.GDT 3/24/14

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LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:
Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-2

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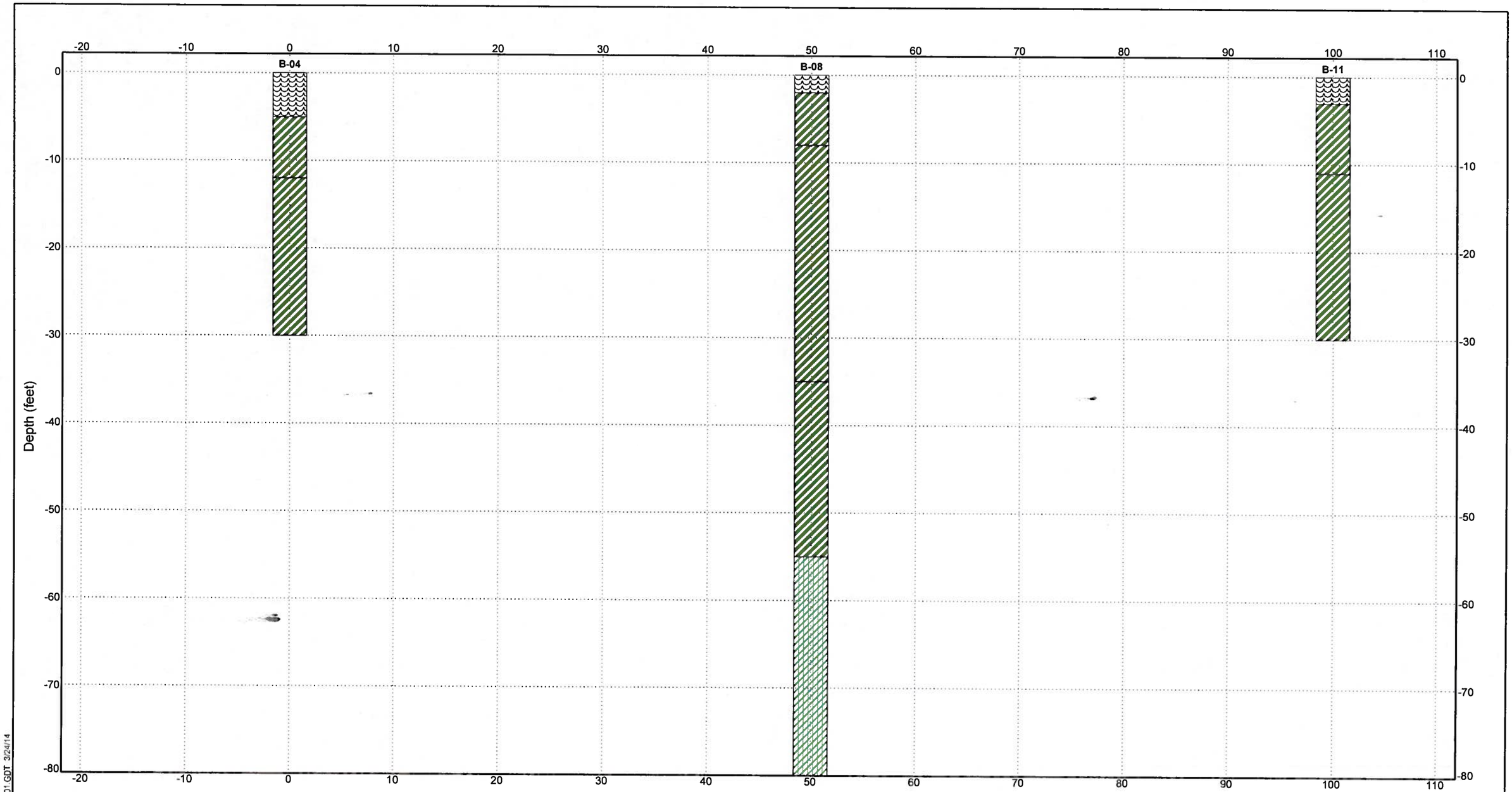

Ardaman & Associates, Inc.

LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-3

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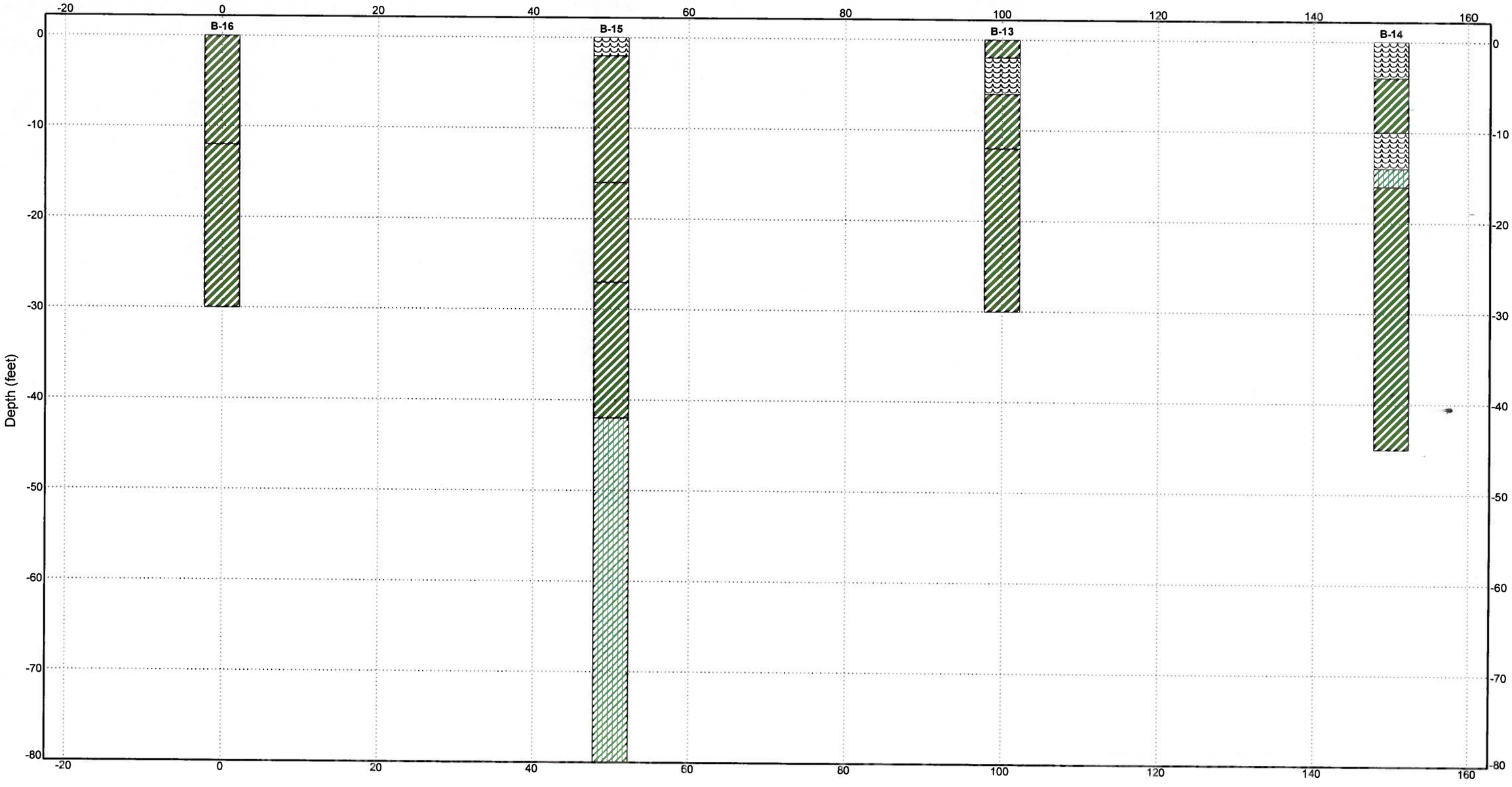

Ardaman & Associates, Inc.

LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-4

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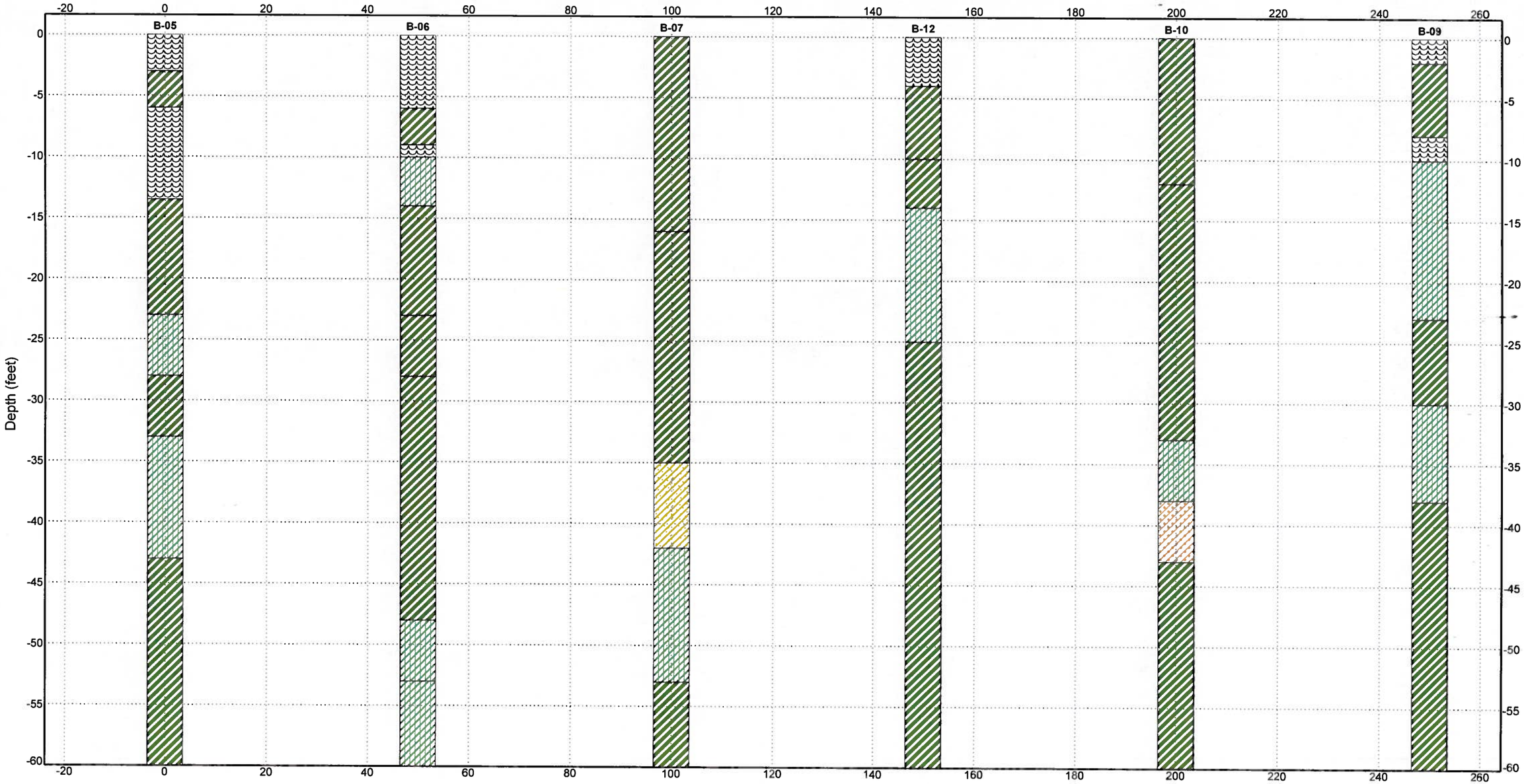

ARD CROSS SECTION 13-80-3705.GPJ LOG01.GDT 3/24/14

Ardaman & Associates, Inc.

LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-5

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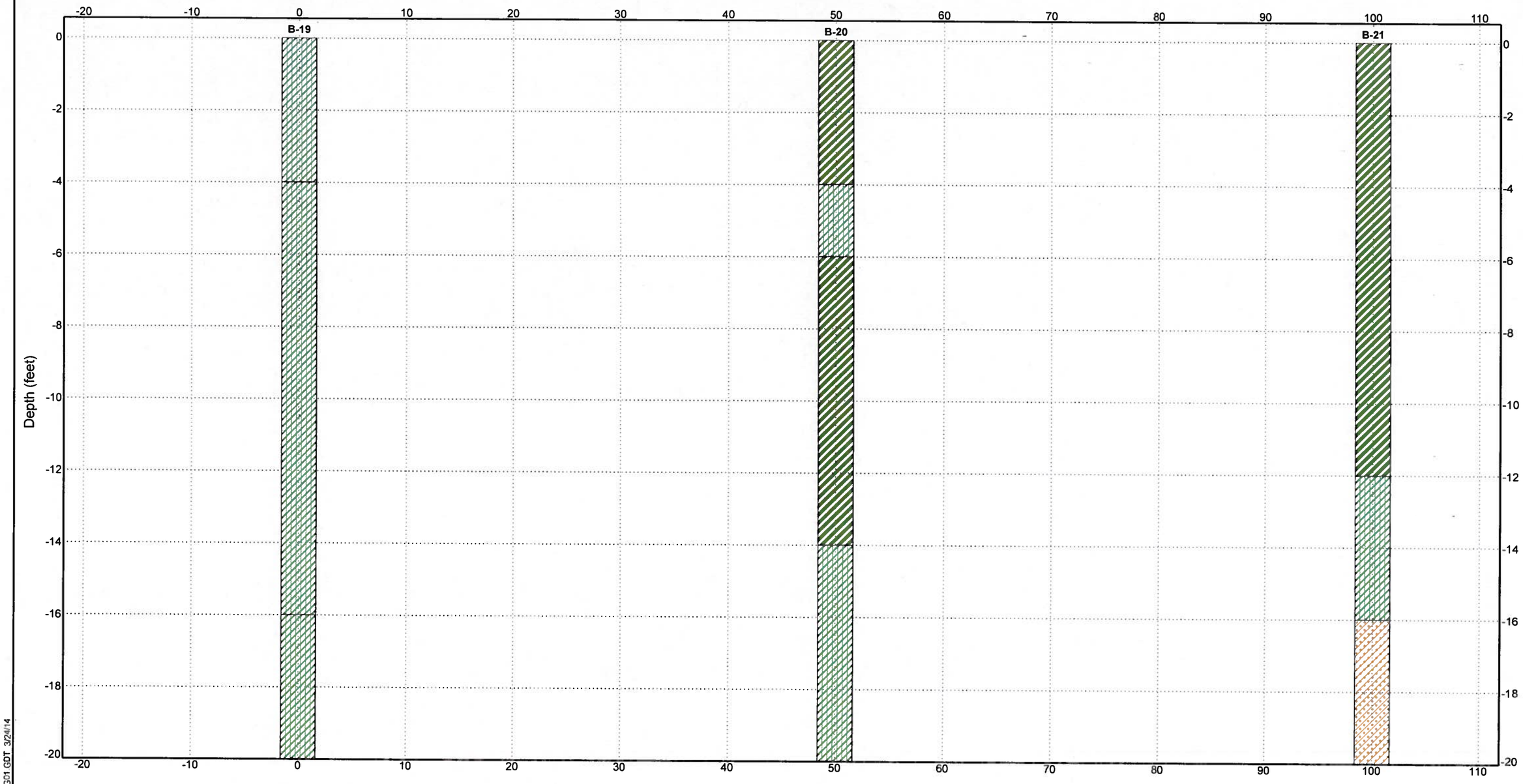

ARD CROSS SECTION 13-80-3705.GPJ LOG01.GDT 3/24/14

Ardaman & Associates, Inc.

LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-6

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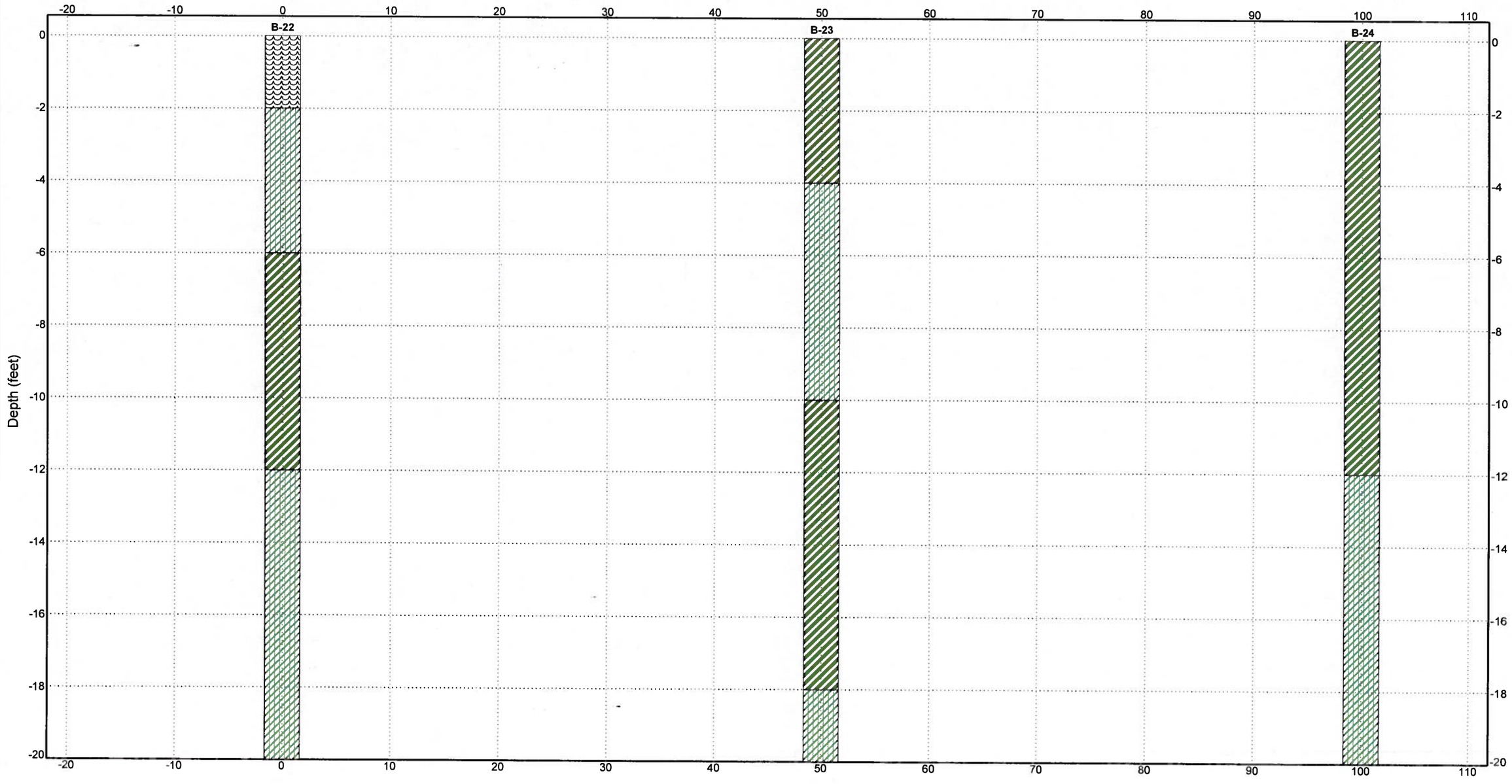

ARD CROSS SECTION 13-80-3705.GPJ LOG01.GDT 3/24/14

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LITHOLOGY GRAPHICS

- USCS High Plasticity Clay
- USCS Low Plasticity Clay
- USCS Low Plasticity Sandy Clay
- USCS High Plasticity Organic silt or clay
- USCS Clayey Sand
- USCS Silty Sand

Title: Cross Section Diagram

Geotechnical Investigation:

Cole's Bayou Marsh Restoration (TV-63)
Vermillion Parish, LA
for:
CPRA

Ardaman & Associates, Inc.
Baton Rouge, LA

File No.: 13-80-3705

Figure No.: A-7

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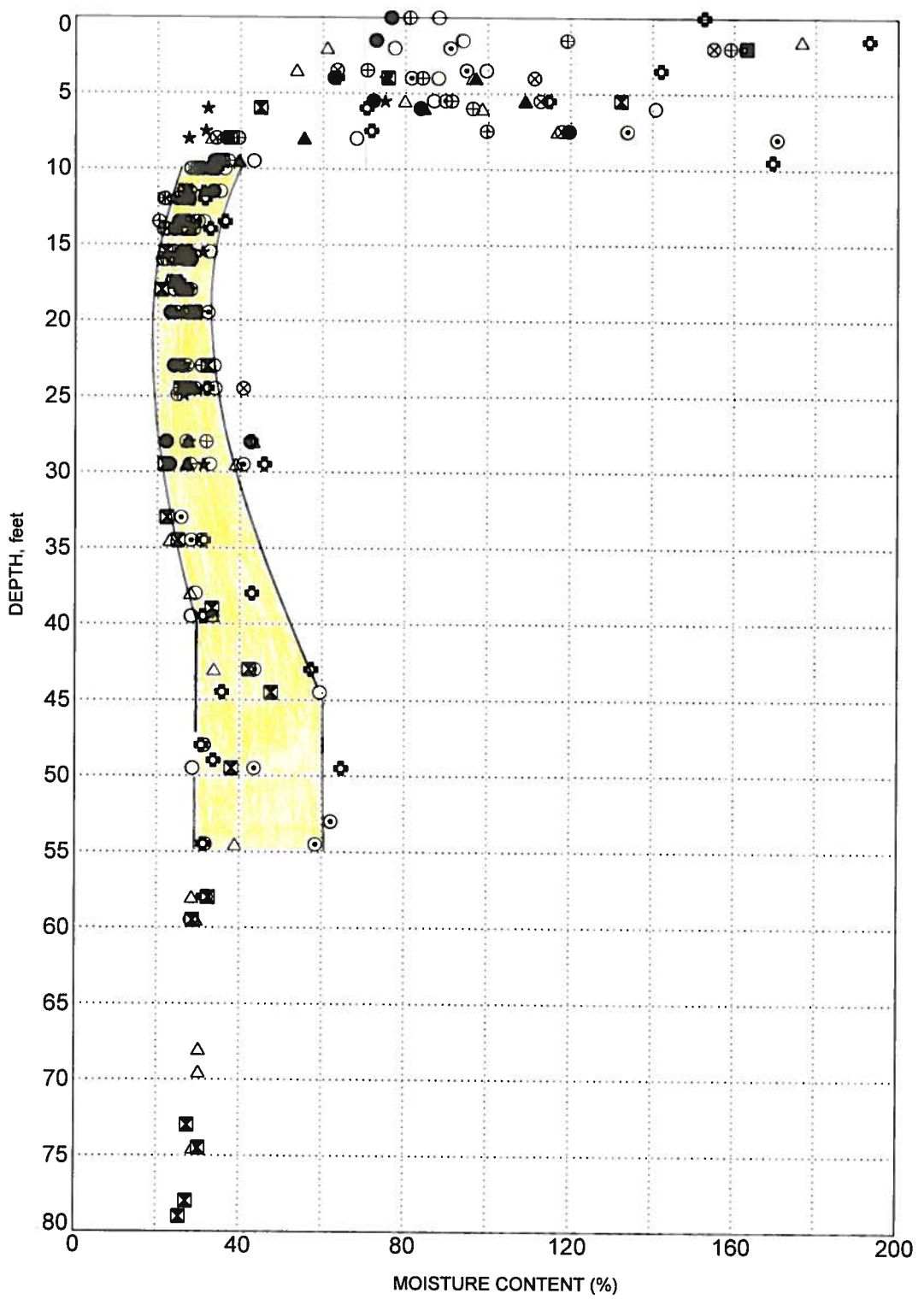
APPENDIX B. LABORATORY DATA PLOTS

This Appendix contains the following:

- Figure B-1A – Moisture Content vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-1B – Moisture Content vs. Depth (Marsh Creation Area 3)
- Figure B-1C – Moisture Content vs. Depth (Hydraulic Structures)
- Figure B-2A – Wet Density vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-2B – Wet Density vs. Depth (Marsh Creation Area 3)
- Figure B-2C – Wet Density vs. Depth (Hydraulic Structures)
- Figure B-3A – Dry Density vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-3B – Dry Density vs. Depth (Marsh Creation Area 3)
- Figure B-3C – Dry Density vs. Depth (Hydraulic Structures)
- Figure B-4A – Organic Content vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-4B – Organic Content vs. Depth (Marsh Creation Area 3)
- Figure B-4C – Organic Content vs. Depth (Hydraulic Structures)
- Figure B-5A – Plasticity Chart (Marsh Creation Areas 1 & 2)
- Figure B-5B – Plasticity Chart (Marsh Creation Area 3)
- Figure B-5C – Plasticity Chart (Hydraulic Structures)
- Figure B-6A – Liquid Limit vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-6B – Liquid Limit vs. Depth (Marsh Creation Area 3)
- Figure B-6C – Liquid Limit vs. Depth (Hydraulic Structures)
- Figure B-7A – Plasticity Index vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-7B – Plasticity Index vs. Depth (Marsh Creation Area 3)
- Figure B-7C – Plasticity Index vs. Depth (Hydraulic Structures)
- Figure B-8A – Liquidity Index vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-8B – Liquidity Index vs. Depth (Marsh Creation Area 3)
- Figure B-8C – Liquidity Index vs. Depth (Hydraulic Structures)
- Figure B-9 – Hydrometer Particle Size Analyses Results
- Figure B-10 – Correlations between Compressibility and Index Properties
- Figure B-11 – $C_{\alpha\varepsilon}$ vs. CR
- Figure B-12 – Stress History Profiles
- Figure B-13 – Coefficient of Consolidation vs. Stress Ratio
- Figure B-14A – UUC Shear Strength vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-14B – UUC Shear Strength vs. Depth (Marsh Creation Area 3)
- Figure B-14C – UUC Shear Strength vs. Depth (Hydraulic Structures)
- Figure B-15A – Vertical Effective Stress and Undrained Shear Strength vs. Depth (Marsh Creation Areas 1 & 2)
- Figure B-15B – Vertical Effective Stress and Undrained Shear Strength vs. Depth (Marsh Creation Area 3)
- Figure B-16 – Moisture Content vs. Depth (Borrow Area)
- Figure B-17 – Wet Density vs. Depth (Borrow Area)

- Figure B-18 – Dry Density vs. Depth (Borrow Area)
- Figure B-19 – Organic Content vs. Depth (Borrow Area)
- Figure B-20 – Plasticity Chart (Borrow Area)
- Figure B-21 – Liquid Limit vs. Depth (Borrow Area)
- Figure B-22 – Plasticity Index vs. Depth (Borrow Area)
- Figure B-23 – Liquidity Index vs. Depth (Borrow Area)
- Figure B-24 – UUC Shear Strength vs. Depth (Borrow Area)
- Figure B-25 – 2006 Aerial and Borrow Area Boring



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MOISTURE CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-1A

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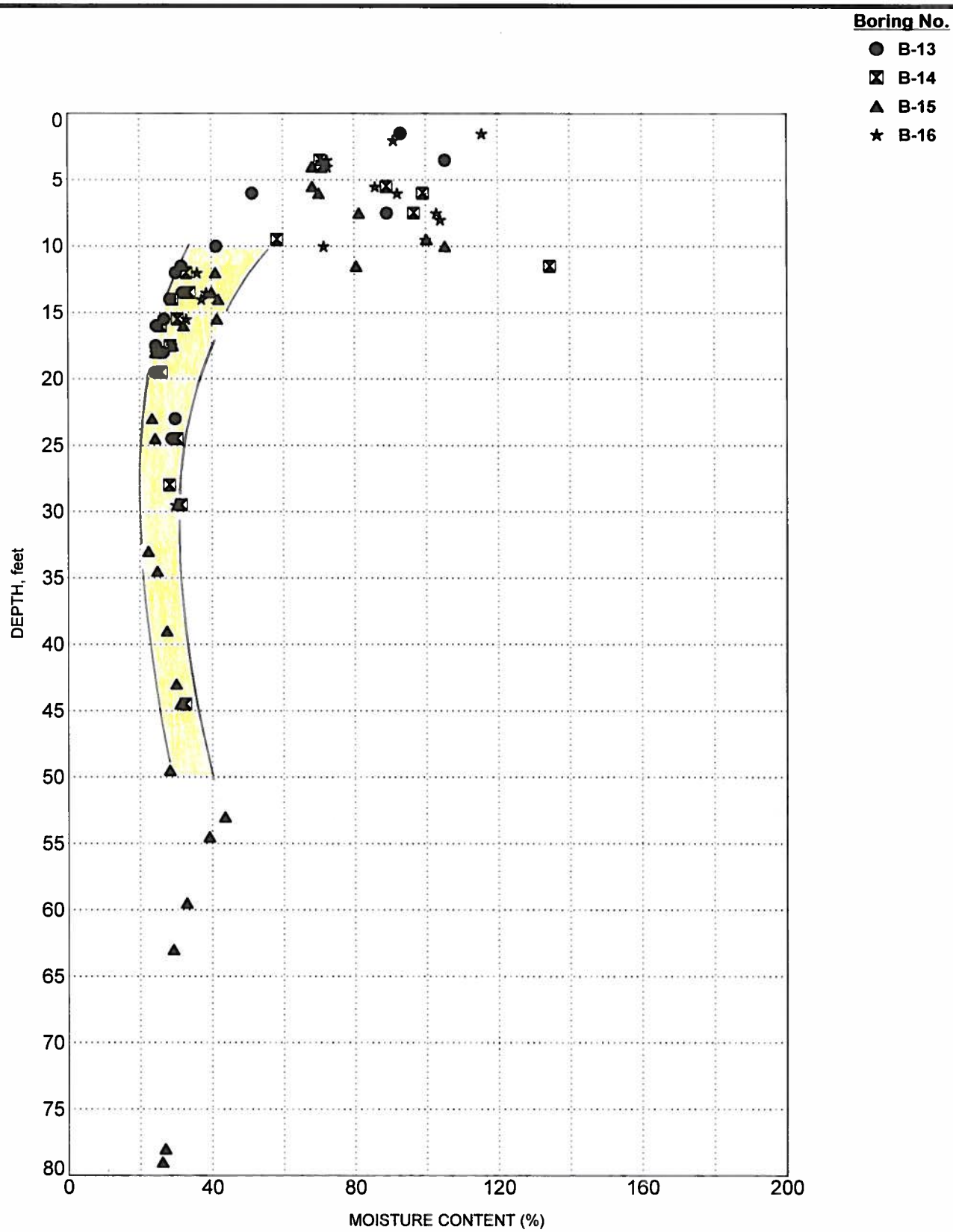
Work Product
ARD DEPTH VS MOISTURE CONTENT 13-80-3705.GPJ US_LAB.GDT 3/27/14



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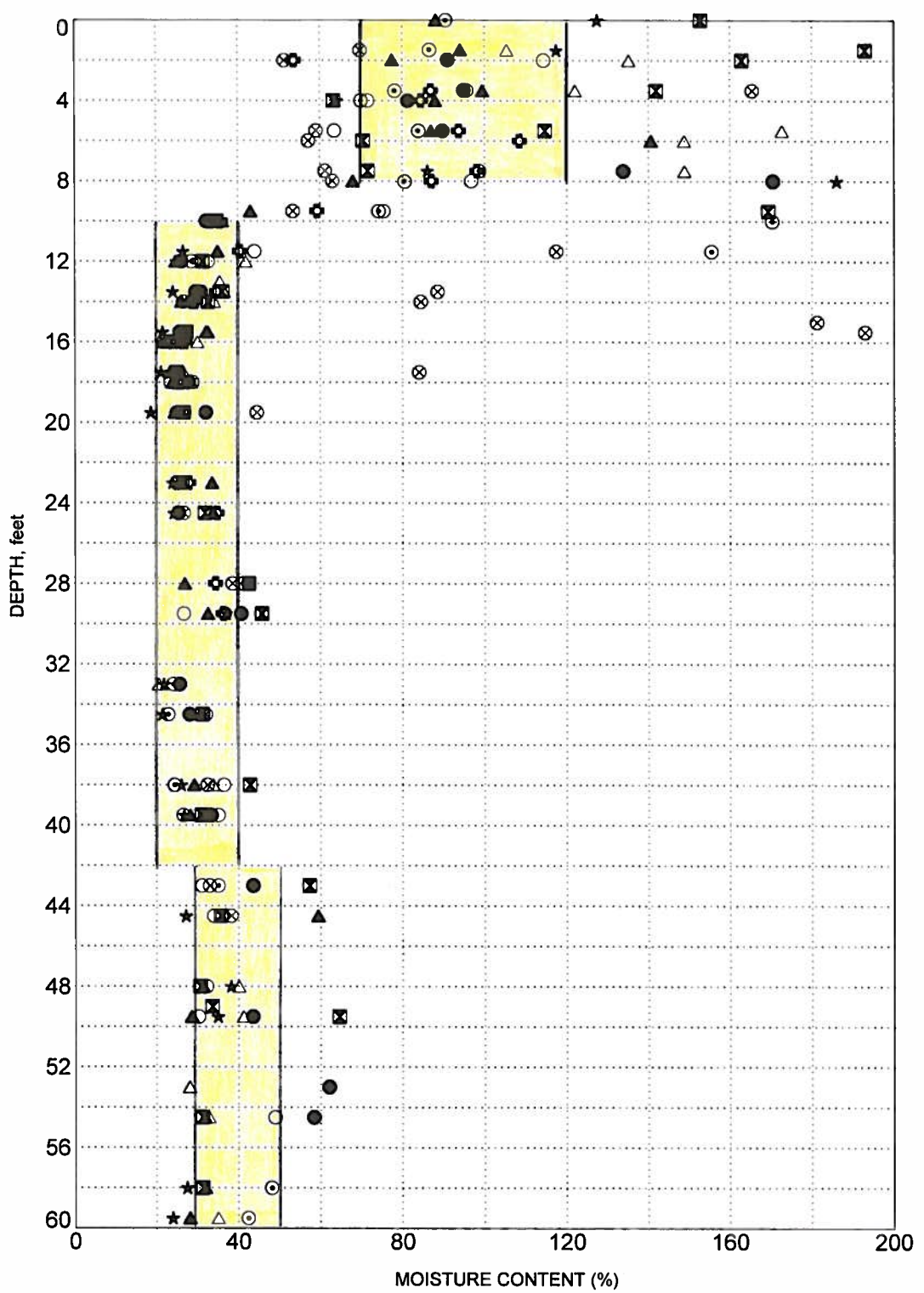
MOISTURE CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-1B



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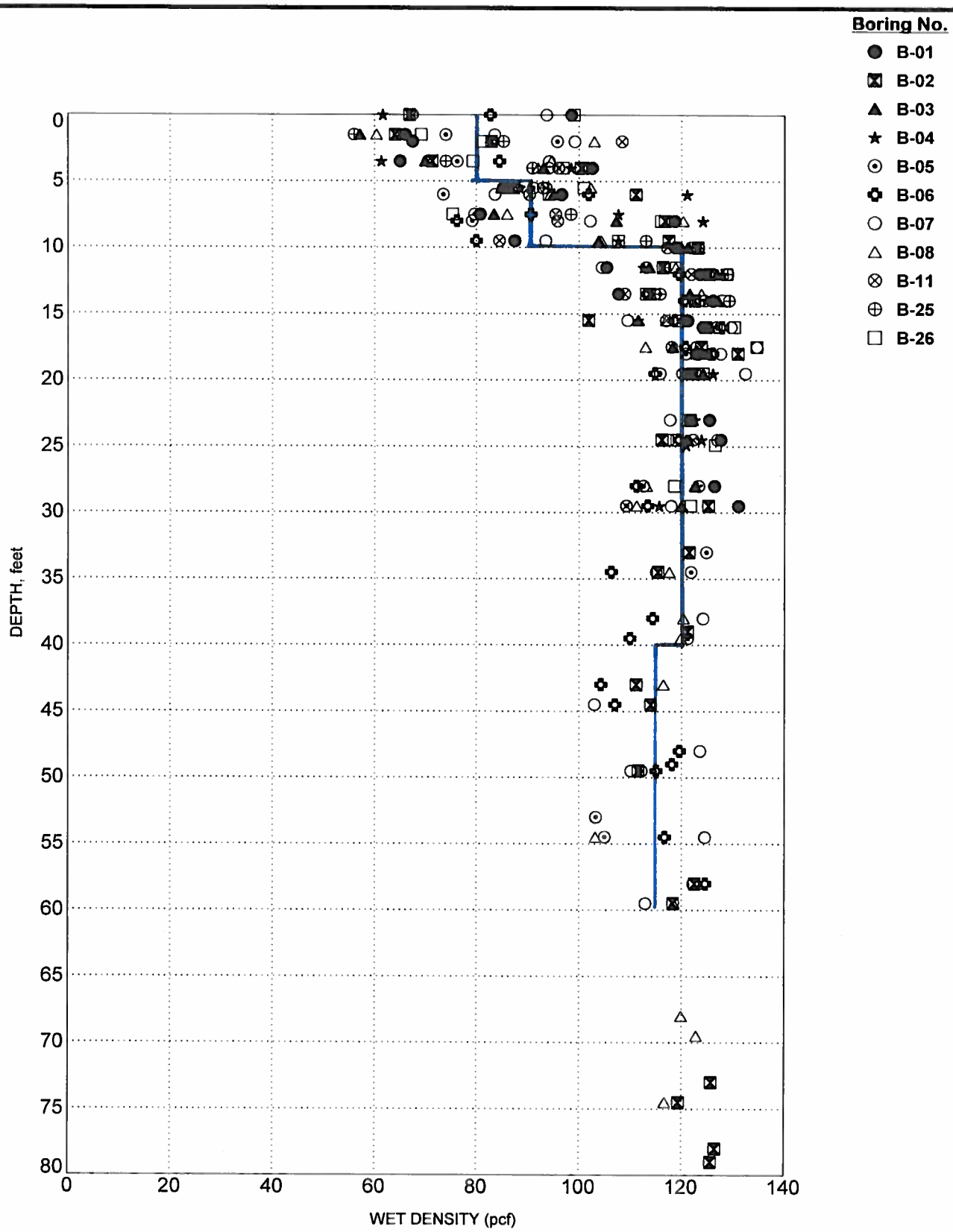
MOISTURE CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-1C



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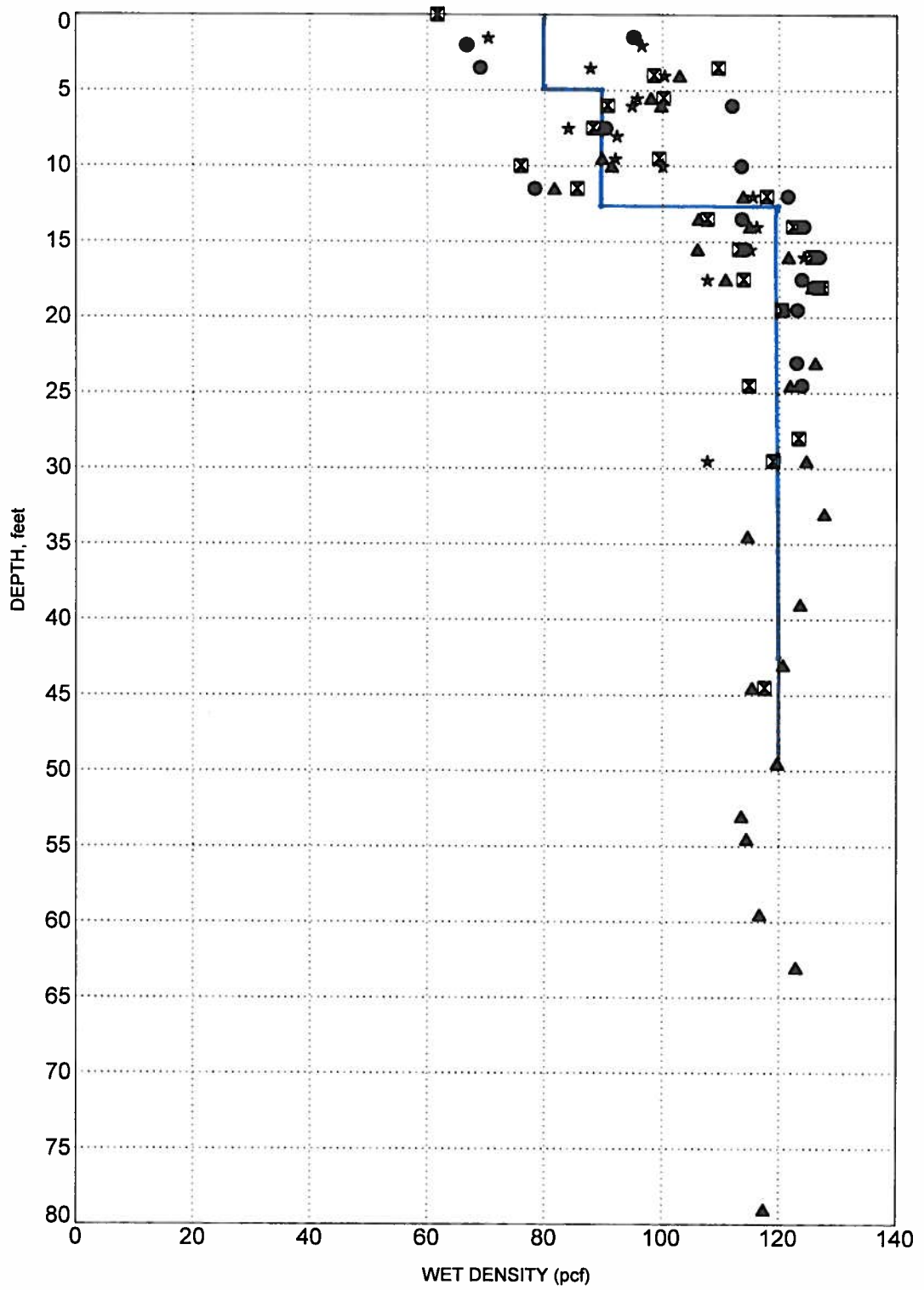
WET DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-2A



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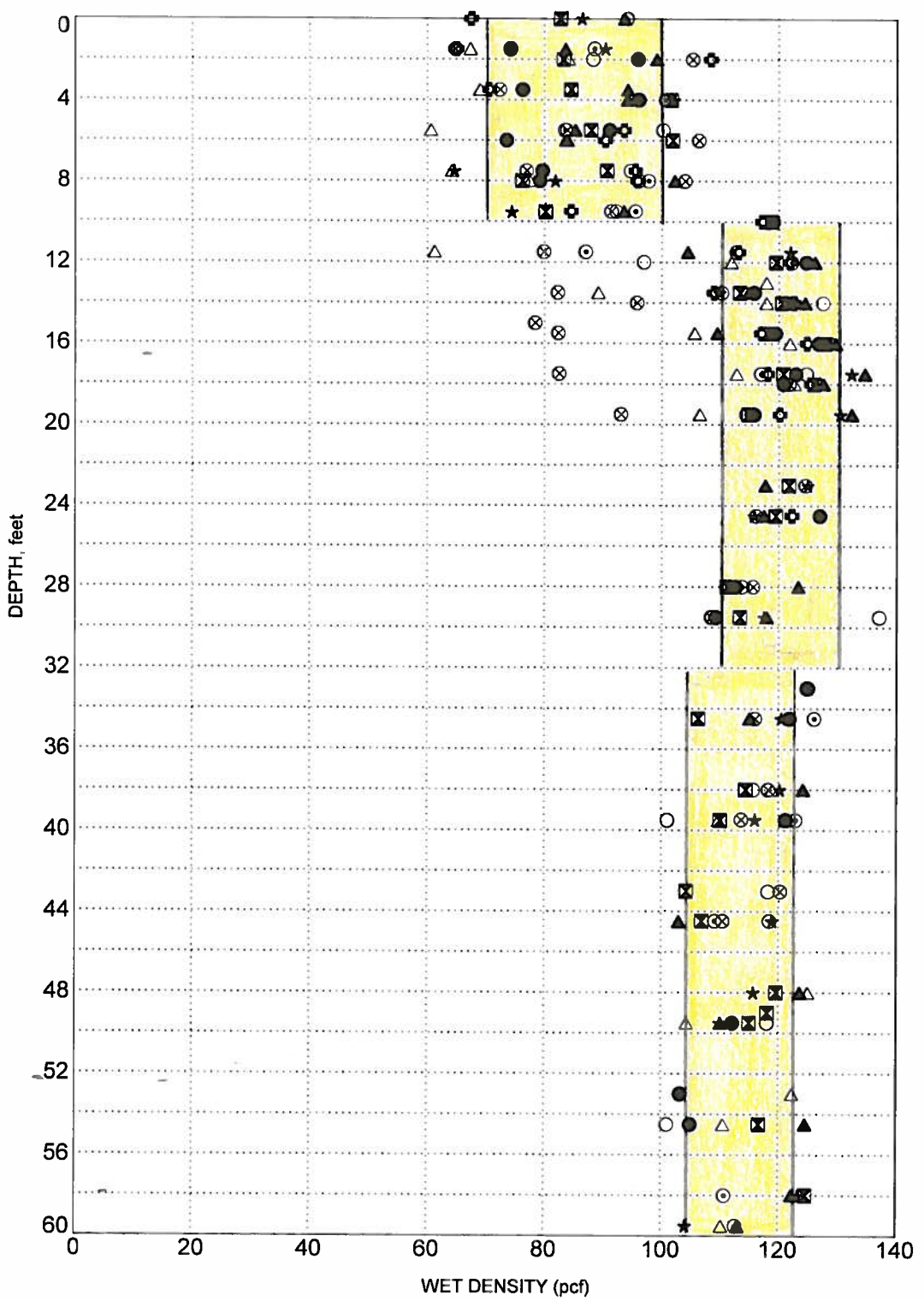
WET DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-2B



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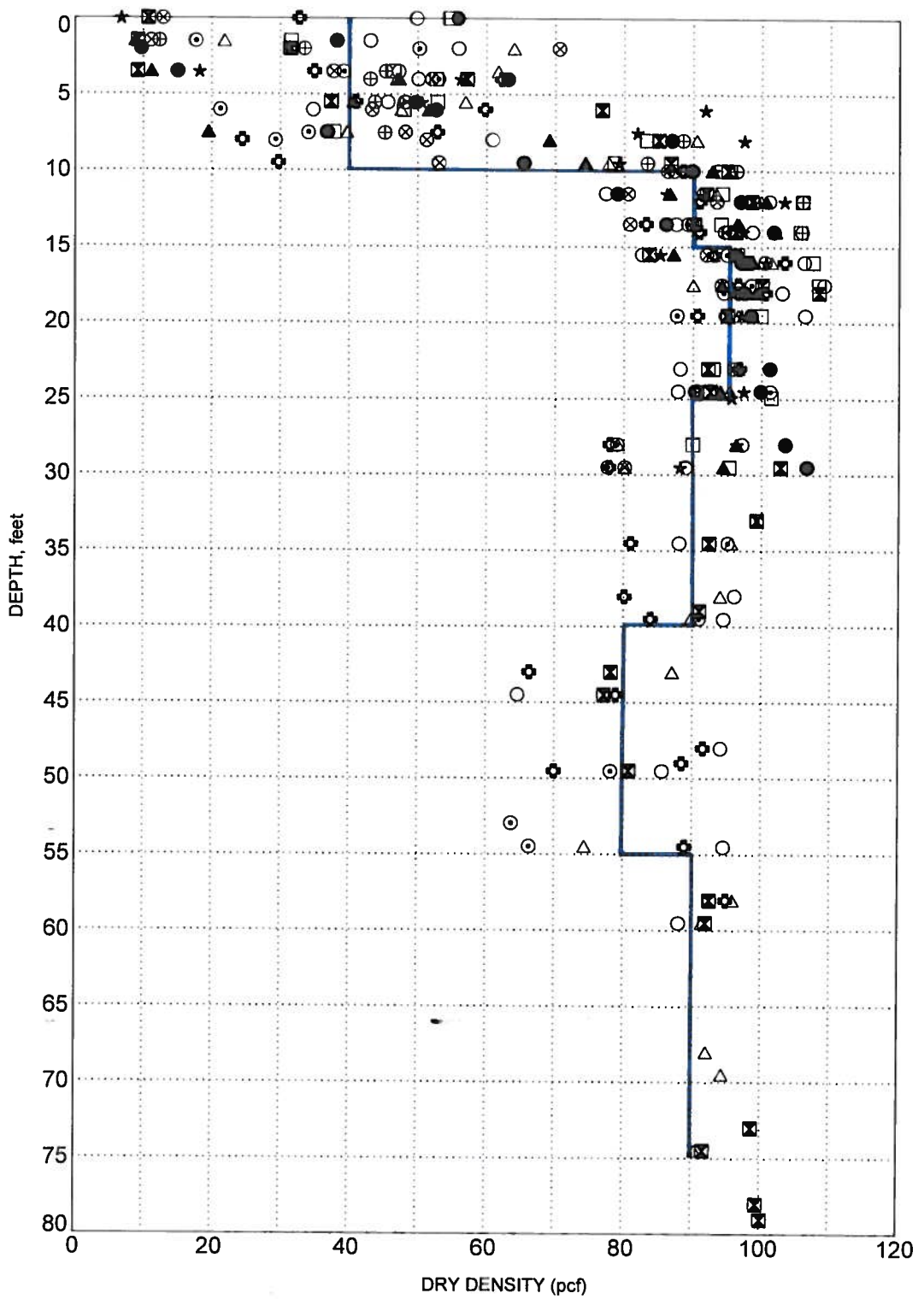
WET DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-2C



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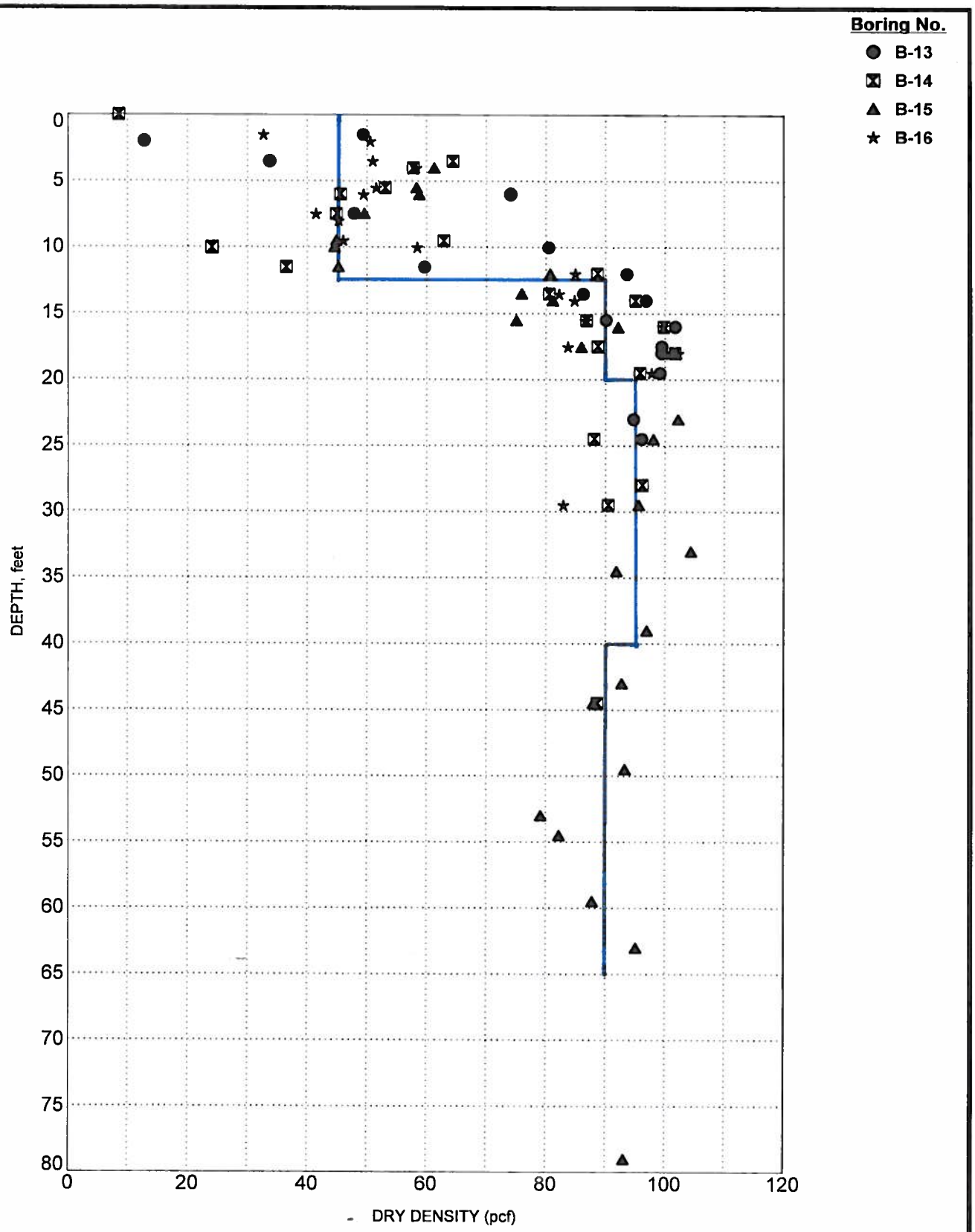
DRY DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-3A



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Work Product

ARD DEPTH VS DRY DENSITY 13-80-3705.GPJ US_LAB.GDT 3/24/14

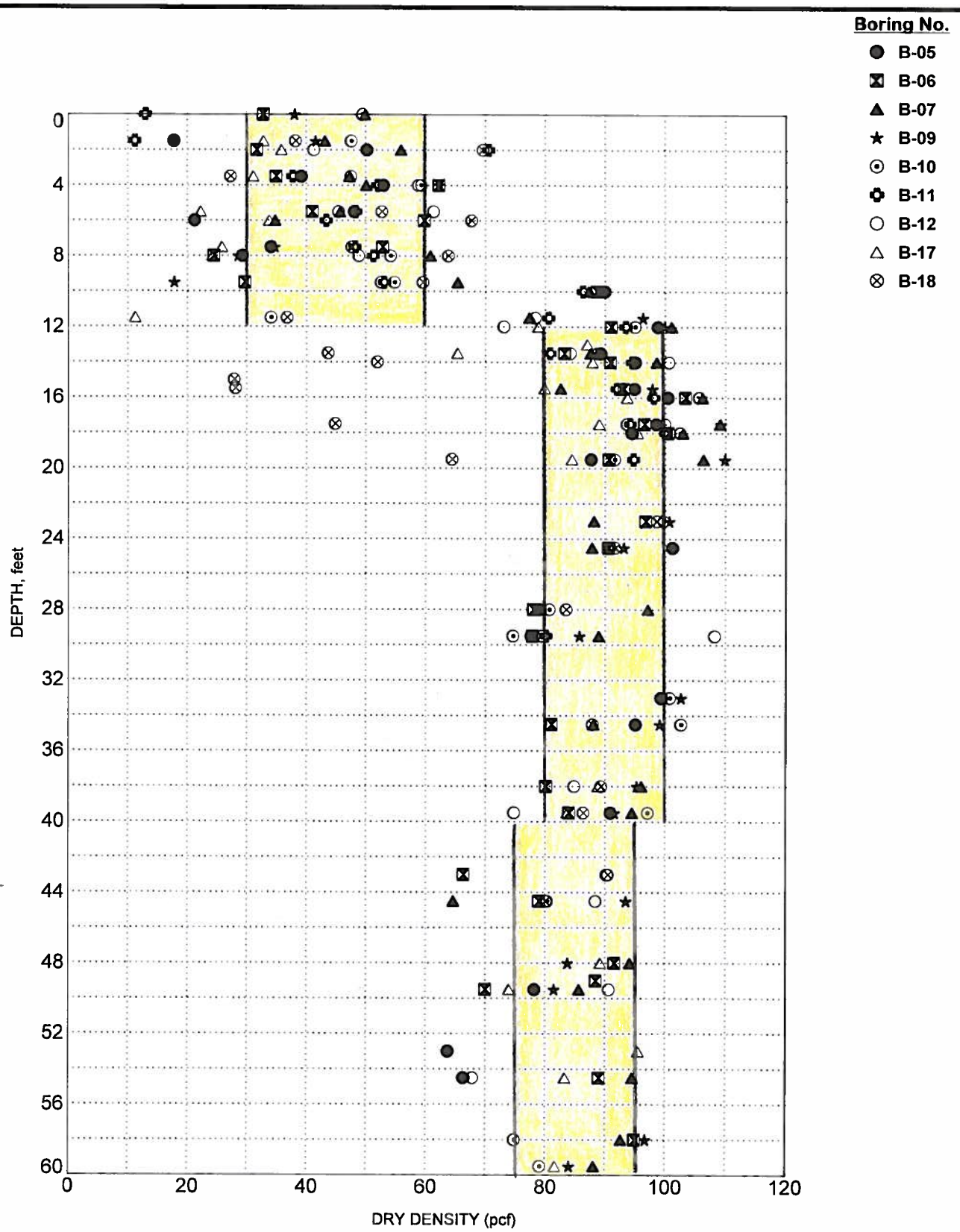
Ardaman & Associates, Inc.

DRY DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-3B



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Work Product

ARD DEPTH VS DRY DENSITY 13-80-3705.GPJ US LAB.GDT 3/24/14

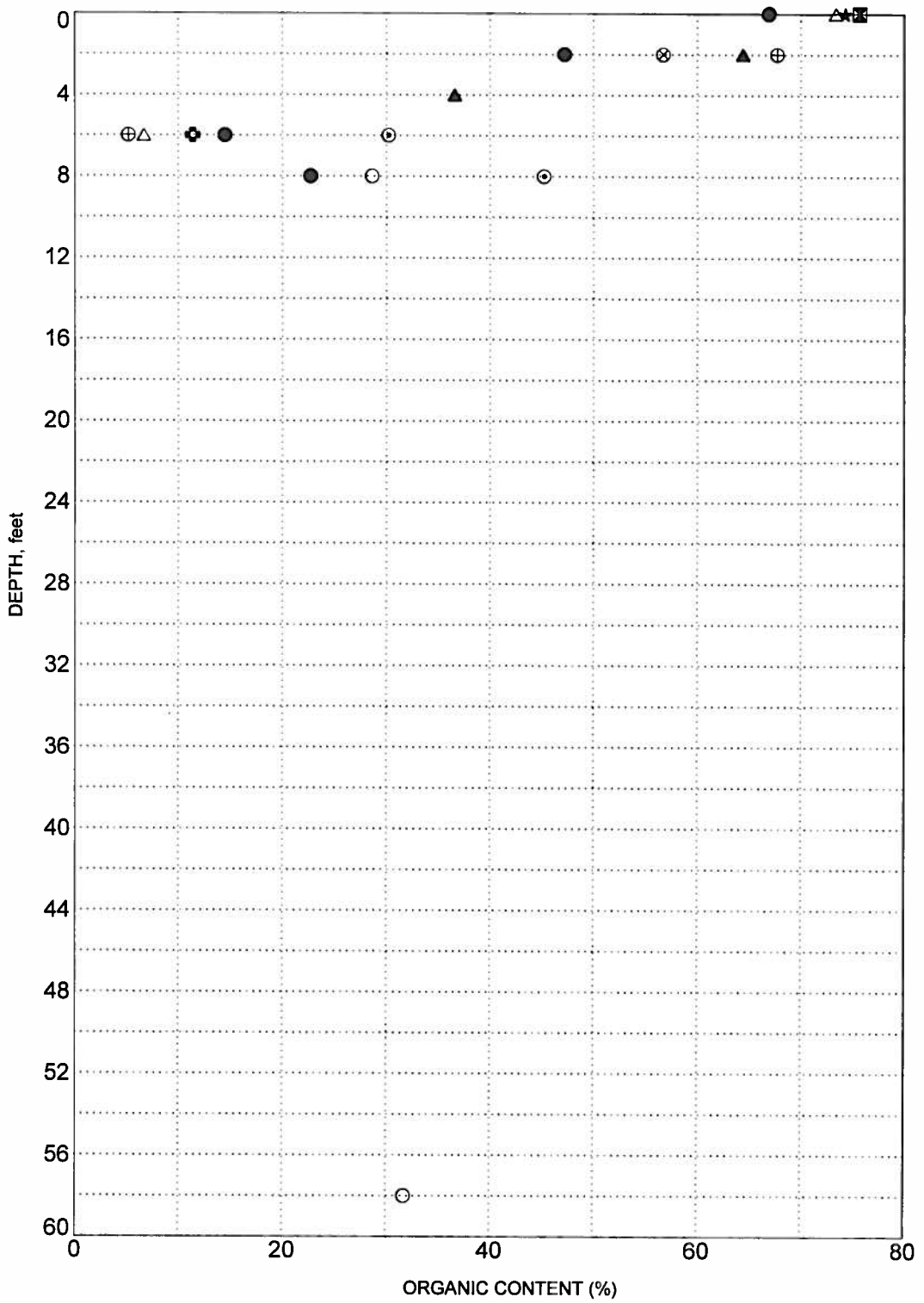
Ardaman & Associates, Inc.

DRY DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-3C



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ARD DEPTH VS ORGANIC CONTENT 13-80-3705.GPJ US_LAB.GDT 3/27/14

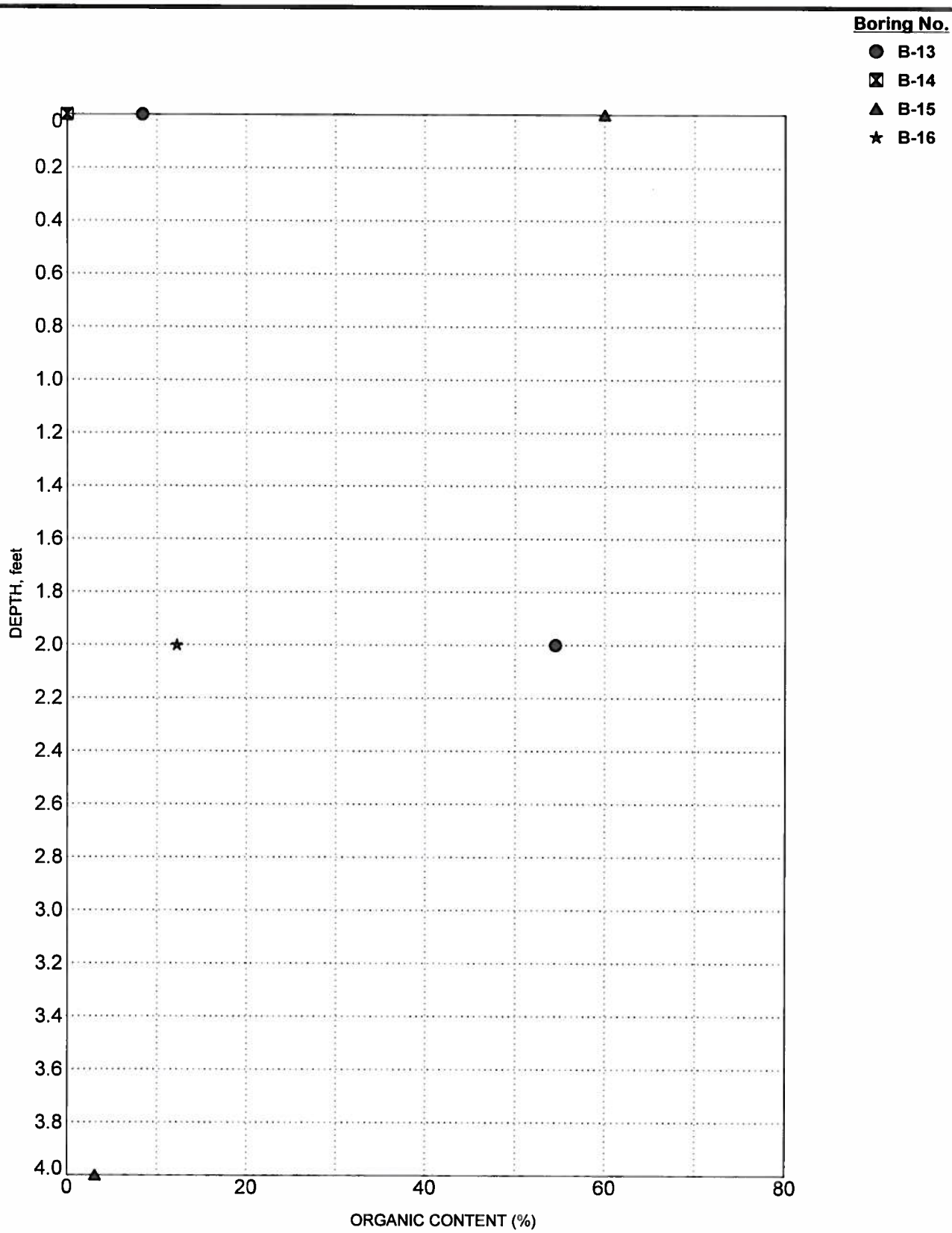
Ardaman & Associates, Inc.

ORGANIC CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-4A



Ardaman & Associates, Inc.

ORGANIC CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-4B

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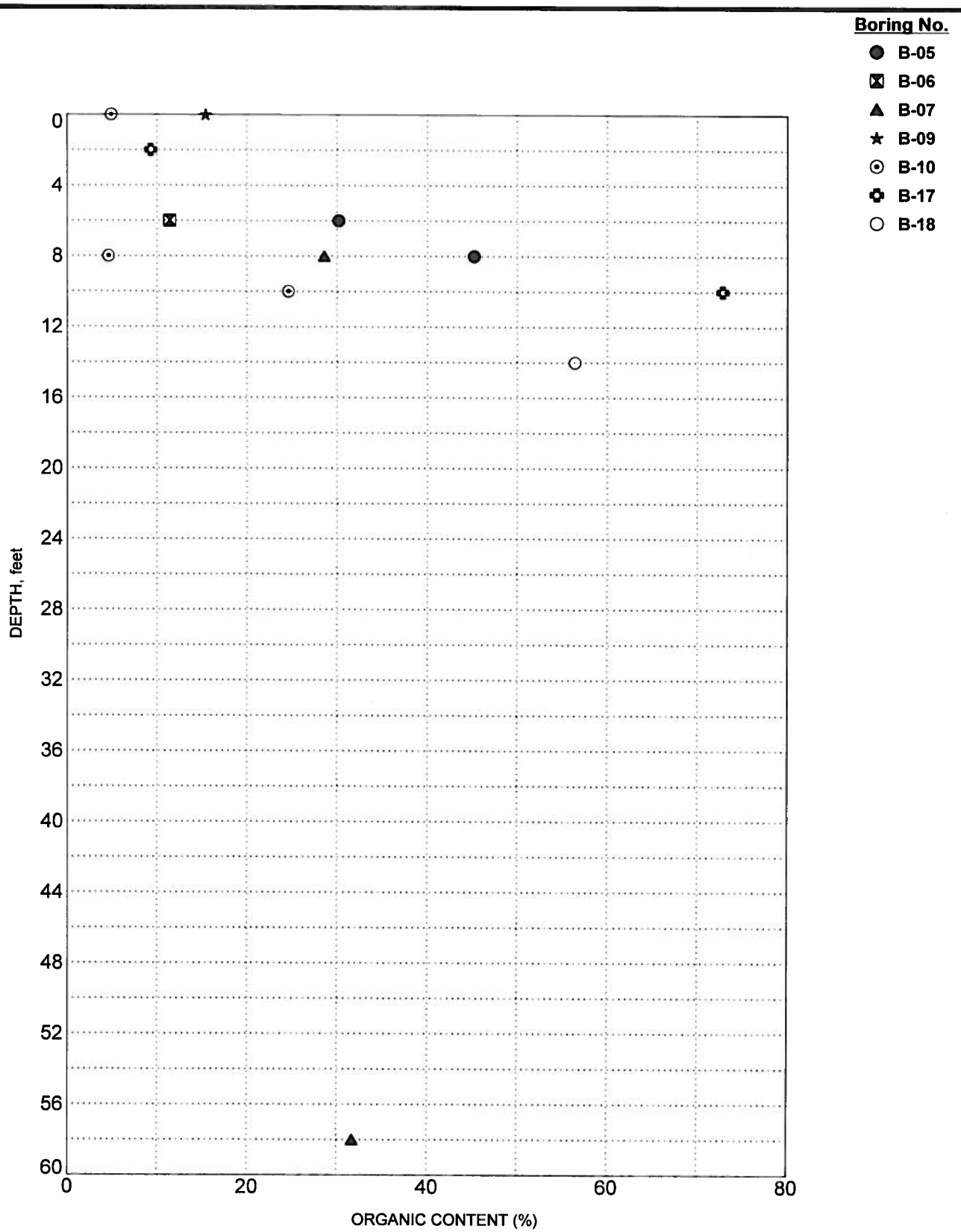

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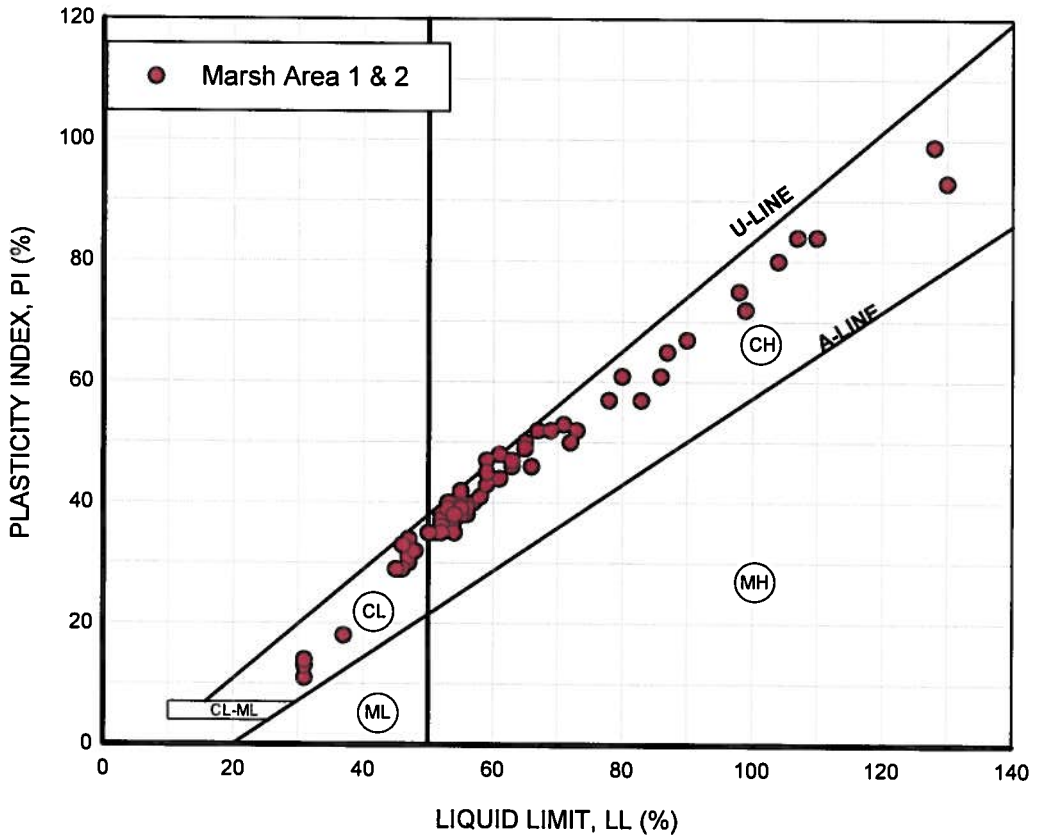
Ardaman & Associates, Inc.

ORGANIC CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-4C



PLASTICITY CHART

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Geotechnical, Environmental and Materials Consultants

COLE'S BAYOU MARSH CREATION

CPRA

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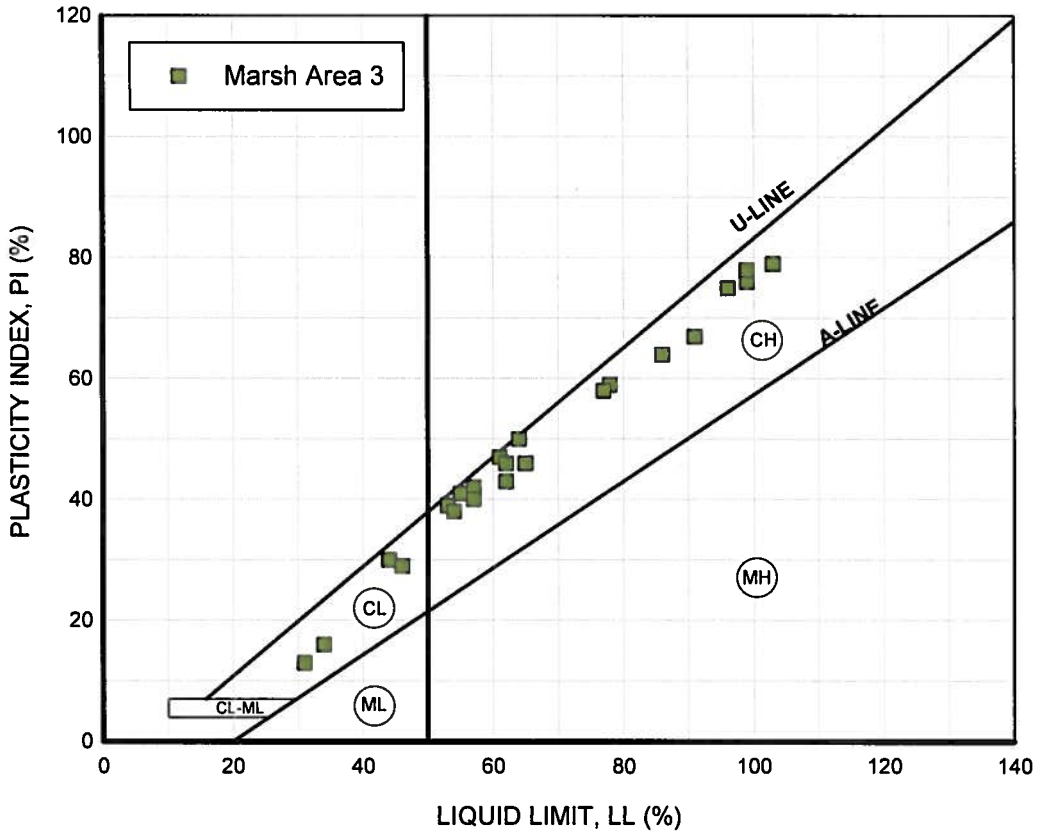
RJB	RER	12/19/13
FILE NO 13-80-3705	APPROVED BY MB	FIGURE B-5A



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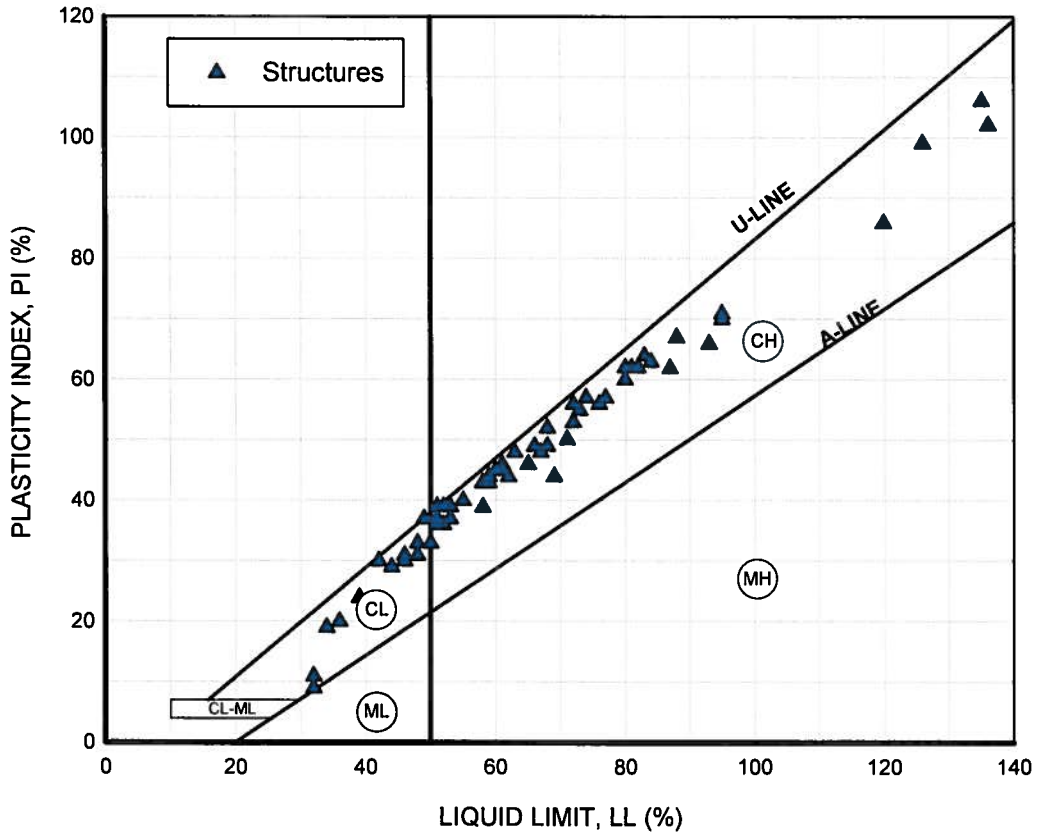
COLE'S BAYOU MARSH CREATION

CPRA

DRAWN BY	CHECKED BY	DATE
RJB	RER	12/19/13
FILE NO. 13-80-3705	APPROVED BY MB	FIGURE B-5B



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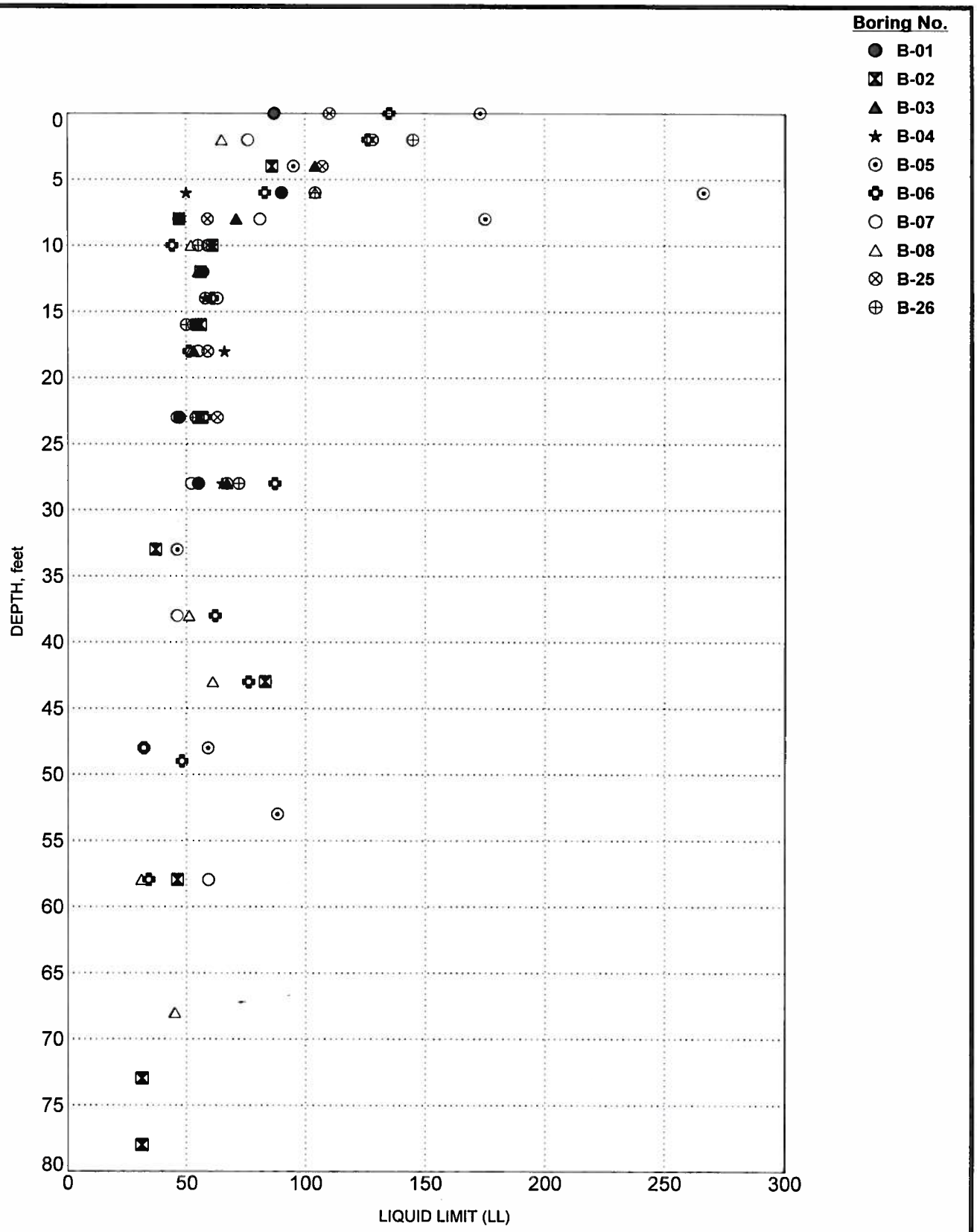
COLE'S BAYOU MARSH CREATION

CPRA

DRAWN BY: RJB	CHECKED BY: RER	DATE: 12-19-13
FILE NO: 13-80-3705	APPROVED BY: MB	FIGURE: B-5C



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ARD DEPTH VS LL 13-80-3705.GPJ US_LAB.GDT 3/24/14

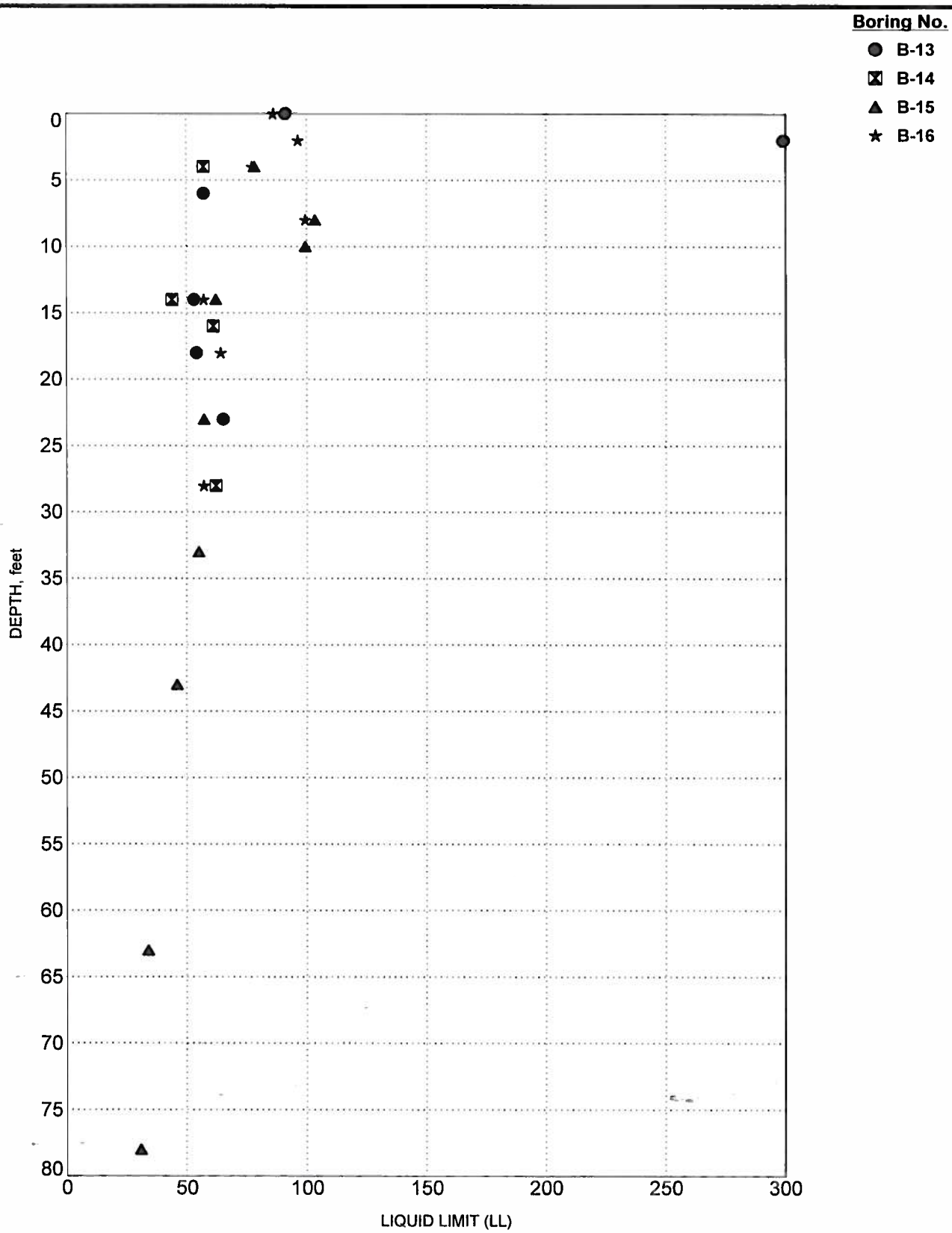
Ardaman & Associates, Inc.

LIQUID LIMIT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-6A



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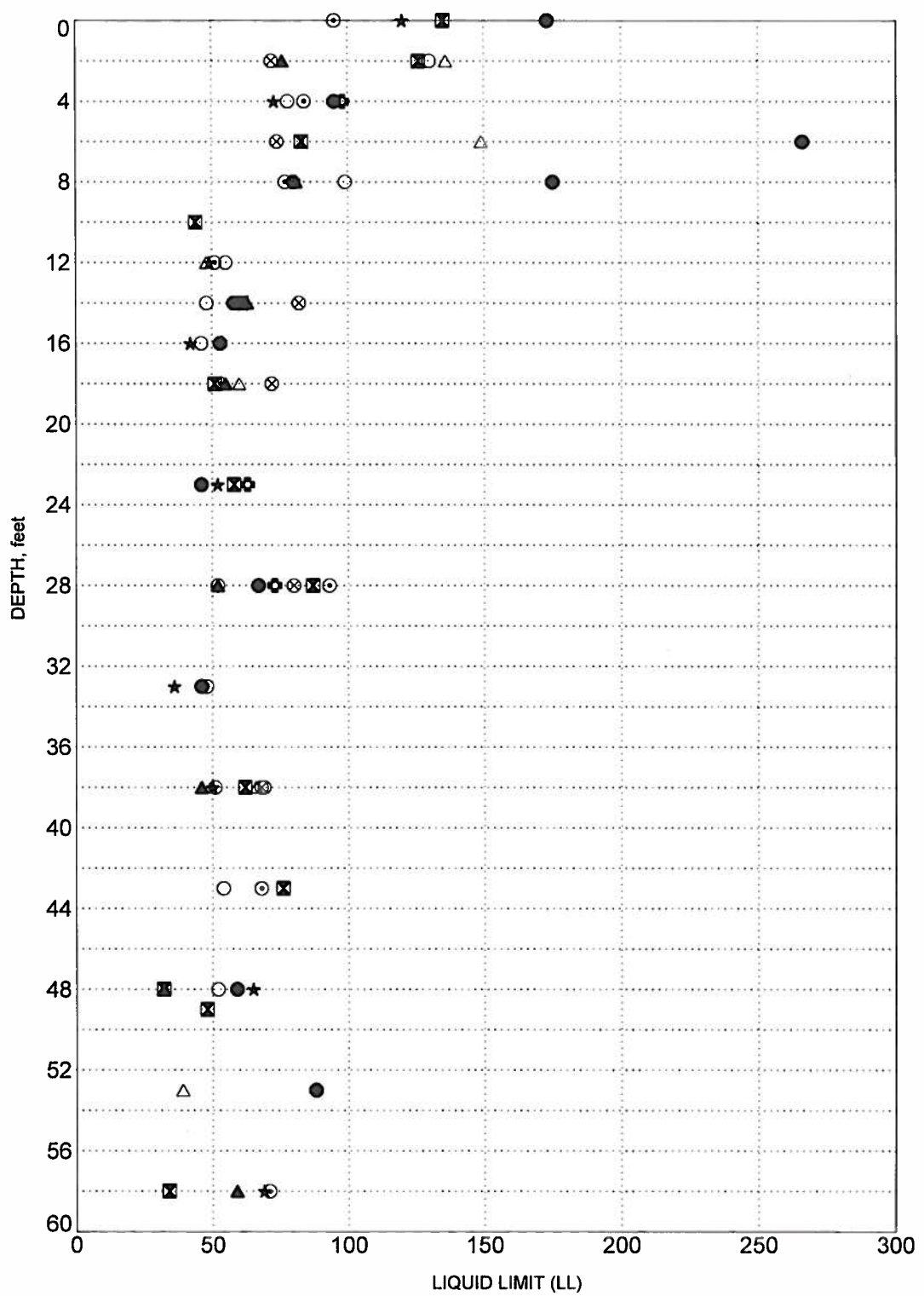
Ardaman & Associates, Inc.

LIQUID LIMIT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-6B



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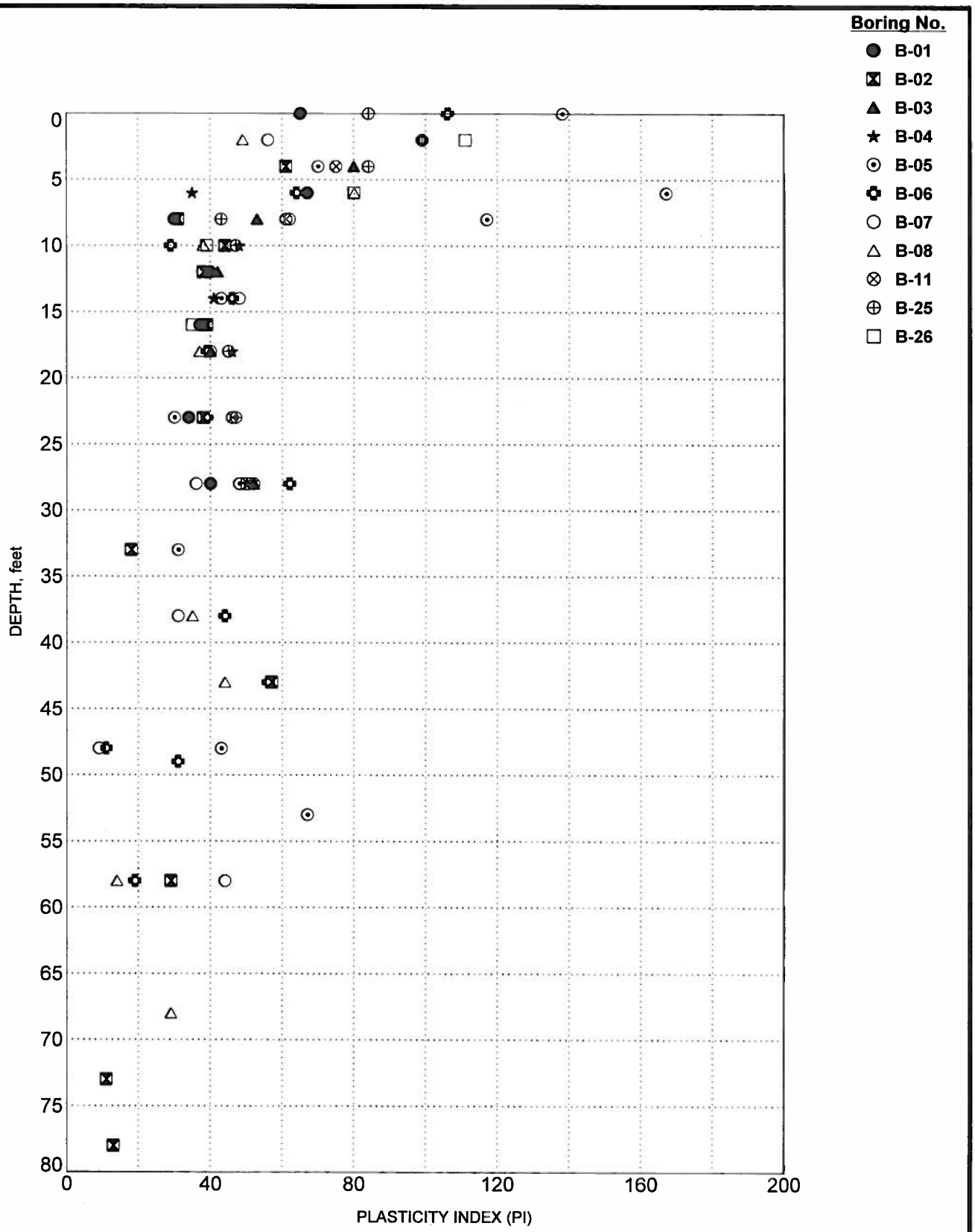
LIQUID LIMIT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-6C



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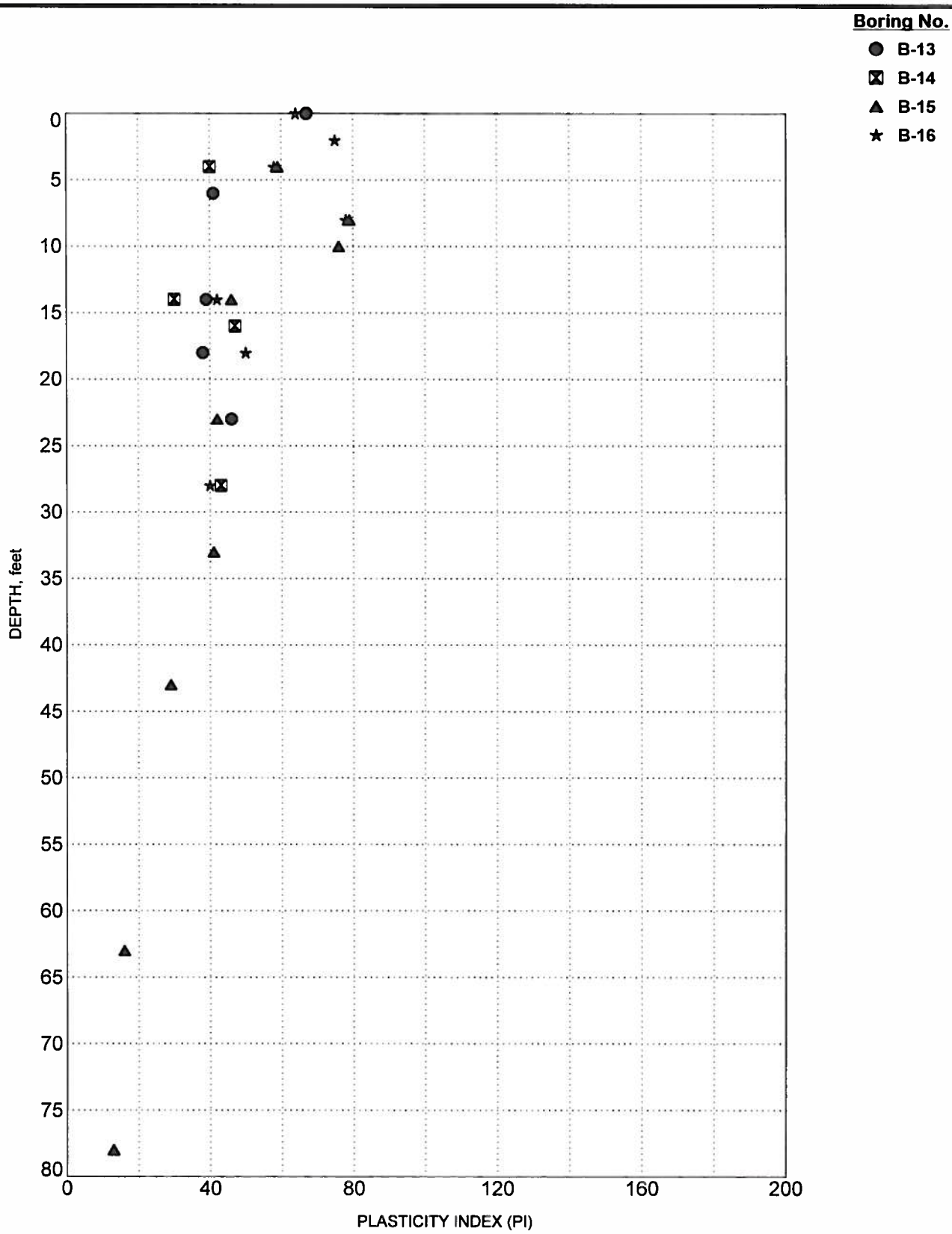
PLASTICITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-7A



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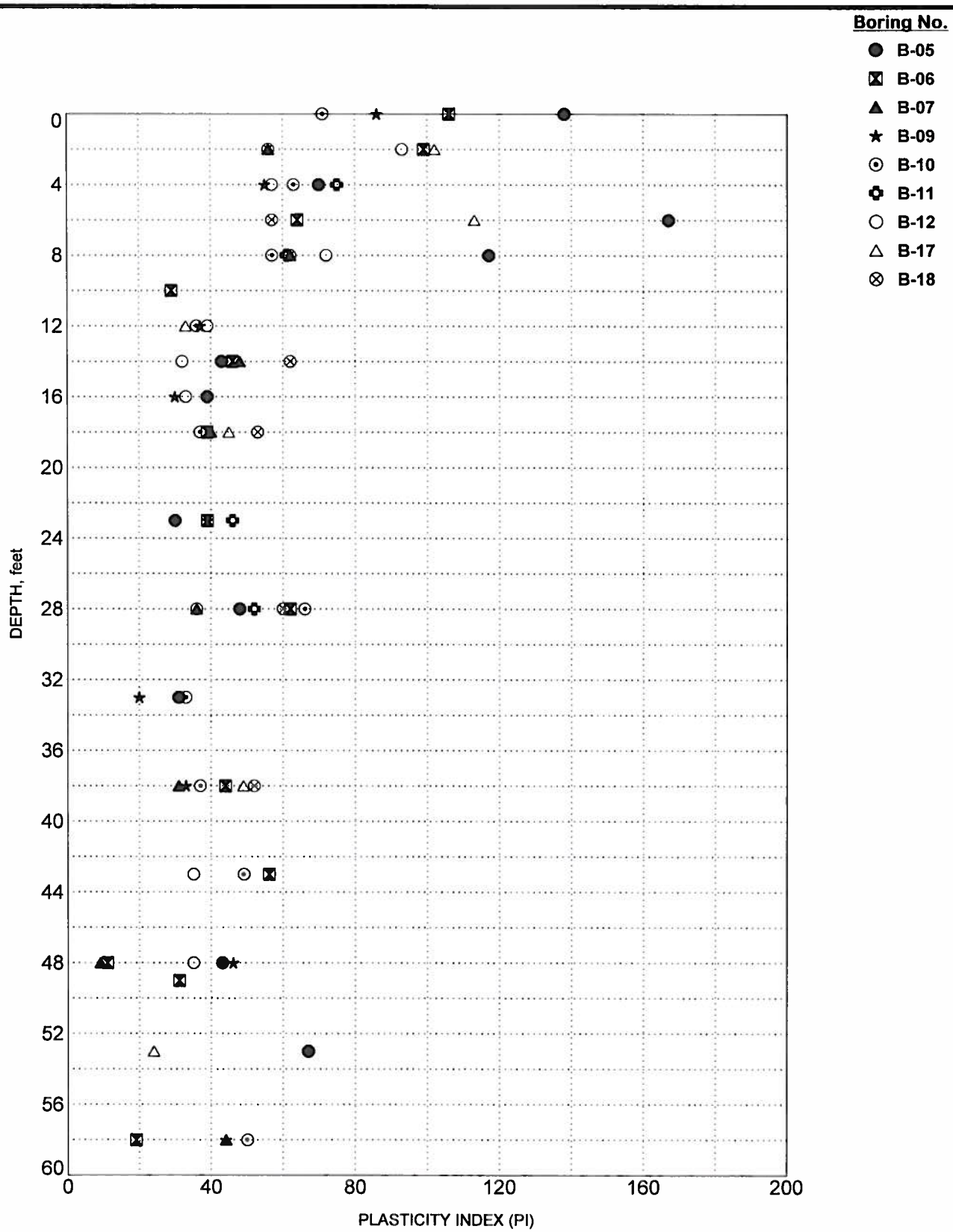
PLASTICITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-7B



Ardaman & Associates, Inc.

PLASTICITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-7C

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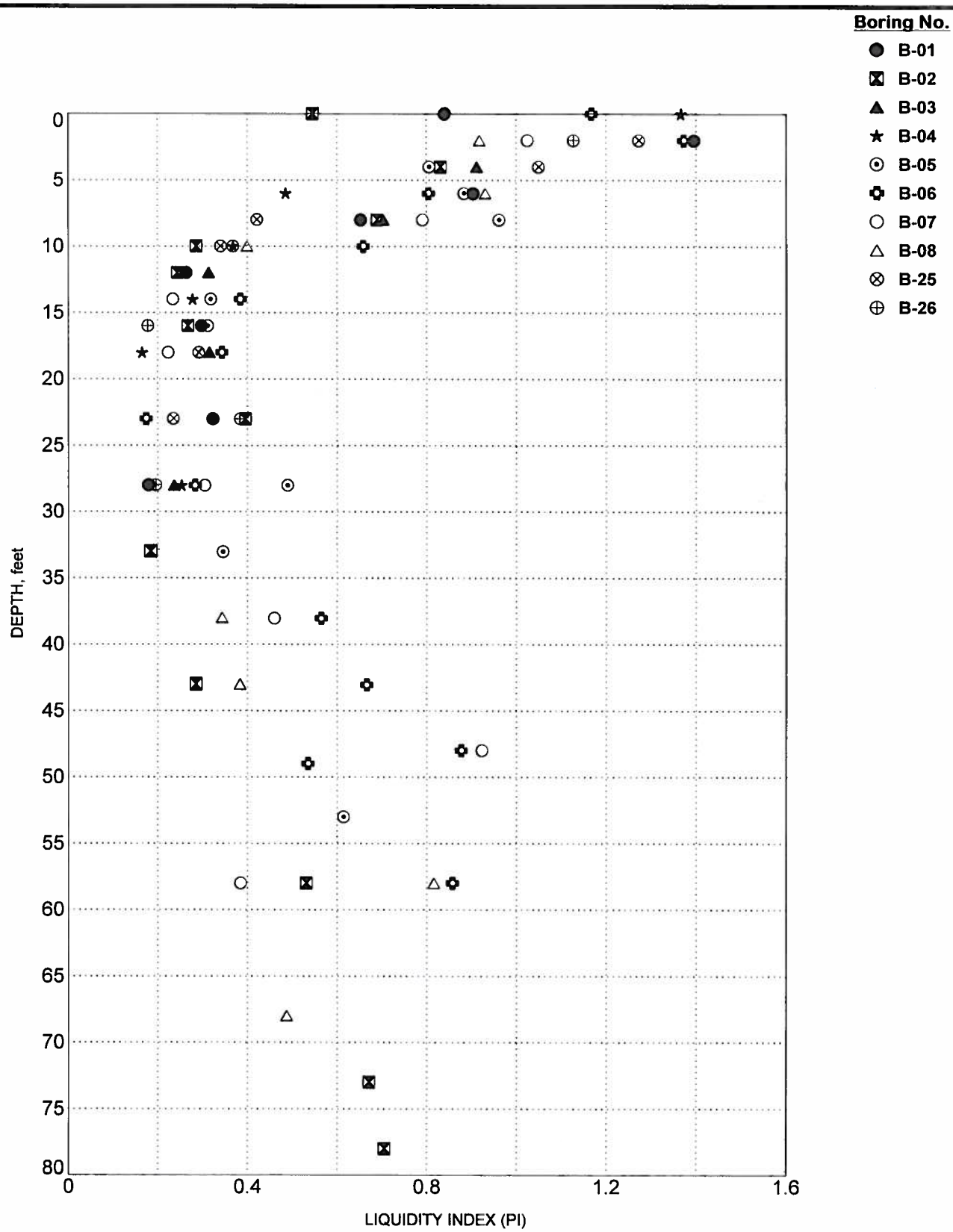

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ARD DEPTH VS LI 13-80-3705.GPJ US_LAB.GDT 3/24/14

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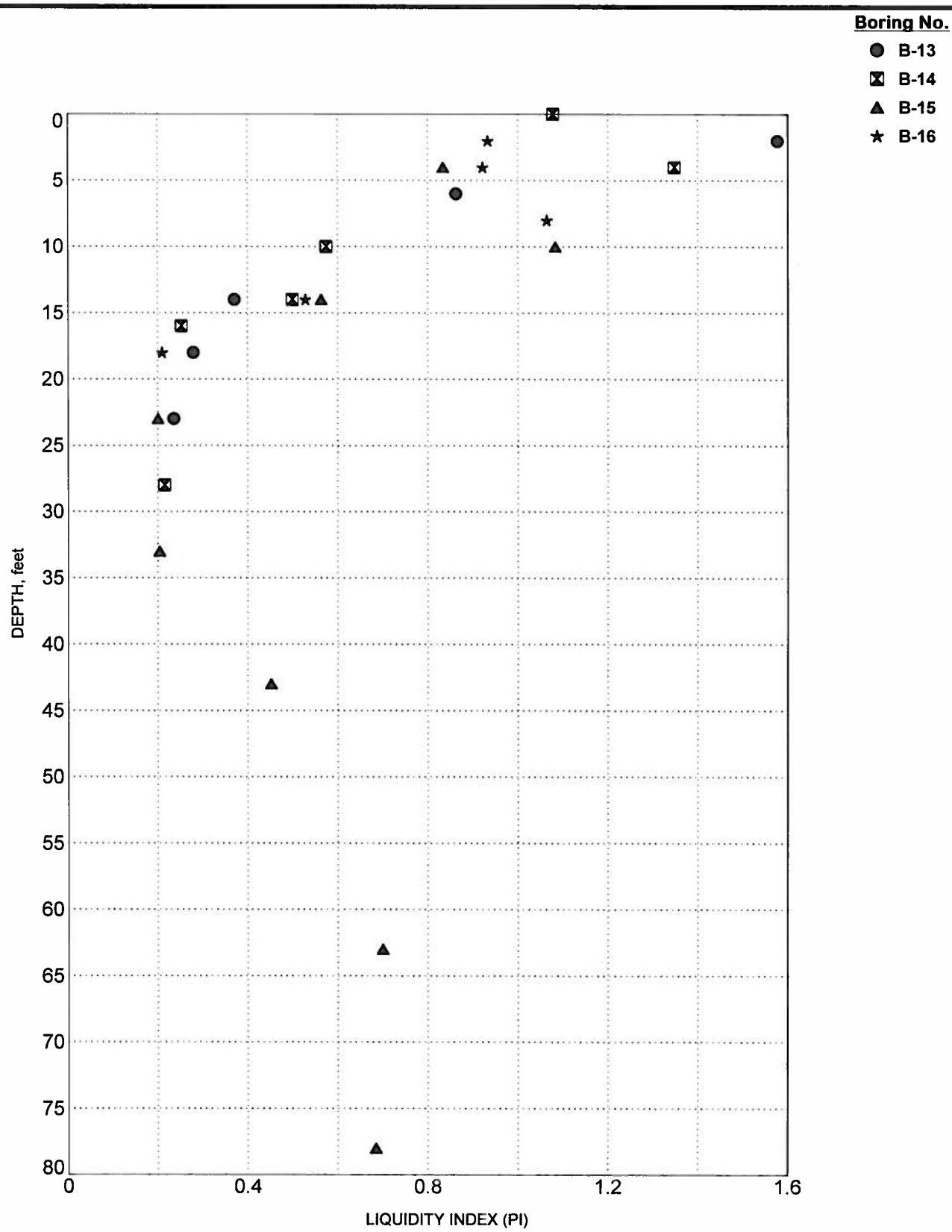
LIQUIDITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-8A



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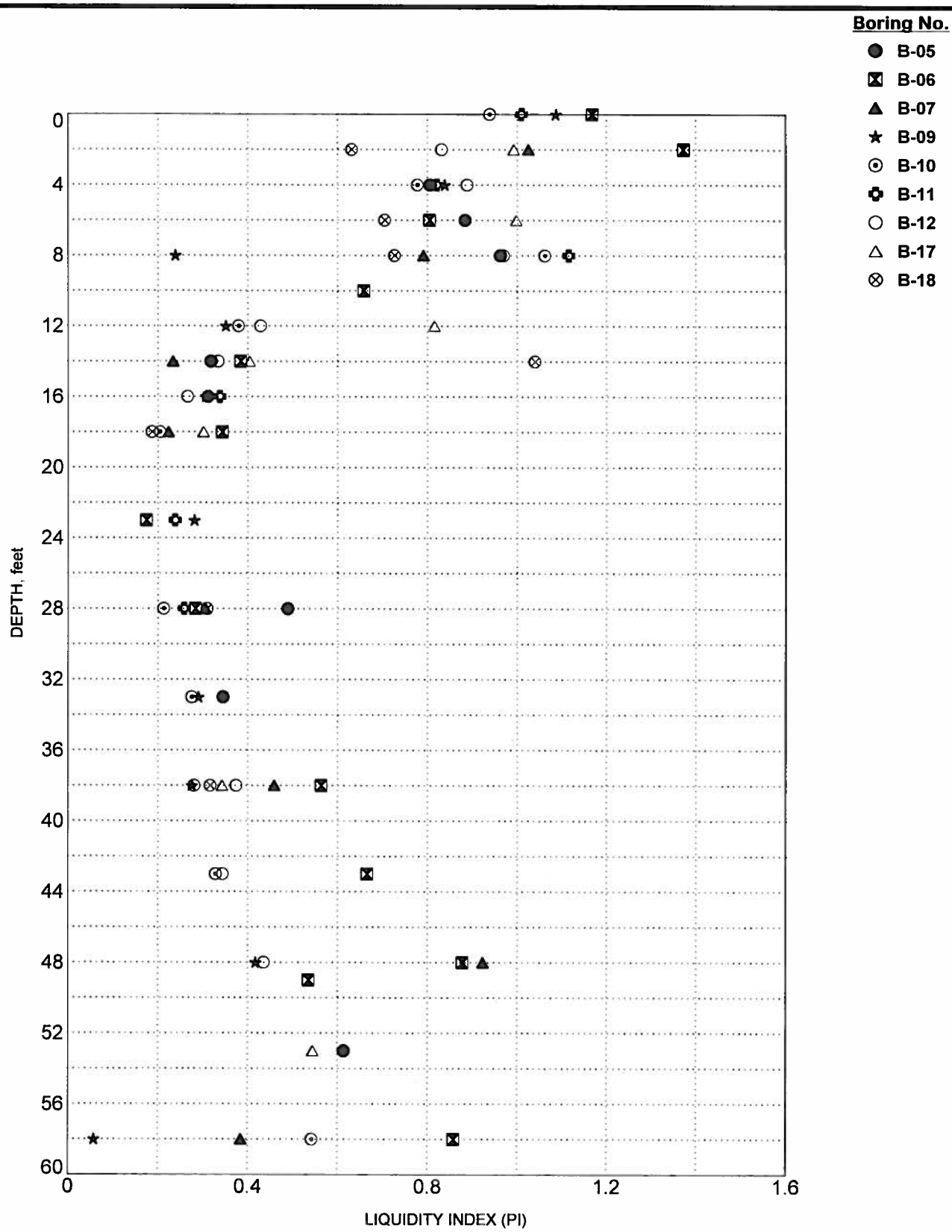
Ardaman & Associates, Inc.

LIQUIDITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-8B



Ardaman & Associates, Inc.

LIQUIDITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-8C

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ARD DEPTH VS LI 13-80-3705.GPJ US LAB.GDT 3/24/14

Figure B-9
Hydrometer Particle Size Analyses Results

Boring ID	Sample Depth	% Gravel	% Sand	% Silt	% Clay
B-01	23-25	0	3	72	25
B-02	33-35	0.4	17	63	19
B-02	38-40	0	0	39	61
B-02	48-50	0	35	54	11
B-02	73-75	0	6	76	18
B-02	78-80	0	11	57	32
B-04	23-25	0	21	61	18
B-05	23-25	0	21	44	35
B-05	33-35	0	5	62	33
B-05	48-50	0	1	48	51
B-06	2-4	0	3	25	72
B-06	6-8 BL 2	0	1	39	60
B-06	23.5-25	0	3	60	37
B-06	28-30	0	0	5	95
B-06	48-50	0	3	76	21
B-06	58-60	0	1	74	25
B-07	48-50	0	5	63	32
B-07	58-60	0	1	77	22
B-08	58-60	0	3	74	23
B-09	33-35	0	14	51	35
B-10	33-35	0	15	44	41
B-10	43-45	0	0	31	69
B-12	43-45	0	0	46	53
B-20	16-18	0	2	67	32
B-23	18-20	0	3	65	32
B-25	23-25	0	11	60	29
B-26	23-25	0	9	76	15

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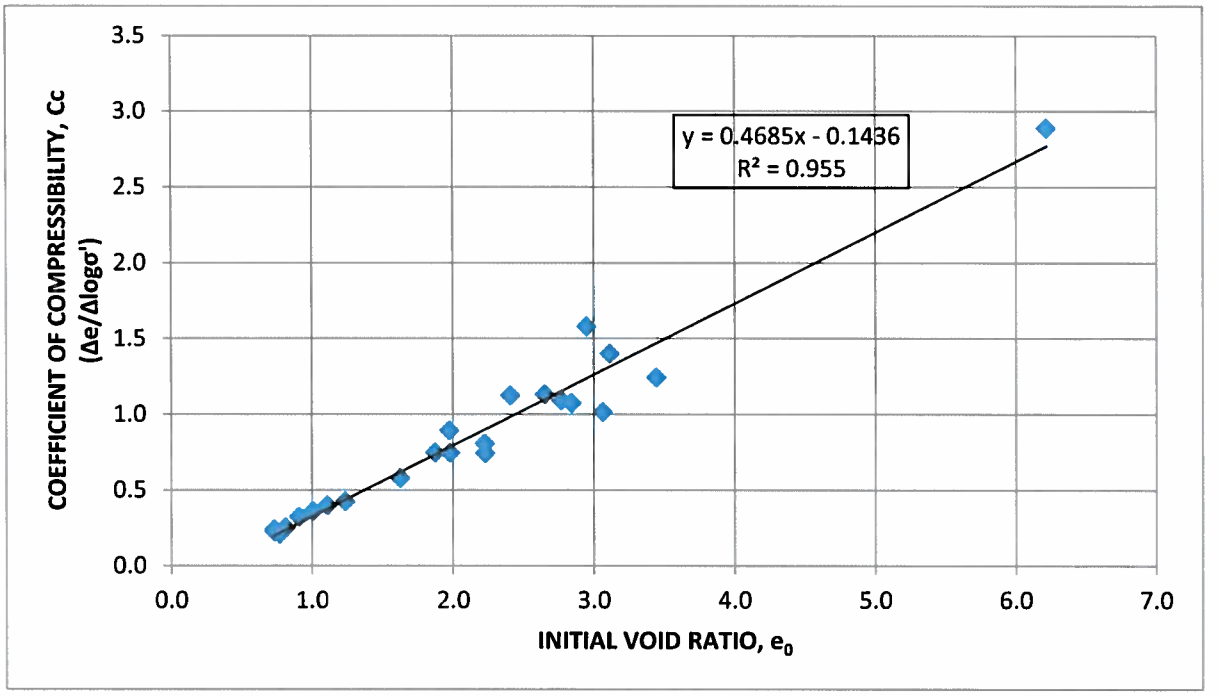

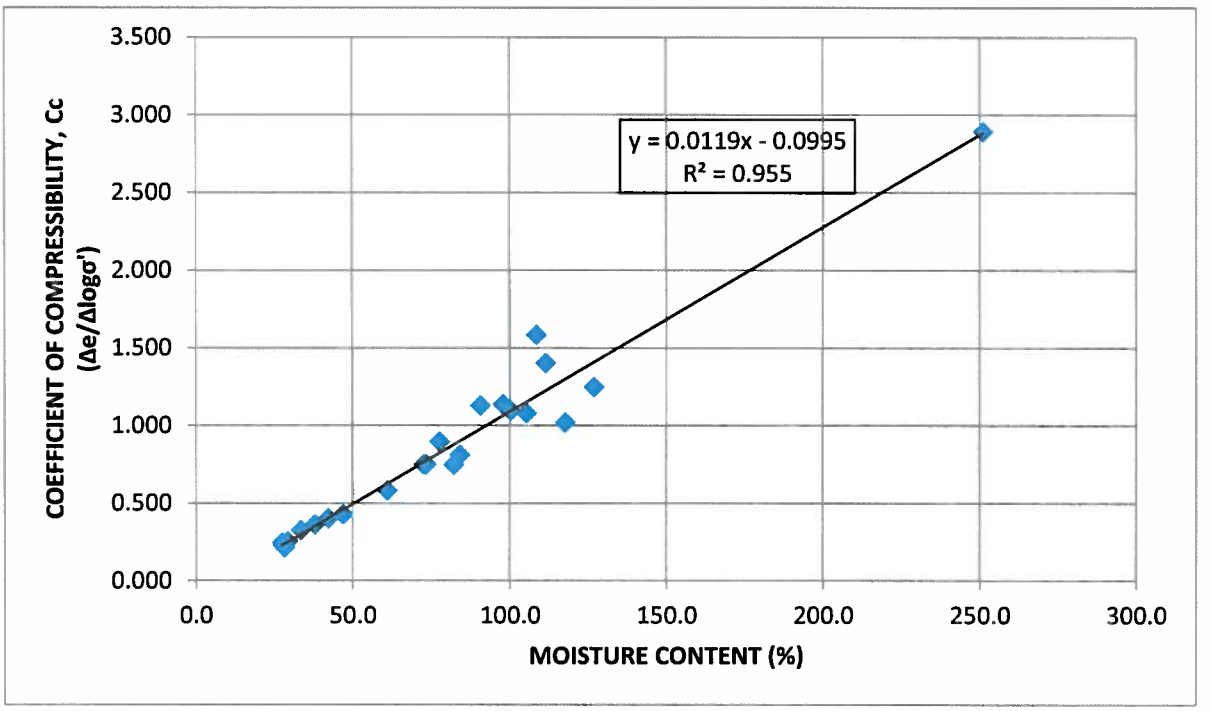

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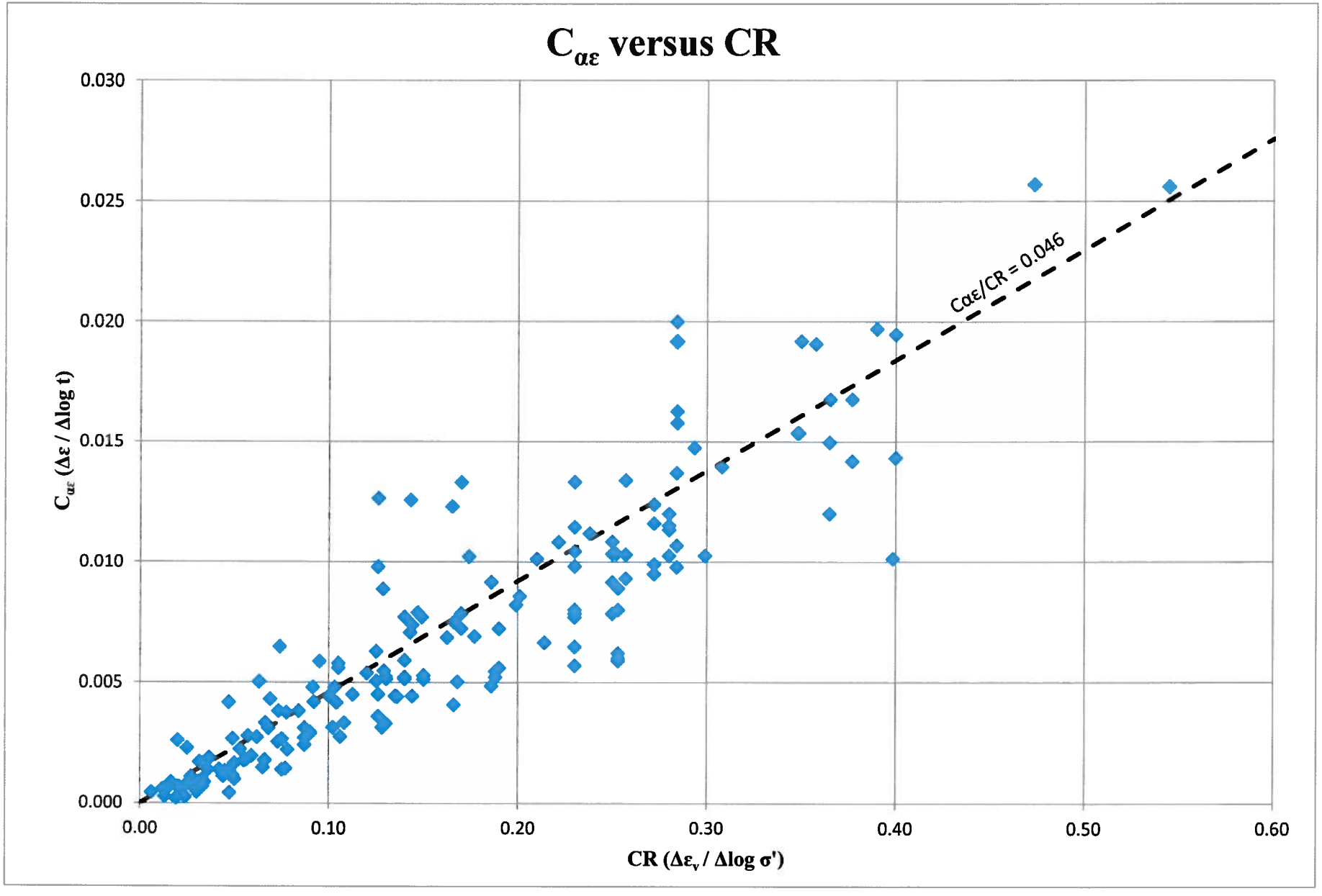
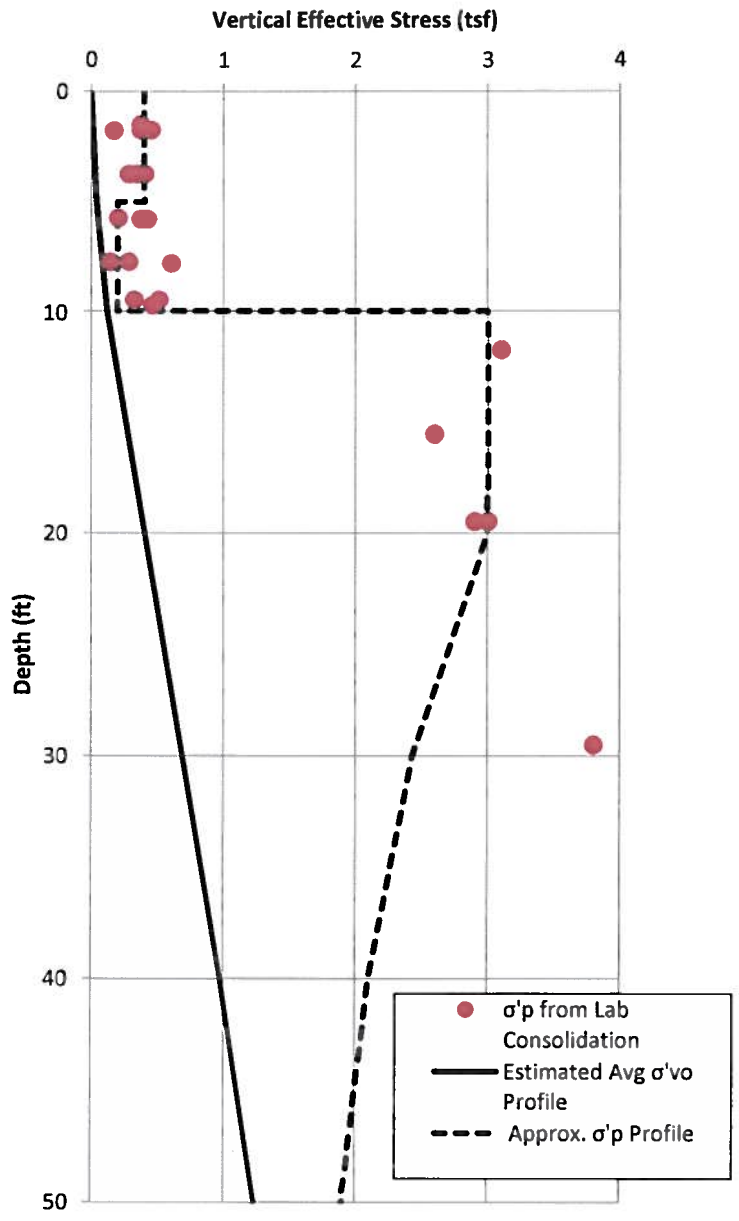


Figure B-11

Ardaman Associates, Inc.
316 Highlandia Drive
Baton Rouge, LA 70810



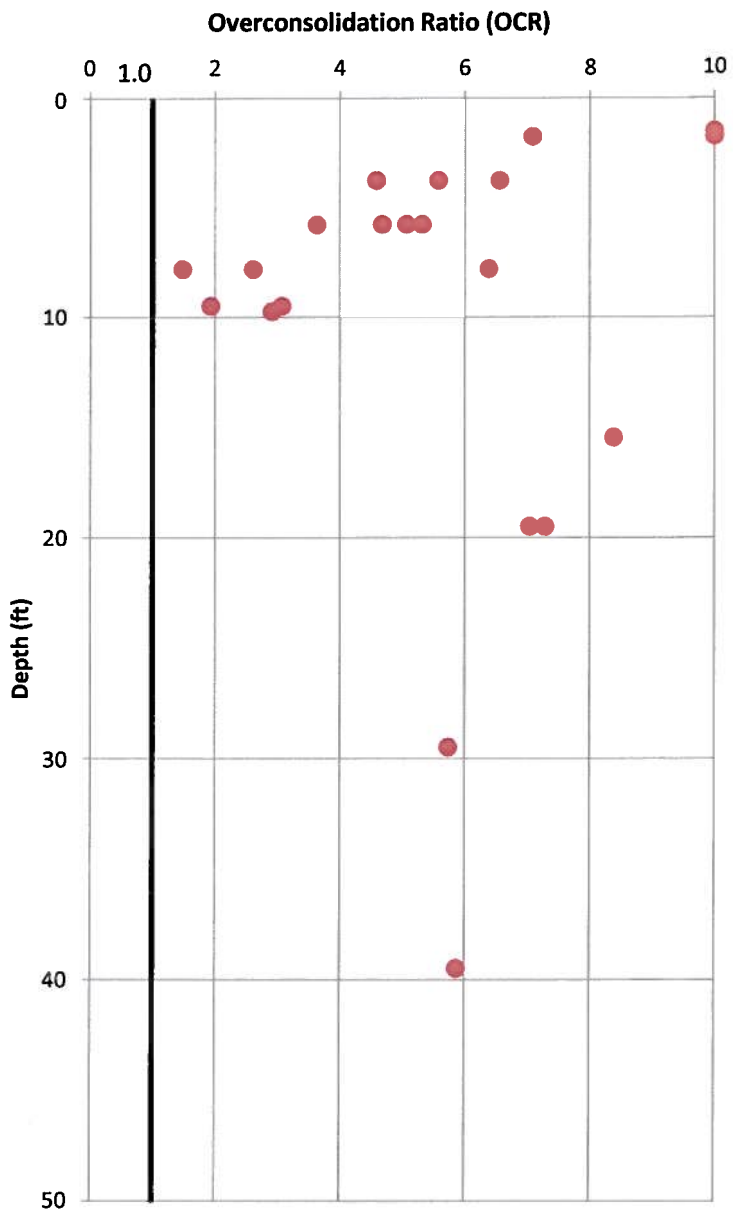


Figure: B-12

Client: CPRA
Project: Cole's Bayou
AAI Project No. 13-80-3705

STRESS HISTORY PROFILES

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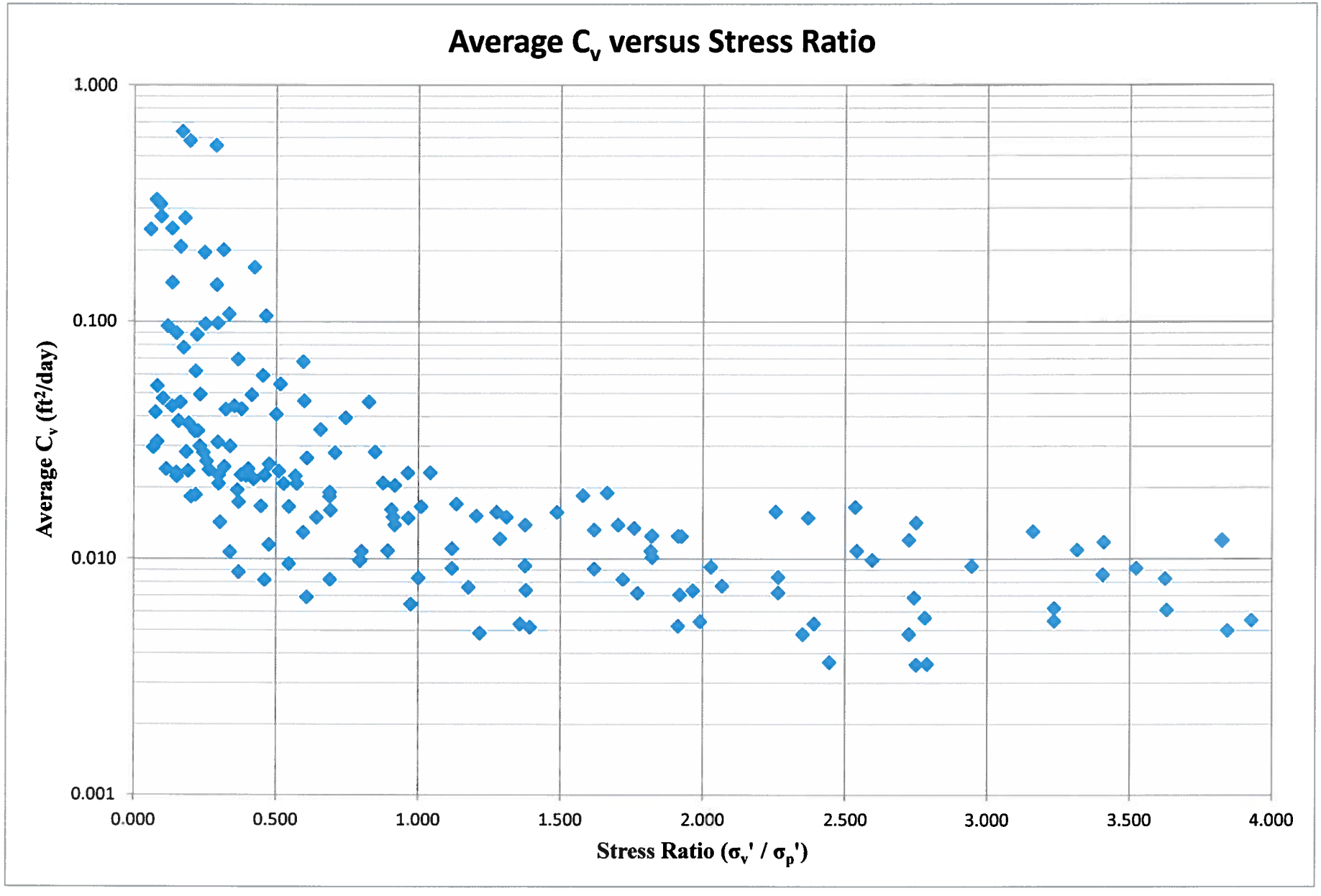
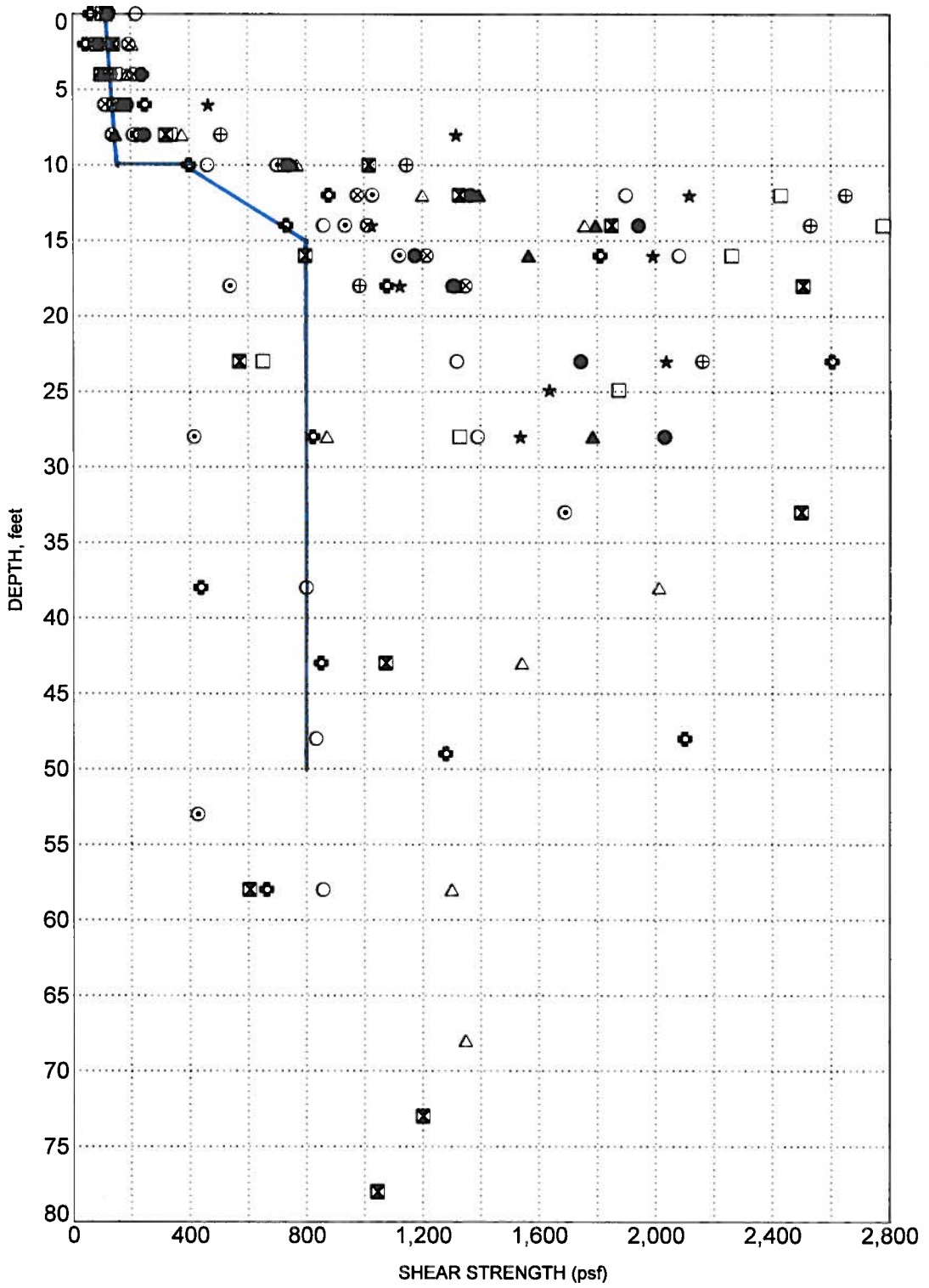


Ardaman & Associates, Inc.

Figure B-13

ArdamanAssociates, Inc.
316 Highlandia Drive
Baton Rouge, LA 70810



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ARD DEPTH VS SHEAR STRENGTH 13-80-3705.GPJ US LAB.GDT 3/28/14

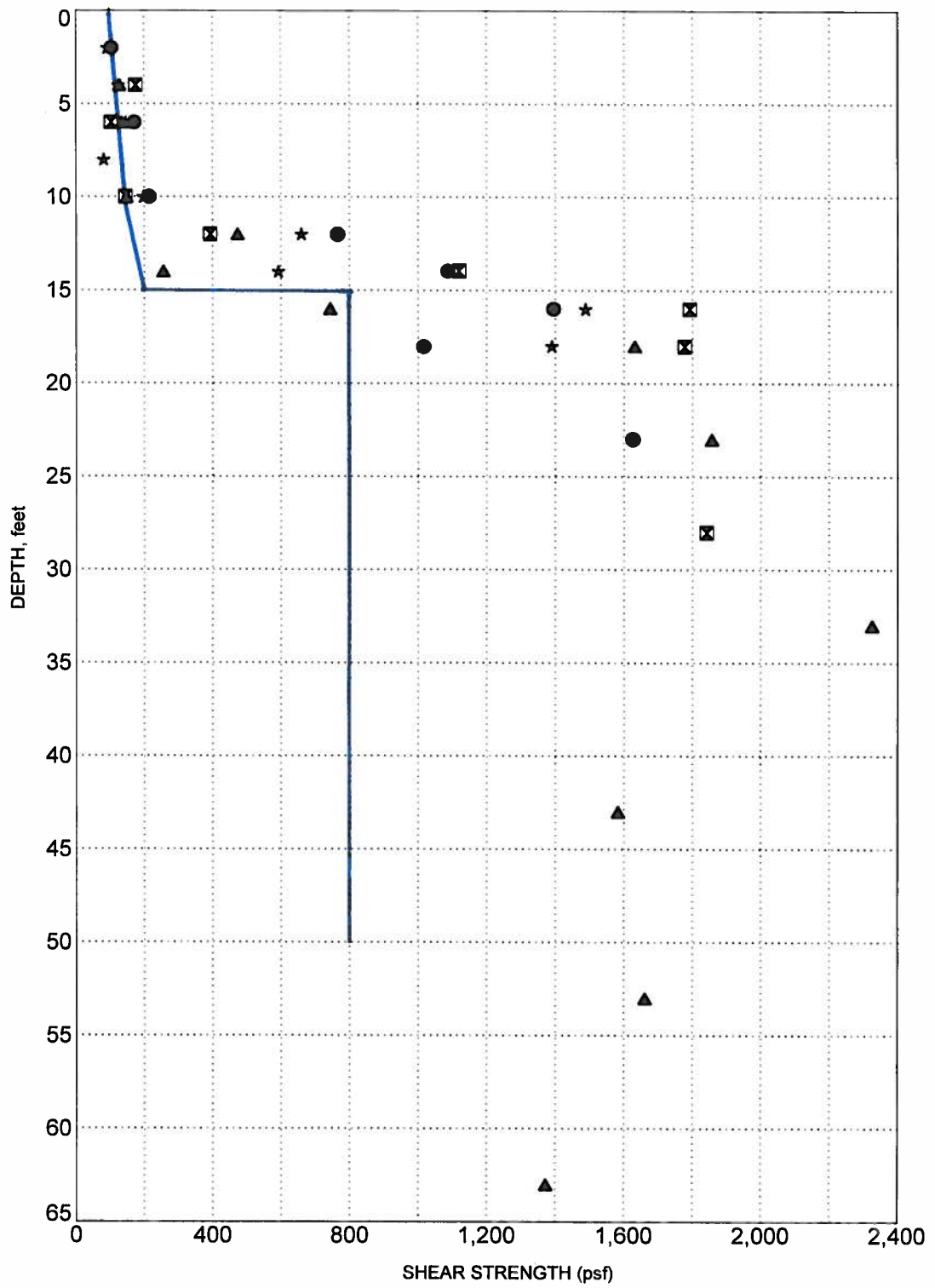
Ardaman & Associates, Inc.

UUC SHEAR STRENGTH vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-14A



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ARD DEPTH VS SHEAR STRENGTH 13-80-3705.GPJ US LAB.GDT 3/28/14

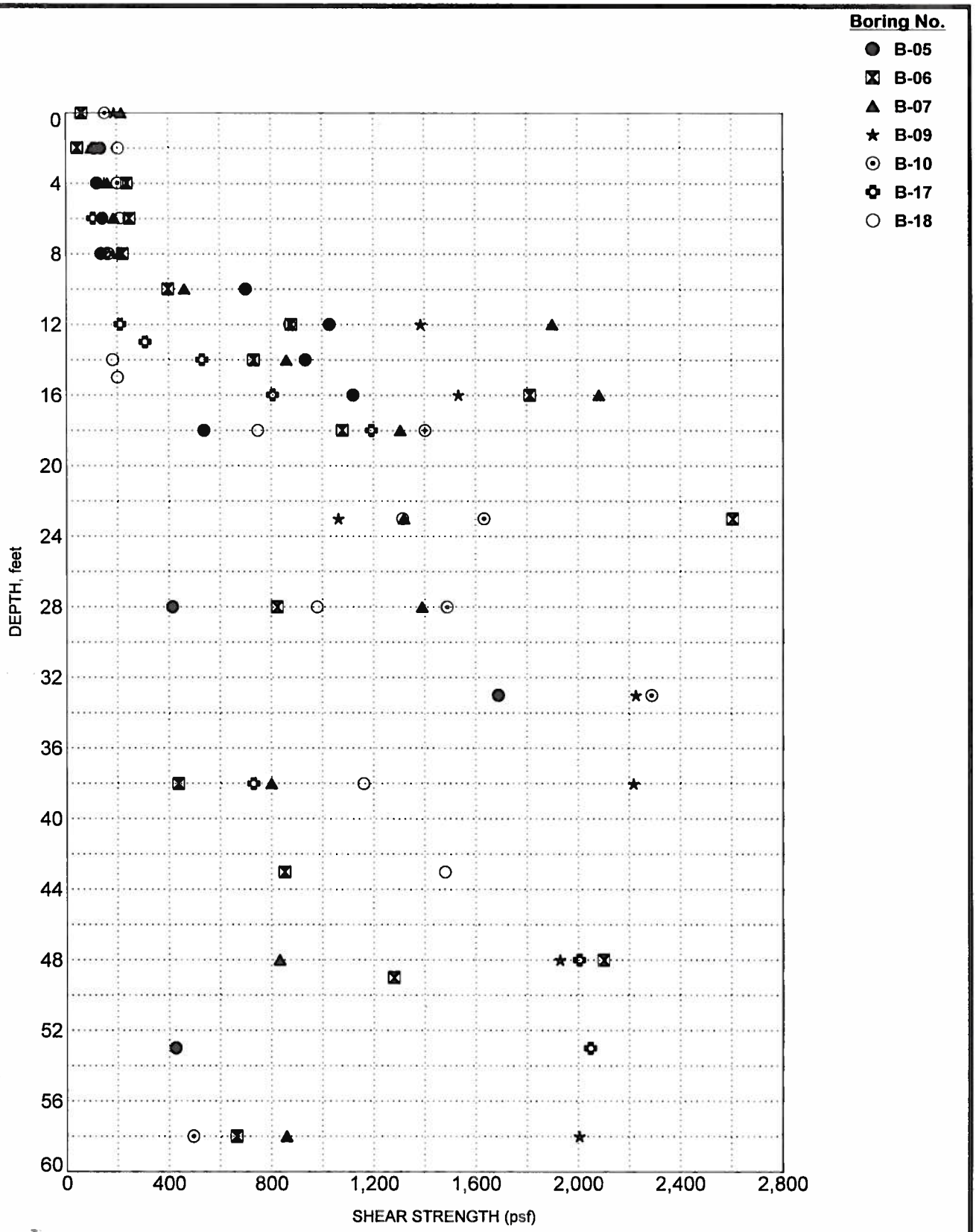
Ardaman & Associates, Inc.

UUC SHEAR STRENGTH vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-14B



Ardaman & Associates, Inc.

UUC SHEAR STRENGTH vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-14C

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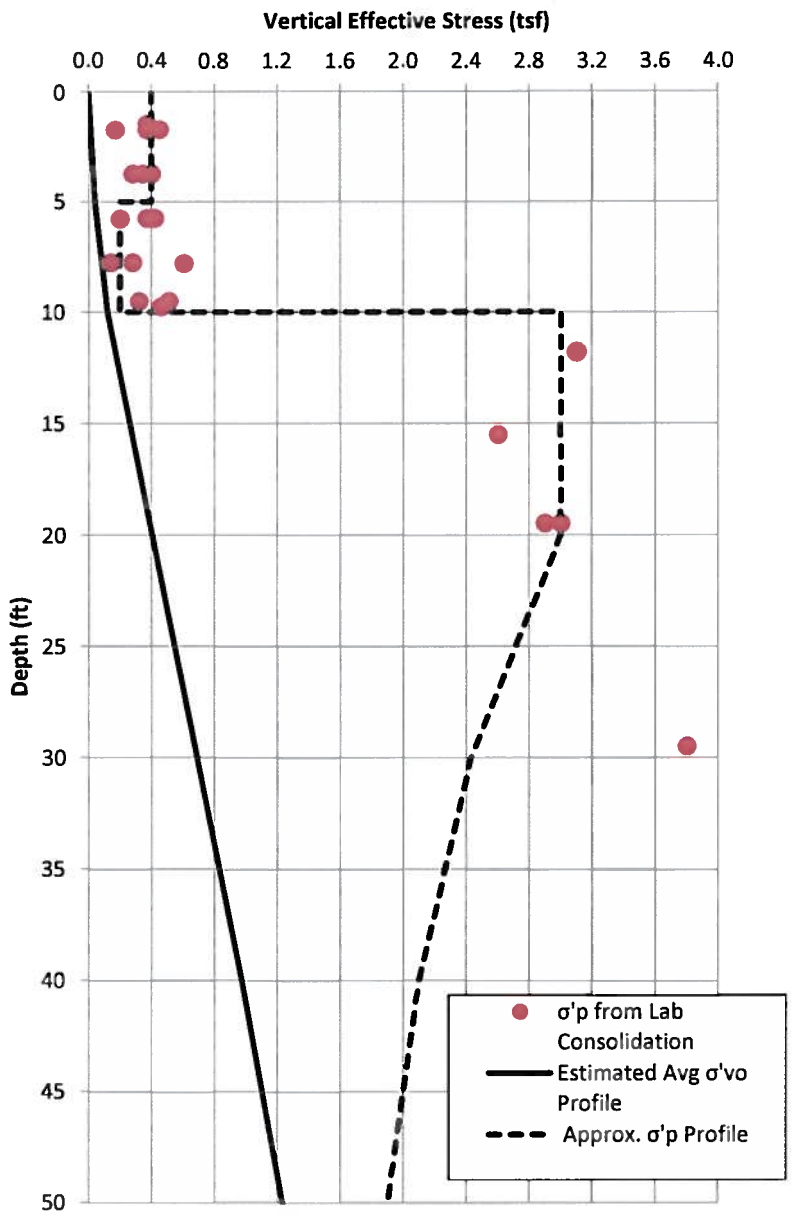
ARD DEPTH VS SHEAR STRENGTH 13-80-3705.GPJ US LAB.GDT 3/24/14



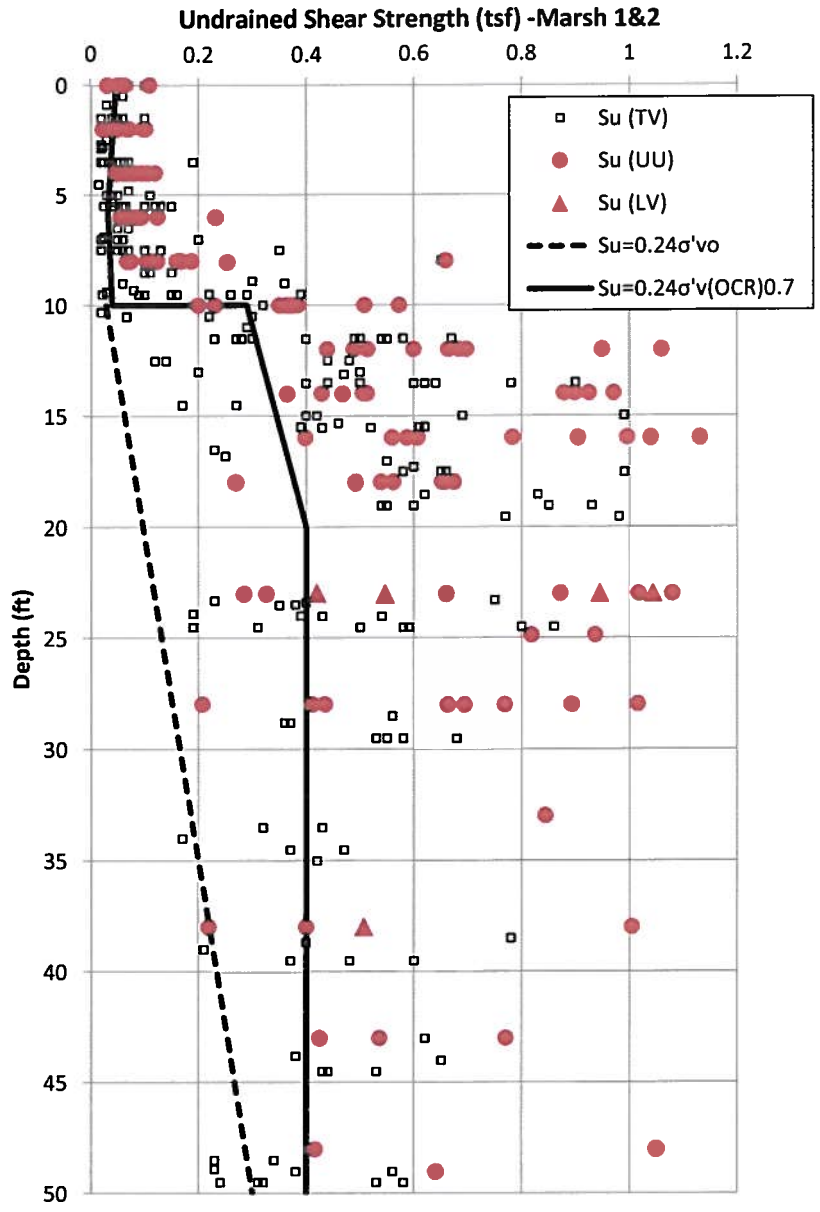


Figure: B-15A

Client: CPRA
Project: Cole's Bayou
AAI Project No. 13-80-3705

VERTICAL EFFECTIVE STRESS AND UNDRAINED SHEAR STRENGTH VS. DEPTH

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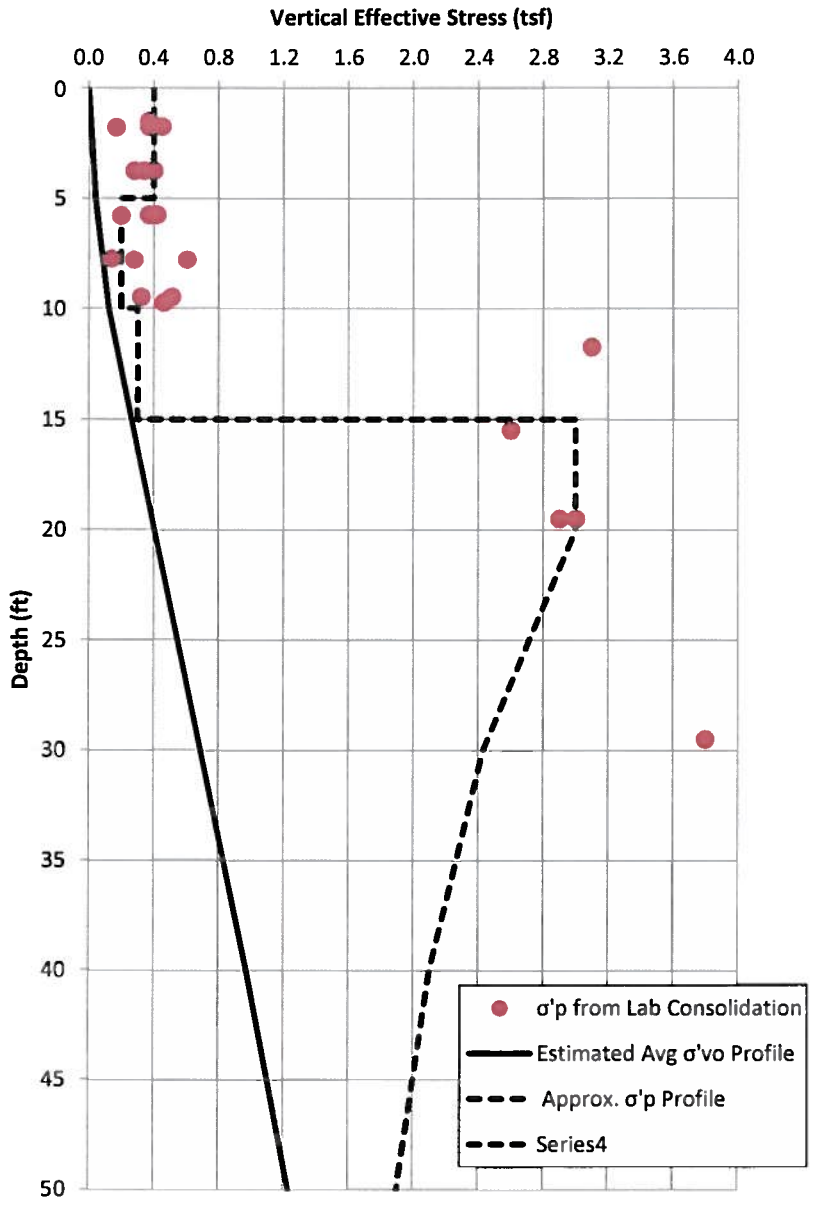

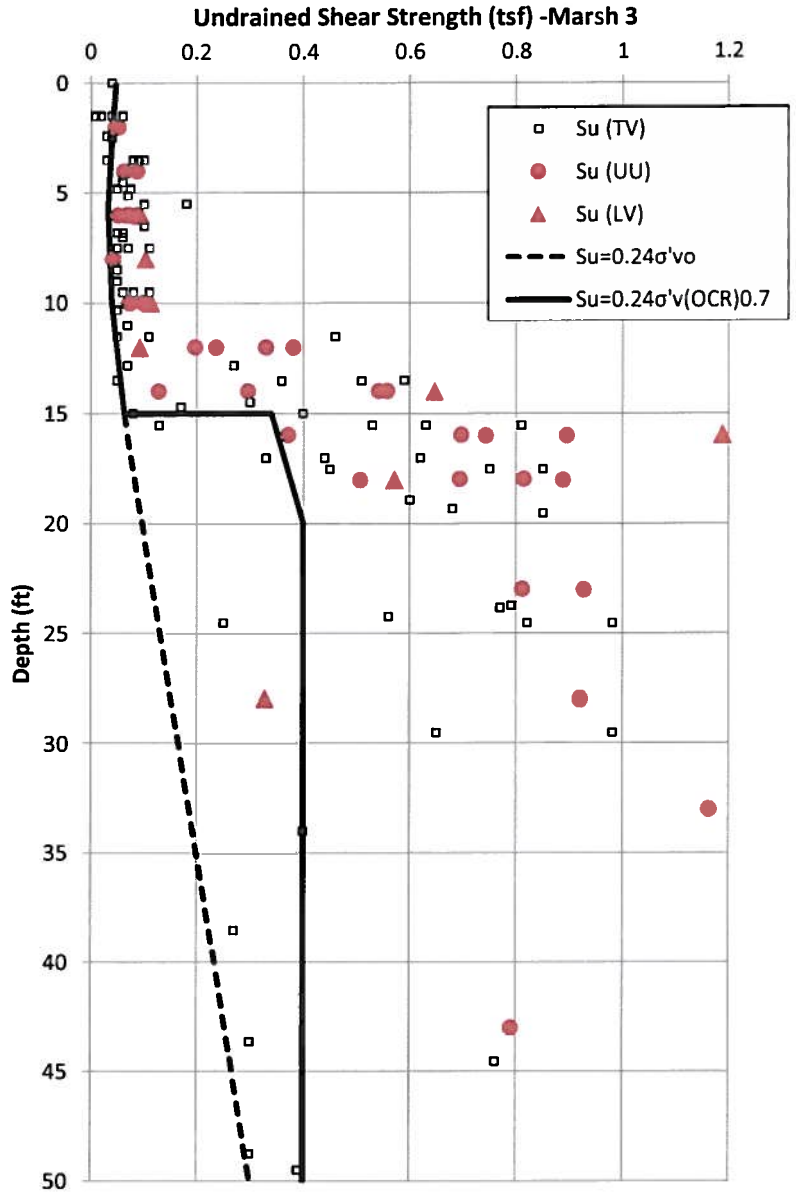

Figure: B-15B

Client: CPRA
Project: Cole's Bayou
AAI Project No. 13-80-3705

VERTICAL EFFECTIVE STRESS AND UNDRAINED SHEAR STRENGTH VS. DEPTH

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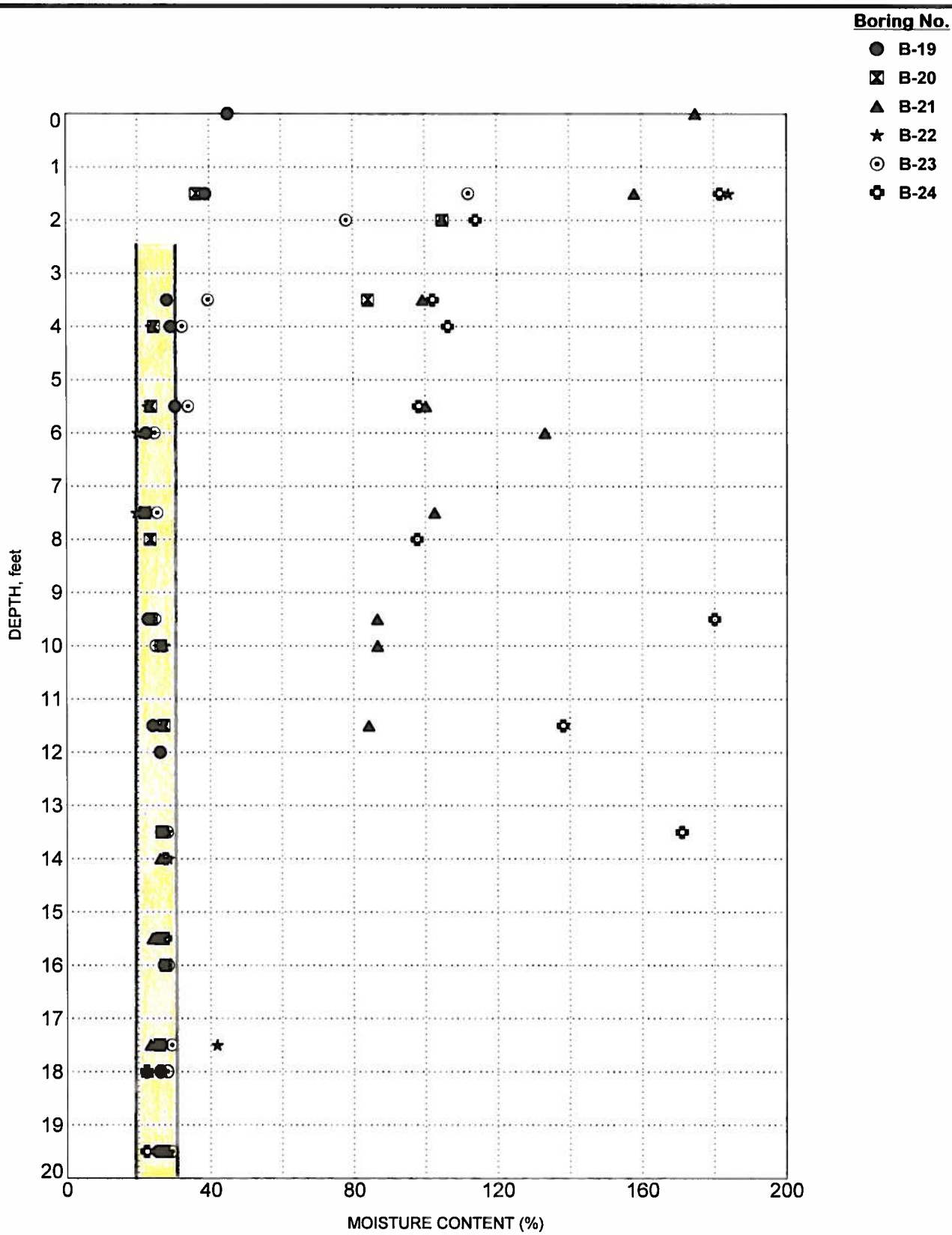

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ARD DEPTH VS MOISTURE CONTENT 13-80-3705.GPJ US_LAB.GDT 3/27/14

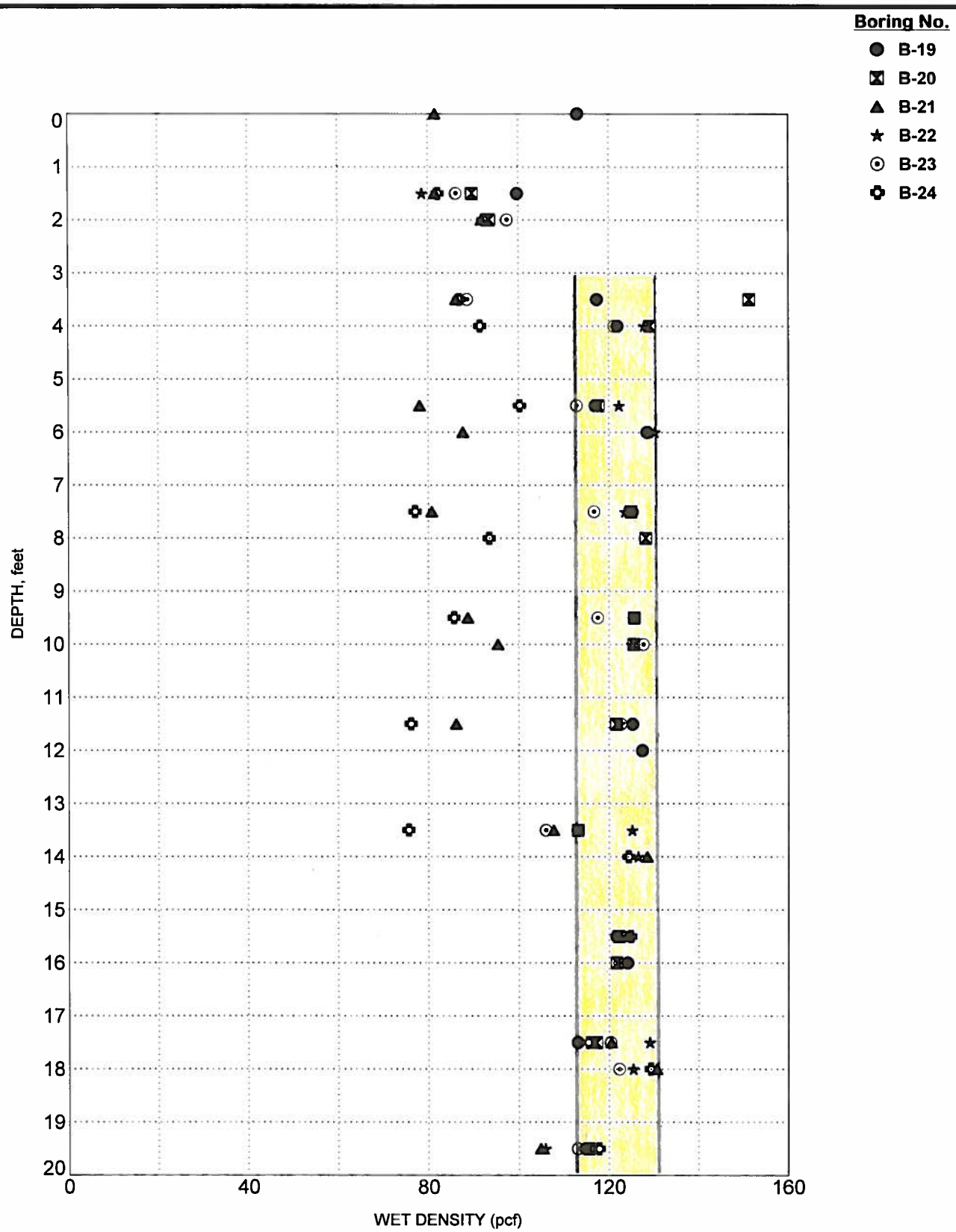
Ardaman & Associates, Inc.

MOISTURE CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-16



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ARD DEPTH VS WET DENSITY 13-80-3705.GPJ US_LAB.GDT 3/27/14

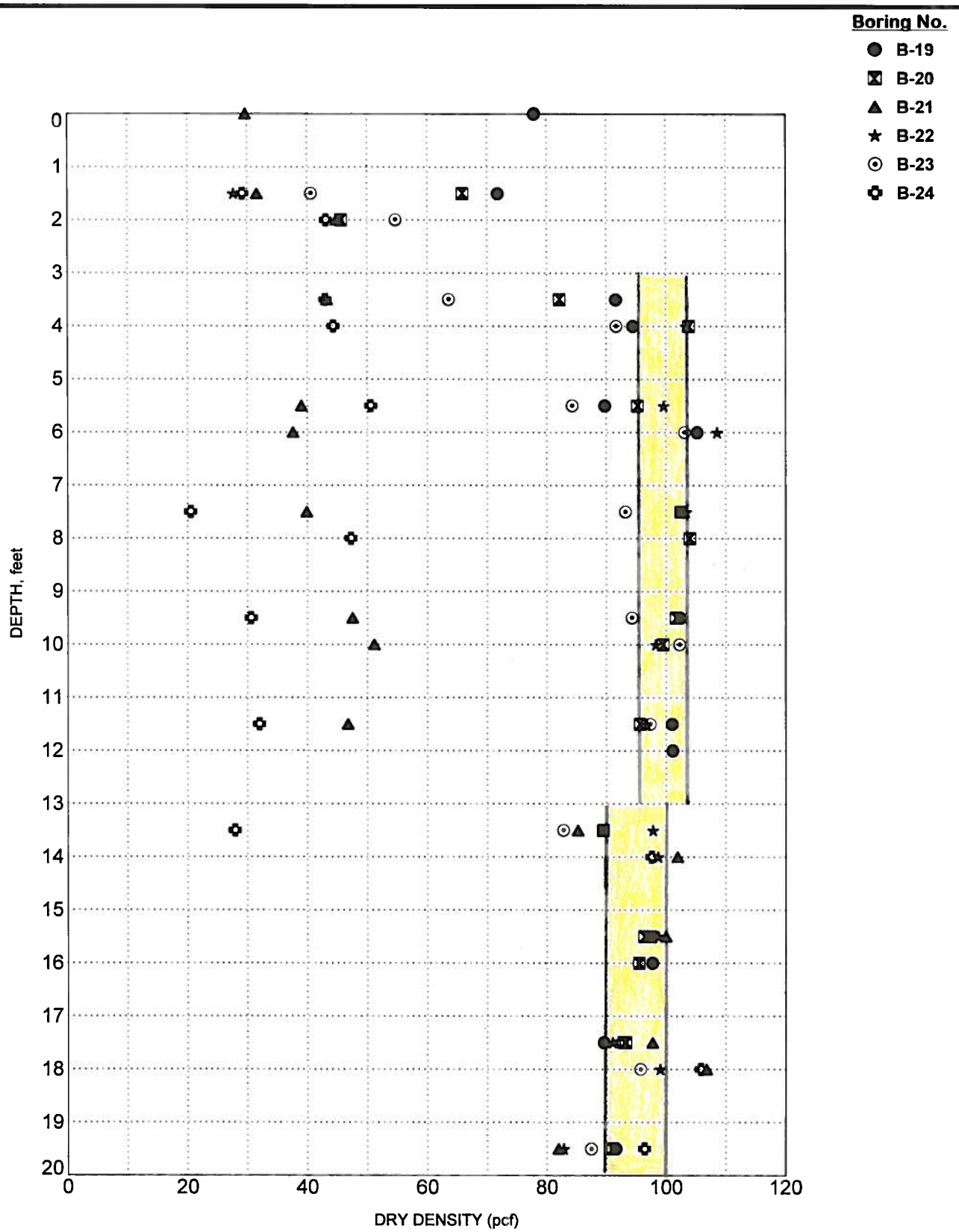
Ardaman & Associates, Inc.

WET DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-17



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ARD DEPTH VS DRY DENSITY 13-80-3705.GPJ US LAB.GDT 3/27/14

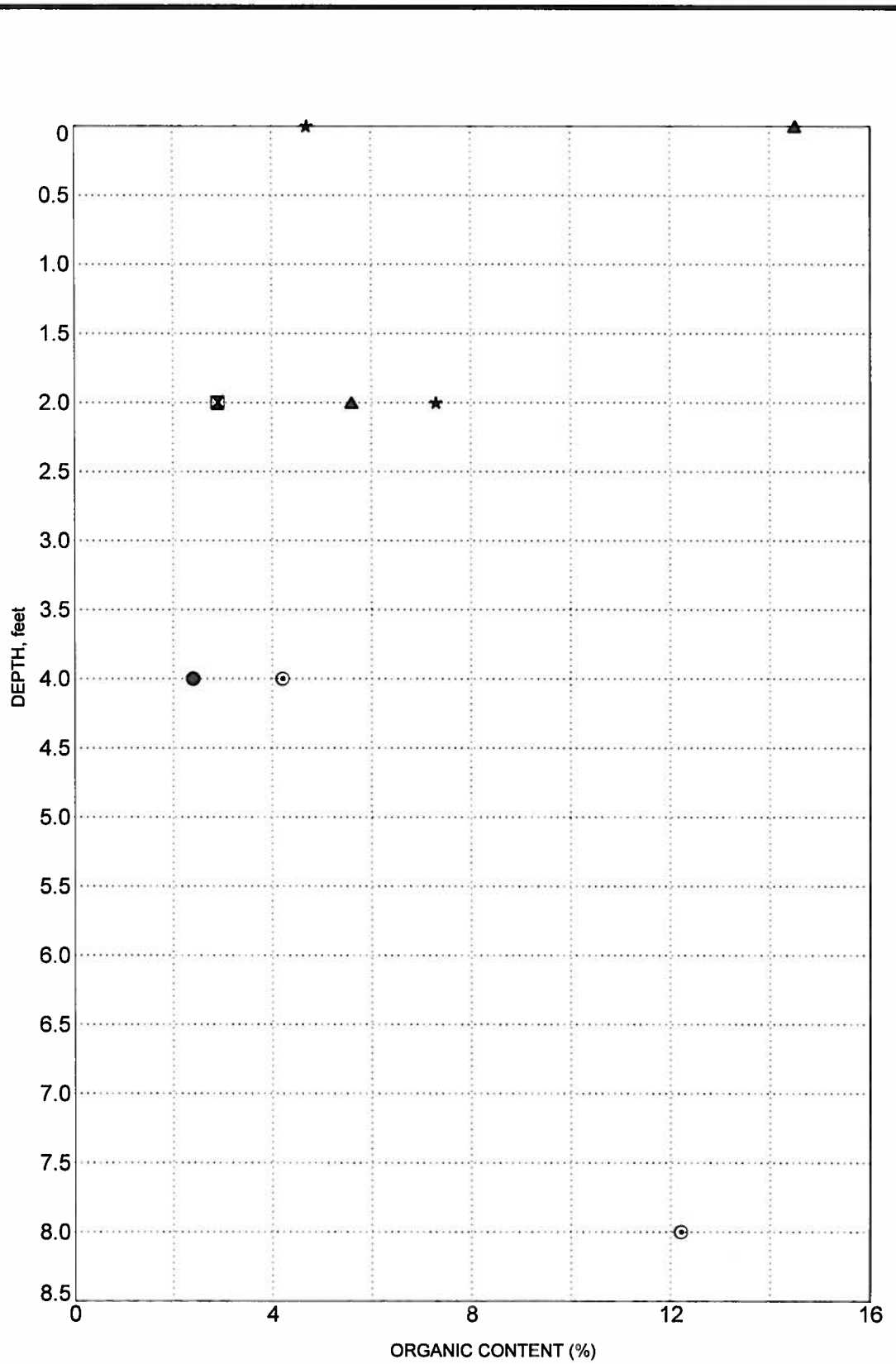
Ardaman & Associates, Inc.

DRY DENSITY vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-18



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ARD DEPTH VS ORGANIC CONTENT 13-80-3705.GPJ US_LAB.GDT 3/27/14

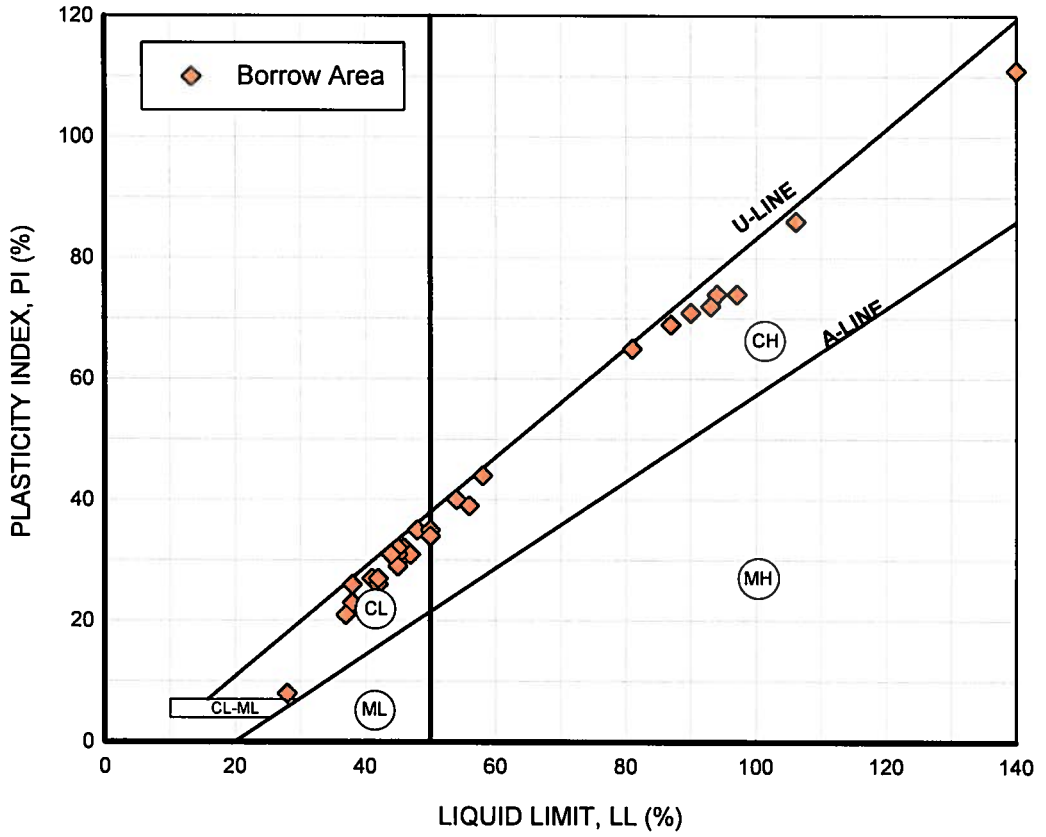
Ardaman & Associates, Inc.

ORGANIC CONTENT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-19



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Geotechnical, Environmental and Materials Consultants

COLE'S BAYOU MARSH CREATION

CPRA

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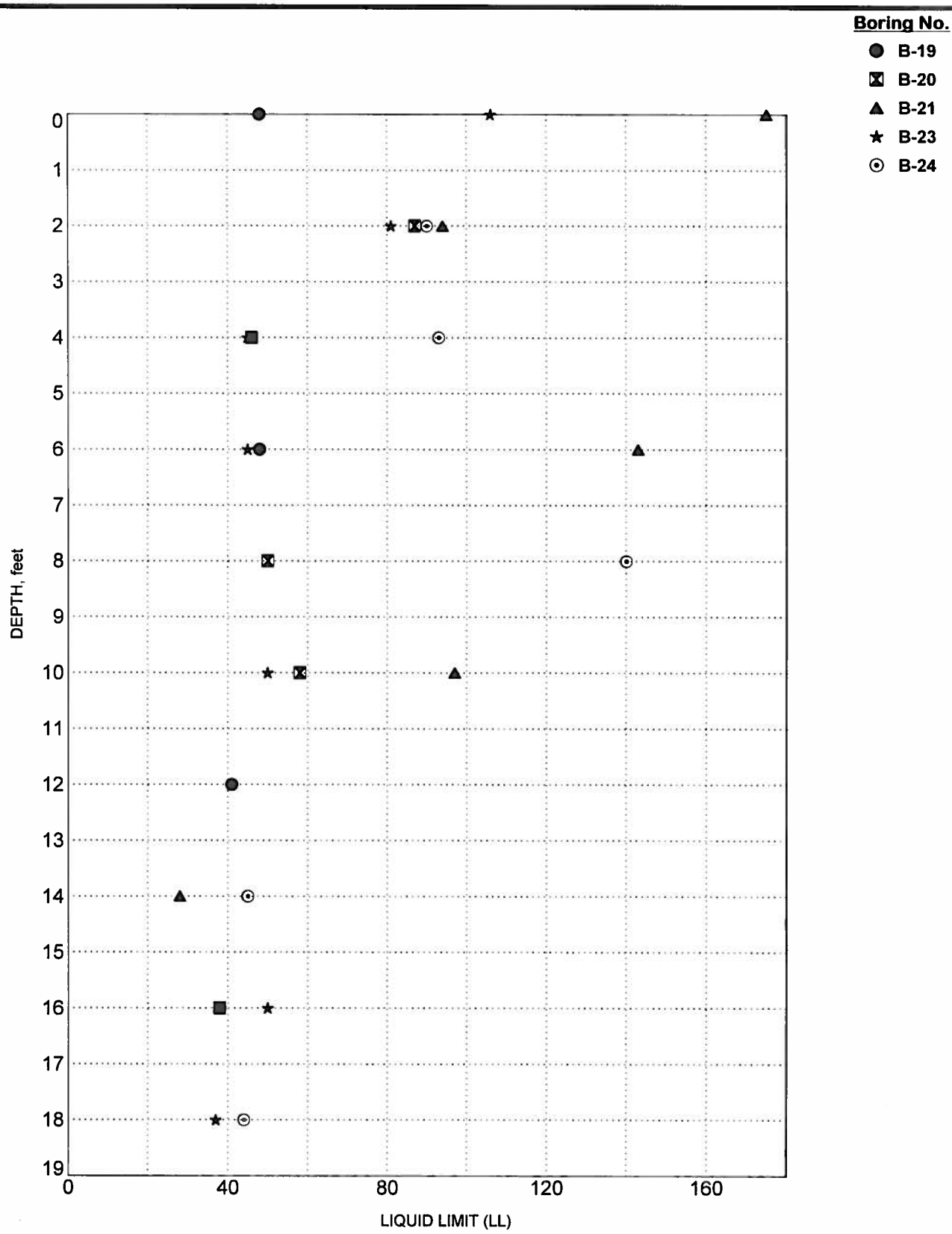
FILE NO. 13-80-3705	APPROVED BY MB	FIGURE B-20



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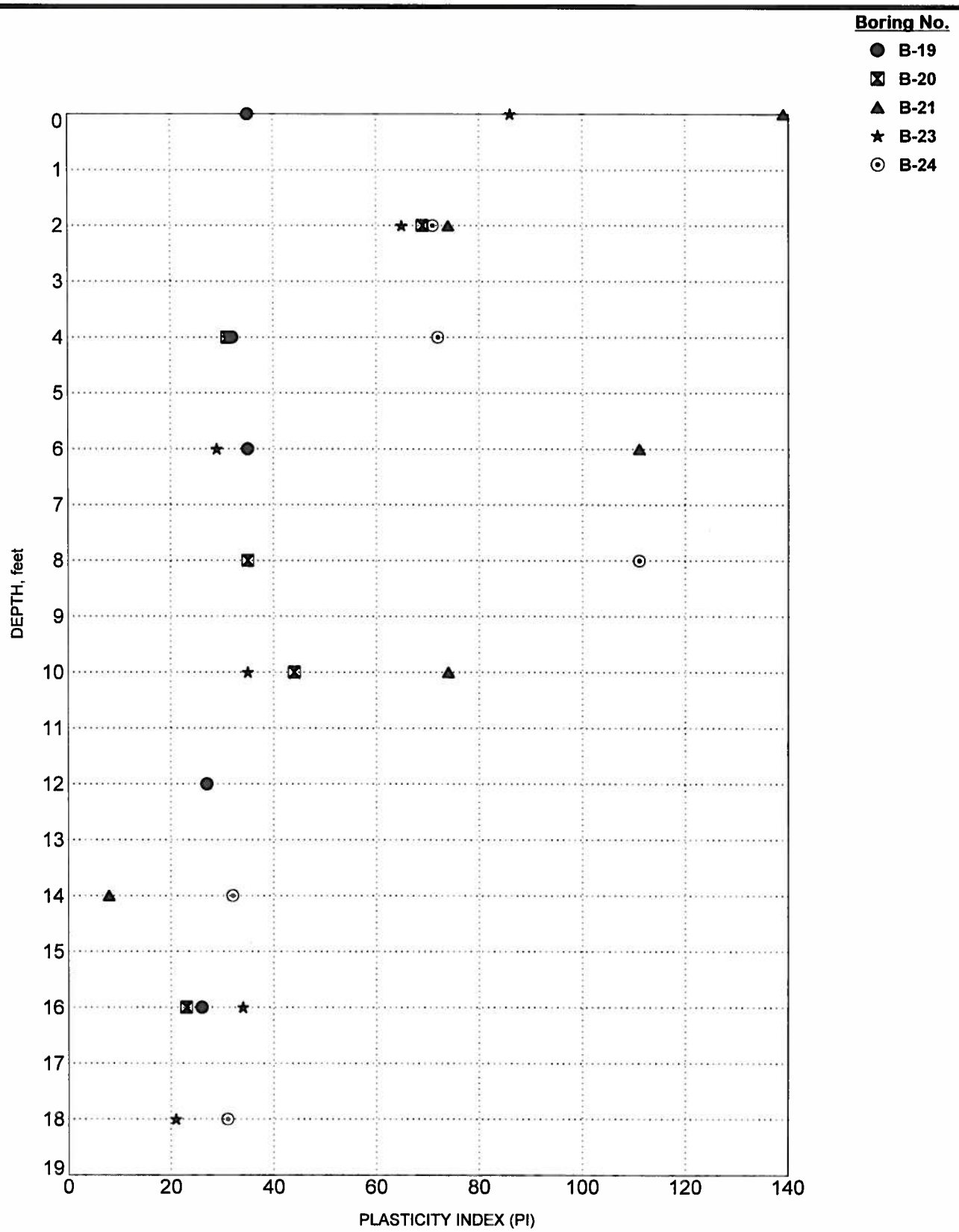
Ardaman & Associates, Inc.

LIQUID LIMIT vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-21



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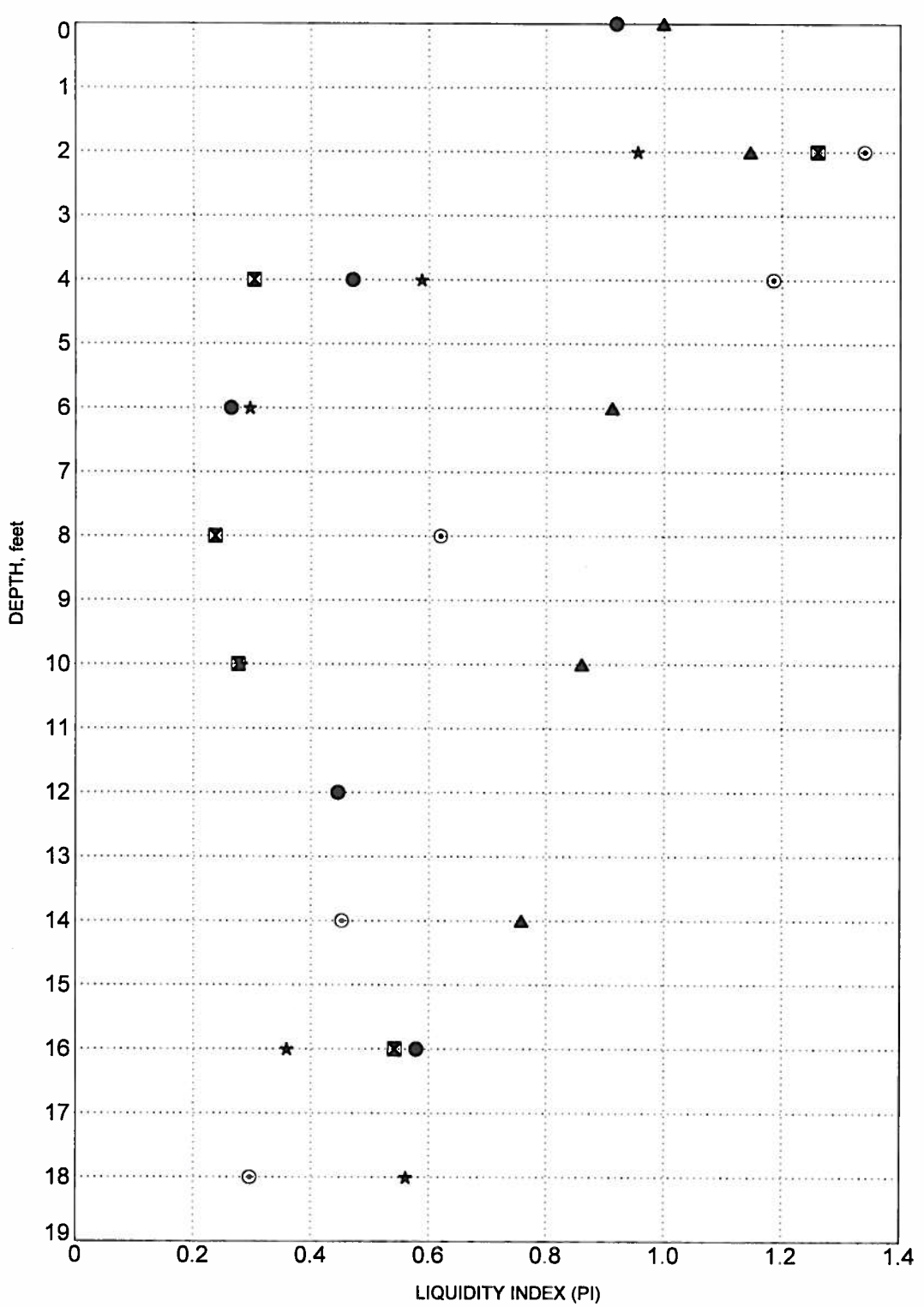
Ardaman & Associates, Inc.

PLASTICITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-22



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ARD DEPTH VS LI 13-80-3705.GPJ US_LAB.GDT 3/27/14

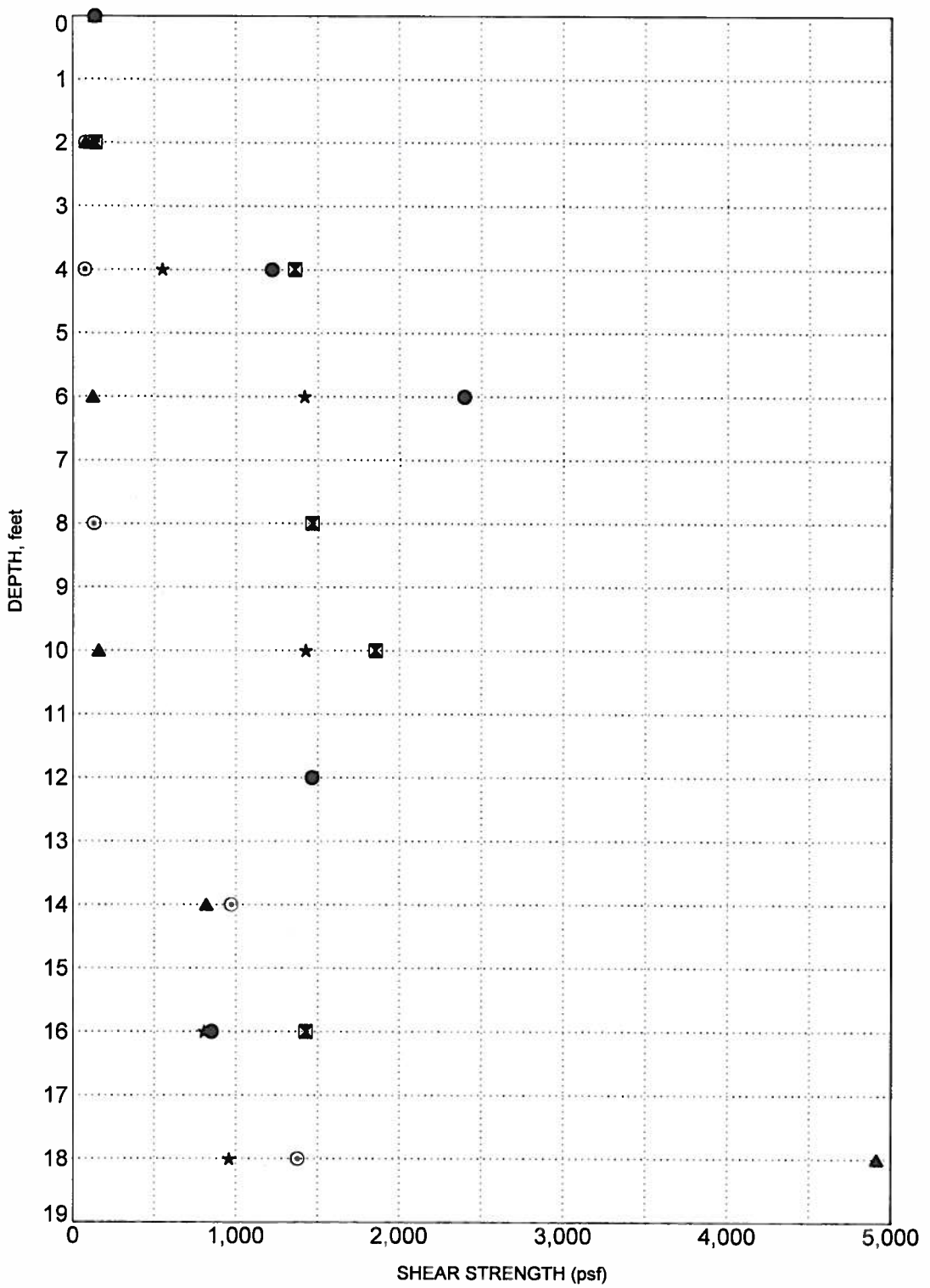
Ardaman & Associates, Inc.

LIQUIDITY INDEX vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-23



Confidential Information:
Privileged & Confidential
Work Product

ARD DEPTH VS SHEAR STRENGTH 13-80-3705.GPJ US_LAB.GDT 3/27/14

Ardaman & Associates, Inc.

UUC SHEAR STRENGTH vs DEPTH

Project: Cole's Bayou Marsh Restoration (TV-63)

Client: CPRA

AAI Project Number: 13-80-3705 Figure No.: B-24

SCALE
1" = 2000'

Confidential Information
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Work Product

REFERENCE
AERIAL BY GOOGLE EARTH 2006

NOTE:
BORING LOCATIONS ARE APPROXIMATE.

COLE'S BAYOU MARSH CREATION AND HYDROLOGIC RESTORATION
VERMILION PARISH, LOUISIANA

for

COASTAL PROTECTION & RESTORATION AUTHORITY
BATON ROUGE, LOUISIANA

AAI
Ardaman & Associates, Inc.
Baton Rouge, LA Jefferson, LA

Project Engineer:	Drawn by:	Checked by:
R. ROUSSET	RJB	RER
File No.: 13-80-3705	Date: 12-17-13	Figure No.: B-25

Title:
2006 AERIAL OF BORROW AREA BORINGS

Mar 27, 2014 - 1:49pm
P:\2013\13-80-3705 Cole's Bayou\AutoCAD\13-80-3705 Aerial of Borrow Area.dwg

APPENDIX F. SETTLING AND SLURRY CONSOLIDATION TEST RESULTS

This Appendix contains the following:

- Settling Column Test Results
- Slurry Consolidation Test Results

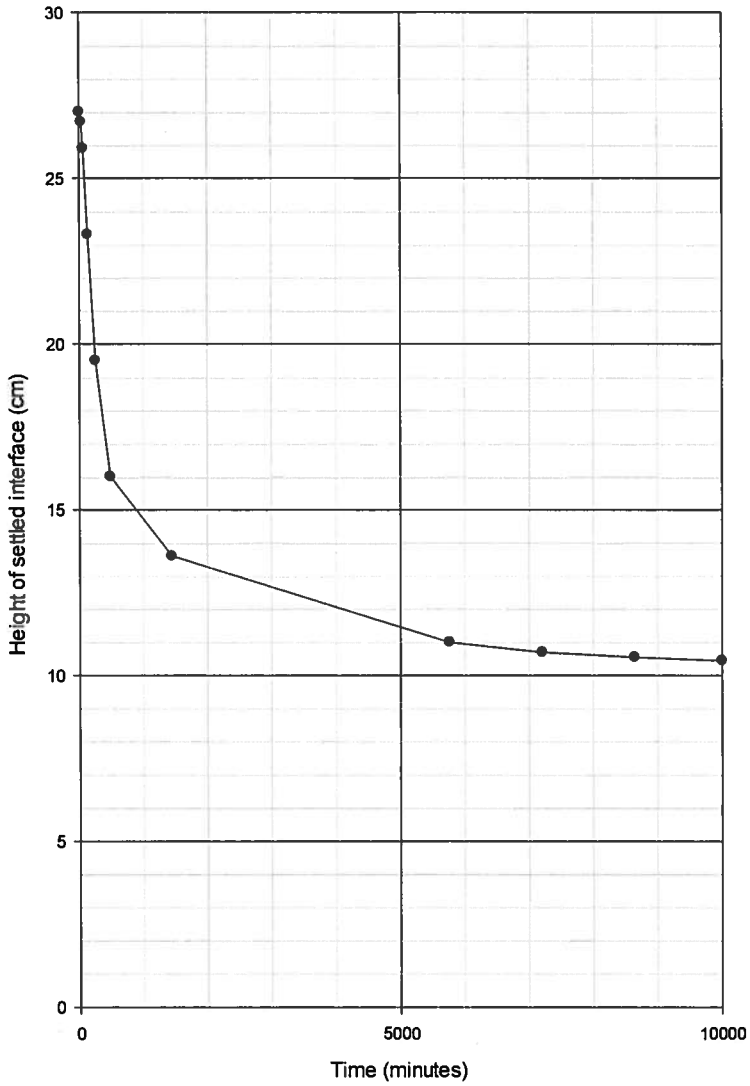
ARDAMAN & ASSOCIATES, INC. GEOTECHNICAL TESTING LABORATORY
SETTLING TEST REPORT

CLIENT: CPRA
PROJECT: Cole's Bayou
FILE NO.: 13-80-3705

DATE SAMPLE RECEIVED: -
DATE TEST SET-UP: 01/30/14
DATE REPORTED: 03/14/14

INCOMING SAMPLE NO.: **Composite 1**
BORING: -
SAMPLE: -
DEPTH: Borrow Area Composite ☐ ft; ☐ m
LABORATORY IDENTIFICATION: 133705/CS1
SAMPLE DESCRIPTION: Gray clay

Settling Tests on Cole's Bayou Composite 1



Settling Column Diameter: 10.765 cm
Initial Slurry Height: 27.0 cm
Initial Slurry Solids Content: 13.4 %
Final Settled Solids Content: 31.4 %
Final Settled Height: 10.1 cm
Test Duration: 13 days

Time (minutes)	Settled Ht. (cm)	Time (minutes)	Settled Ht. (cm)
0.0	27.00		
30	26.70		
60	25.90		
127	23.30		
247	19.15		
480	16.00		
1440	13.60		
5760	11.00		
7200	10.70		
8640	10.55		
10,080	10.45		
14,400	10.25		
15,840	10.15		
17,280	10.10		
18,720	10.10		

Initial Settling Velocity

V_s:
Time Interval:

Comments:

Confidential Information:
Privileged & Confidential
Work Product

The test data and all associated project information presented hereon shall be held in confidence and disclosed to other parties only with the authorization of the Client. Physical and electronic records of each project are kept for a minimum of 7 years. Test samples are kept in storage for at least 10 working days after mailing of the test report, prior to being discarded, unless a longer storage period is requested in writing and accepted by Ardaman & Associates, Inc.

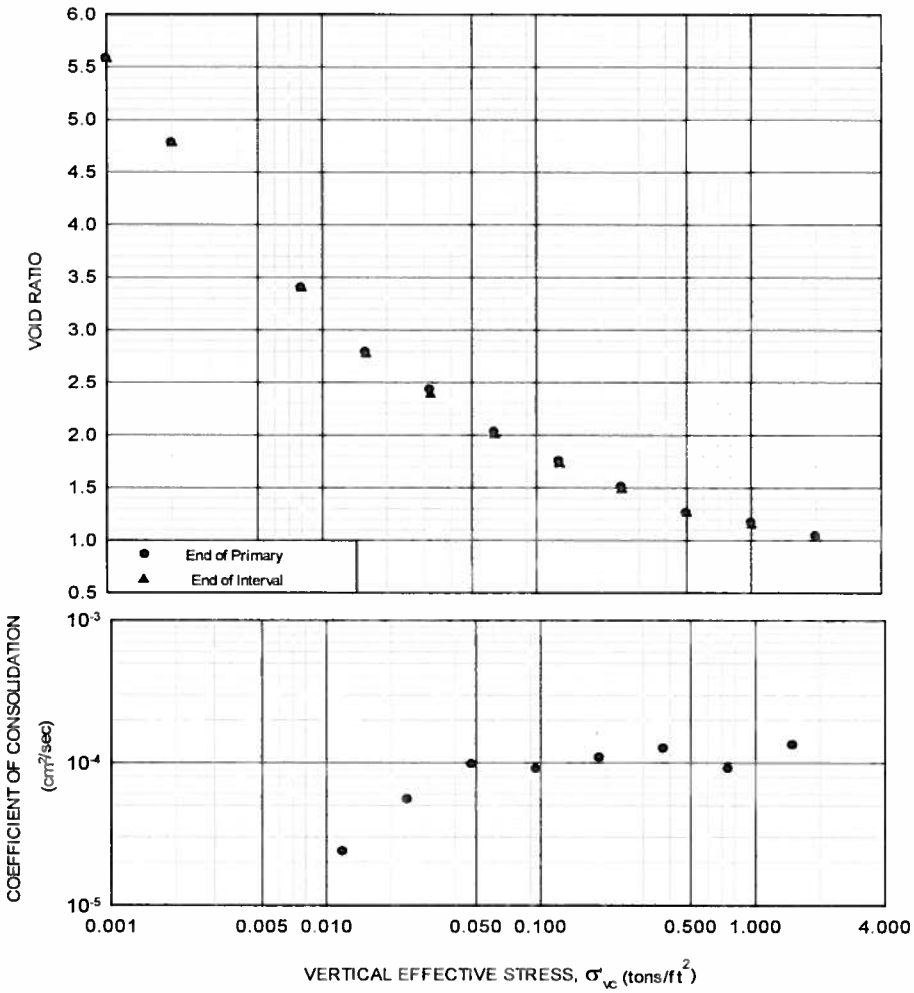
Checked By: Date: 3/24/14

ARDAMAN & ASSOCIATES, INC. GEOTECHNICAL TESTING LABORATORY
ONE-DIMENSIONAL INCREMENTAL LOADING CONSOLIDATION TEST REPORT

CLIENT: CPRA
PROJECT: Cole's Bayou
FILE NO.: 13-80-3705

DATE SAMPLE RECEIVED: -
DATE SAMPLE SET-UP: 02/12/14
DATE REPORTED: 03/20/14

INCOMING SAMPLE NO.: ---
BORING: **Composite Sample #1** SAMPLE:
DEPTH: ☐ ft; ☐ m
LAB IDENTIFICATION NO.: 133705/CS1
SAMPLE DESCRIPTION.: Brown clay



Test Methods & Procedures

ASTM Standard D 2435
- ☐ Method A
- ☑ Method B

c_v Interpretation Method
- ☑ c_v [Log Time]
- ☐ c_v [Sq. Root Time]

Trimming Method
- ☐ cutting shoe
- ☑ other Compacted

Initial Sample Diameter - (cm)

Test Conditions

- ☐ Tested at Natural Moisture Content
- ☑ Specimen Tested Inundated

Inundated at σ'_{vc} 0.05 (tsf)

Inundation Fluid:
- ☑ tap water
- ☐ other

Specimen Conditions

Parameter	Initial	Final
D (cm)	10.015	10.015
H (cm)	5.500	1.452
w_c (%)	244.7	37.6
γ_d (pcf)	22.2	84.0
e	6.657	1.021
S (%)	100	100

G_s: 2.72 ☐ Assumed ☑ Measured

Index Properties

LL	-
PL	-
PI	-

Particle-Size Analysis	U.S. Standard Sieve Size	Gravel			Coarse Sand	Medium Sand		Fine			
☐ ASTM D 422 ☑ ASTM D 1140-Method		3/4"	3/8"	No. 4	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200
Dry Mass(g): 154.49	Soil Passing (%, dry mass basis)	-----	-----	-----	-----	-----	-----	-----	-----	-----	98.0

Confidential Information: Privileged & Confidential Work Product

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Where: H=Specimen height; D = Specimen diameter; w_c = Moisture content (ASTM D 2216); γ_d = Dry density; e = Void ratio; S = Saturation; G_s= Specific gravity; c_v = Coefficient of consolidation; and $C_{\alpha e}$ = Secondary compression index.

Checked By: [signature] Date: 3/24/14