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Geotechnical Standards

Marsh Creation and Coastal Restoration Projects

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Coastal Protection and Restoration Authority

This document was prepared by the Louisiana Coastal Protection and Restoration Authority (CPRA) to support the 2017 Coastal Master Plan.

The Geotechnical Standards for Marsh Creation and Coastal Restoration is a living document and CPRA will continue to improve and update these guidelines as new information is discovered and as design techniques evolve.

Engineering Design Documents, Plans, and Specifications shall be prepared by or under the direct supervision of a licensed professional engineer and registered in the state of Louisiana following professional engineering standards as per La. R.S. Title 37, and Louisiana Administrative Code Title 46, Part LXI, Professional and Occupational Standards, as governed by the Louisiana Professional Engineering and Land Surveying Board.

Executive Summary

The Coastal Protection and Restoration Authority (CPRA) is developing the Geotechnical Standards to be used as the minimum standard consistent with sound engineering practices for the implementation of CPRA and State of Louisiana funded Marsh Creation and Coastal Restoration Projects within the Louisiana Coastal Zone. The CPRA Louisiana Flood Protection Design Guidelines (LFPDG) Geotechnical Section, along with the technical expertise gained from the design and construction of marsh creation and coastal restoration projects in coastal Louisiana soils, were utilized in the development of this document.

The Geotechnical Standards for Marsh Creation and Coastal Restoration Projects provides guidance pertaining to subsurface investigations, soil boring layout, CPT layout, laboratory testing requirements, earthen containment dike geometry, slope stability design, and estimated consolidation settlement design requirements for marsh creation and coastal restoration projects within the Louisiana Coastal Zone.

This document was prepared by the CPRA Engineering Division and is intended as a guide for designing and constructing marsh creation and coastal restoration projects and is not intended to replace the professional engineering judgment of the design engineer.

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List of Abbreviations

CPRA	Coastal Protection and Restoration Authority
CWPPRA	Coastal Wetlands Planning Protection and Restoration Act
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
USCS	Unified Soil Classification System
NAVFAC	Naval Facilities Engineering Command
ASTM	American Society of Testing and Materials
FOS	Factor of Safety
EM	USACE - Engineering Manual
NHI	National Highway Institute
ER	USACE - Engineering Regulations
ETL	USACE - Engineering Technical Letter
CPT	Cone Penetrometer Test
ECD	Earthen Containment Dike
MC	Marsh Creation
CMF	Construction Marsh Fill
MHW	Mean High Water
MLW	Mean Low Water
RSLR	Relative Sea Level Rise
CIMS	Coastal Information Management System
LASARD	Louisiana Sand Resources Database

List of Project Definitions

ECD	The earthen containment dike (ECD) is typically required to contain the hydraulically dredged slurry material and is typically constructed utilizing in-situ material. The borrow area is typically located on the interior. See Figure B-1.
MC	The goal of a marsh creation (MC) project is to create wetland habitat in typically degraded coastal marsh regions in an effort to maximize ecological benefits for the project design life duration. MC projects are implemented by constructing an earthen containment dike (ECD) template, hydraulically dredging material from the borrow area, providing a pipeline corridor for material conveyance, and pumping the slurry material to designated marsh creation areas to create acreage. The project benefits are typically based on the wetland benefits. See Figure B-1.
MLW	Mean Low Water level elevation. See Figure B-1.
MHW	Mean High Water level elevation. See Figure B-1.
WVA	The wetland value assessment (WVA) methodology is a CWPPRA methodology utilized for measuring the wetland benefits of proposed projects and allows for a comparison of benefits between projects.
CMF	The constructed marsh fill elevation, the top of the marsh fill, upon the completion of material placement. See Figure B-1.
TOW	Top of Wall (I-Walls) water elevation is the top of wall which accounts for freeboard.
Design Life	Design life is the length of time a project is estimated to maximize project benefits.
Service Life	Service life is the length of time a project will remain in use to provide its intended function.
Soil Boring	A subsurface investigatory method to advance, collect/conduct samples or in-situ tests by rotary or direct push methods. Undisturbed types include 3-inch continuous sampling using fixed piston sampler, while general type, include 3-inch with open tube samplers, SPTs, and CPTs.

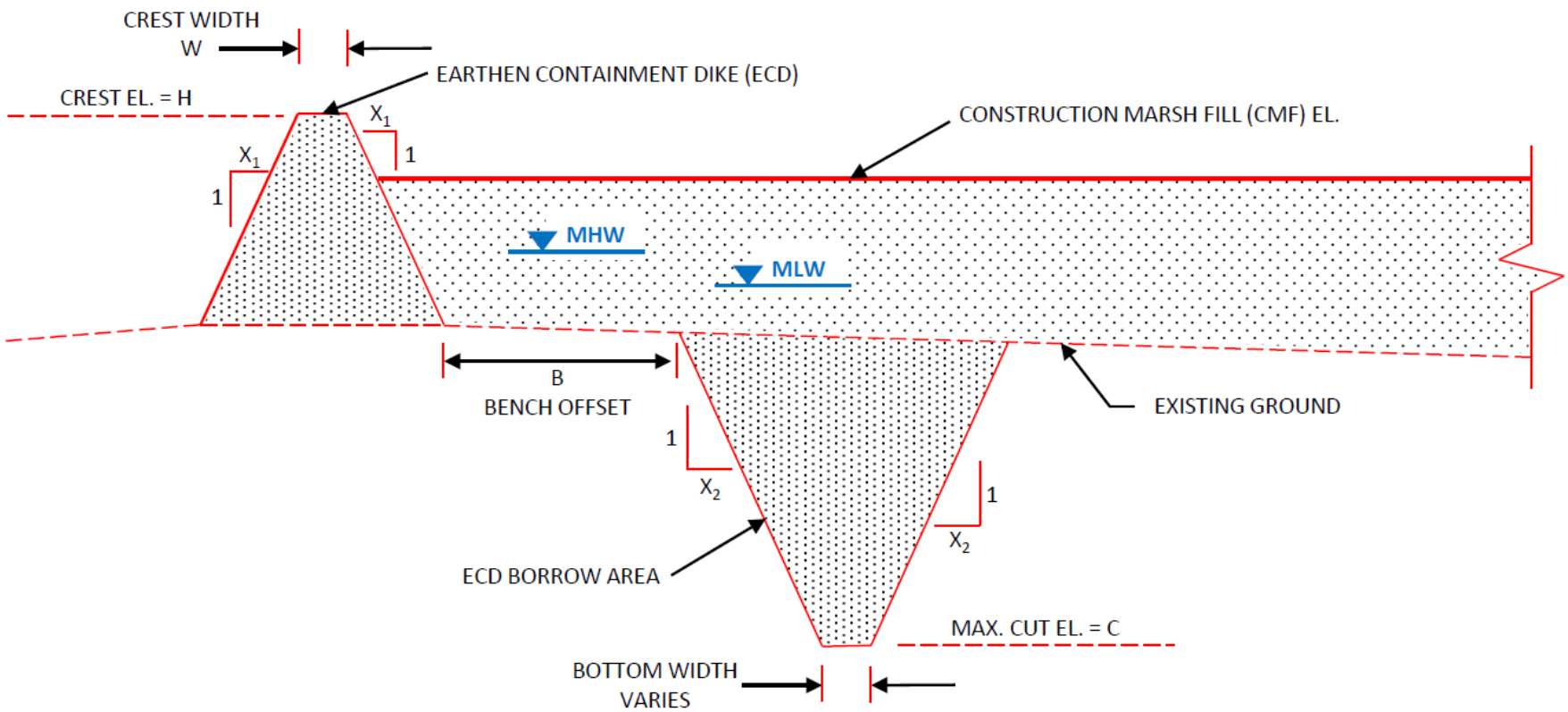


Figure B-1: Typical ECD and Marsh Creation Section

1.0 Introduction

This document has been prepared by CPRA to provide geotechnical engineering design guidance for the State of Louisiana on marsh creation and coastal restoration projects.

This is a living document and is subject to change based on new conditions, data, analyses, research, or regulatory guidelines, as coordinated between the CPRA and local entities.

The primary purpose of this document is to provide a consistent framework for the geotechnical evaluation, design, and construction of marsh creation and coastal restoration projects within the Louisiana Costal Zone. These design guidelines are intended to provide uniformity, expedite analysis, and reduce the timeframe and number of reviews by CPRA, State, local agencies, and Engineering firms. This design methodology should be used by all parties working on marsh creation and restoration projects directly or indirectly affecting State and local systems, and systems where State funding is utilized.

The intent of these geotechnical guidelines is to summarize current applicable project information, design information, and guidelines and methodology for the analysis and design of marsh creation and coastal restoration projects. As such, it is recommended project and site specific design criteria, design approaches, methodologies and acceptance criteria be developed for each project type. Additional coordination is recommended with CPRA if projects or conditions cannot be partially or fully addressed by these guidelines.

2.0 Subsurface Investigation

2.1 General

The purpose of this section is to provide guidance and establish standards to be followed for geotechnical subsurface investigations, documentation of field testing, sample descriptions, field logging, instrumentation, and the handling and storage of samples.

The guidelines and procedures presented herein are intended to be of general use. A specific Subsurface Investigation Plan, including any modifications to these guidelines, should be prepared and documented prior to beginning the subsurface investigation work. Additionally, appropriate revisions may be made based on permitting conditions, field conditions, and equipment access, which should also be coordinated with the CPRA prior to beginning the subsurface investigation work.

The specific requirements of each drilling contract are presented in the Subsurface Investigation Plan to be prepared by the project Geotechnical Engineer. Field personnel should be familiar with the requirements of the Subsurface Investigation Plan.

2.2 Subsurface Investigation Requirements

The Louisiana coast is primarily made up of alluvial deposits due to the formation of the delta complex and the processes at work for thousands of years within this dynamic coastal

environment. A thorough understanding of the physical soil properties is required to adequately design and construct restoration projects within the Louisiana coastal zone.

Adequate site characterization is dependent upon the degree of understanding of the site prior to the field investigation. At a minimum, the review of available geologic maps, geomorphic maps, aerial topography, bathymetry, soils maps, existing soil boring logs and cone penetrometer tests (CPTs), well logs, published papers, reports, and available information from local, state and federal agencies, such as the United States Geological Survey (USGS), the United States Army Corps of Engineers (USACE), LASSARD, and CIMS should be conducted prior to the development of the Subsurface Investigation Plan.

The quantity and quality of existing subsurface information will influence the scope of work and schedule of a proposed subsurface investigation. Other factors to consider when developing a Subsurface Investigation Plan include location, coastal restoration project type, geology, geomorphology, groundwater, borrow material, knowledge of the area, and risk tolerance.

In some cases it may be prudent to approach the subsurface investigation in phases to develop a more comprehensive plan. For example, Cone Penetration Testing (CPTs) could be advanced quickly and in greater numbers to characterize a site, then a targeted soil boring program could be developed to supplement the preliminary CPT results.

2.2.1 Soil Boring and CPT Spacing

The following should be considered as typical spacing guidance for marsh creation and coastal restoration projects, and may be altered based on past knowledge and experience of a particular site.

For example, to optimize subsurface investigation coverage along an ECD alignment, soil borings and CPTs may both be utilized. A geotechnical engineer or geologist experienced in subsurface investigations may utilize a different spacing and soil boring type layout based on engineering experience and professional judgment.

<u>Phased Subsurface Investigations using CPT</u>

The use of phased investigations to quickly capture site wide geotechnical data through the use of CPTs may be considered where multiple subsurface investigations are planned to refine data and information for a restoration project site, with each phase refining the investigation spatially or by capturing more geotechnical testing information and data. In a phased investigation, the initial or preliminary investigation may use CPTs in lieu of soil borings, if the subsequent investigations further refine the overall site investigation with CPTs, and soil borings, to adequately characterize the site, and thoroughly identify foundation shear strengths and compressibility.

<u>Additional Investigations</u>

Depending on factors, such as geologic variability, weak or compressible soil profiles, past performance or critical structures, closer soil boring spacing may be necessary, and should be based on engineering judgment. Executed Subsurface Investigation Plans should be reviewed for their desired outcome. For example, if unanticipated zones of weak soils were encountered, unusable samples obtained from a soil boring, or more information is needed within an area because of unexpected variability, then further exploration should be planned. The overall objective is to optimize the Subsurface Investigation Plan.

Soil Boring and CPT Spacing and Depth

Soil Borings, CPT's, and vibracores should be spaced in accordance with Table B-1 for marsh creation projects and coastal restoration projects.

Table B- 1: Suggested Soil Boring Spacing, CPT Spacing, and Depth for Restoration Projects.

Restoration Project Feature	Soil Boring & CPT Location	Type	Soil Boring & CPT Spacing (ft.)	Soil Boring & CPT Depth (ft.)
Marsh Creation (MC) Area	Proposed Fill Area	3" Undisturbed Boring	1 per 50 to 100 acres (1 min. per area)	30' – 40'
Earthen Containment Dike (ECD)	Centerline	3" Undisturbed Boring/CPT	1,500' – 3000'	40' – 60'
MC "Inland" Borrow Area	Proposed Borrow Area	3" General Type Boring/CPT/ *Vibracore	1 per 100 acres (4-6 min. per area)	‡ 30' – 60'
MC "Offshore" Borrow Area	Proposed Borrow Area	3" General Type Boring/CPT/ *Vibracore	**1 per 1000 feet	‡ 30' – 40'
"Mississippi River" Borrow Area	Identified Borrow Area	3" General Type Boring/CPT/ *Vibracore	1 per 100 acres (4-6 min. per area)	‡ 50' – 100'
Barrier Island Beach Dune	Centerline	3" Undisturbed Boring/CPT	1,000' – 3000'	60' – 80'
Oyster Barrier Reef	Centerline	3" Undisturbed Boring Boring/CPT	1,000' – 3000'	30' – 60'
Shoreline Protection	Centerline	3" Undisturbed Boring/CPT	1,000' – 3000'	40' – 70'
Ridge Restoration	Centerline	3" Undisturbed Boring/CPT	1,500' – 3000'	40' – 65'
Earthen Terraces	Centerline	3" Undisturbed Boring/CPT	1 per 50 to 100 acres	40' – 60'

Note: *Vibracores may be taken in conjunction with soil borings if disturbed soil samples are required to determine material properties required for hydraulic dredging.

**See current version of the CPRA General Guidelines, Exploration for Sediment Resources for Coastal Restoration.

‡ The soil boring depth should be advanced to the maximum extent of the proposed dredging/excavation Work.

2.2.2 Soil Boring and CPT Depth

As represented in Table B-1, selected soil boring depths should be based on the type of restoration project, existing subsurface geotechnical investigation data, the use of geosynthetics, the presence of adjacent excavations/water bodies, evaluation of the critical slope stability failure

surfaces, the zone of stress increase, maximum dredging depths, evaluation of deep foundations, and the presence of incompressible strata.

The depth of investigation should also extend deep enough below the foundation load or deep transferred load to adequately evaluate stress increase to the foundation soils, foundation index and physical properties, and to adequately determine the extent of foundation compressibility or consolidation due to the structure or load applied. As per the NAVFAC DM 7.1, p. 7.1-181, the critical depth is the zone below the foundation load within which the soil compression contributes significantly to the surface settlement, and extends to the point where the applied stress decreases to 10% of the effective overburden pressure. Soil boring depths should not extend below the critical depth for foundation consolidation analyses. Ultimately, the depths of soil borings, CPT's, and vibracores should be determined by the geotechnical engineer based on the proposed project features, existing subsurface geotechnical investigation data, professional experience, and engineering judgment.

Typical soil boring, CPT's, and vibracore depths are shown in Table B-1.

2.2.3 Soil Boring Type and Sampling

Vibracores

Vibracoring is a technology typically utilized by coastal geologists and coastal engineers for collecting disturbed core samples of underwater sediments. The vibrating mechanism of a vibracorer, sometimes called the "vibra-head", operates on hydraulic, pneumatic, mechanical, or electrical power from an external source. The attached metal core tube is driven into the sediment with the use of gravity, enhanced by vibration energy. When the insertion is completed, the vibracorer is turned off, and the tube is withdrawn with the aid of hoist equipment.

Vibracores are primarily utilized to characterize the physical properties of offshore granular sediment in proposed borrow areas for barrier island restoration projects, to ground truth geophysical data, and are limited to 20 feet to 35 feet in depth depending on the soil density.

However, in certain applications if sample recovery is anticipated, vibracores may also be utilized to characterize the physical properties of a marsh creation borrow area as well. Due to the methodology and equipment used for this type of soil sampling, the in-situ soils are disturbed and should not be utilized for conducting undisturbed geotechnical laboratory tests.

The geotechnical laboratory tests typically conducted on the vibracore material for a proposed marsh creation borrow area generally include the USCS Classification, Gradation/Hydrometer, Moisture Content, Atterberg Limits, Unit Weight, and Specific Gravity. These test results will be utilized by the dredger and the designer for estimating dredging production rates and marsh fill behavior, and should be characterized and tested at intervals equivalent to the continuous soil sampling methods. Representative composite samples from the proposed marsh creation borrow area are also typically utilized to conduct the settling column test and the low stress consolidation test to estimate long term consolidation settlement of the hydraulically dredged marsh fill, as per Section 3.5.

The use of this type of sampling methodology should be based on the professional judgement of the designer. For offshore borrow area investigations, please refer to the most current CPRA General Guidelines, Exploration for Sediment Resources for Coastal Restoration.

Soil Boring Types

- Undisturbed – Continuous 3-inch diameter Shelby Tubes with a fixed-piston sampler
- General Type – Cone Penetration Tests, 3-inch diameter Shelby Tubes

Fixed-piston type sampling shall be utilized for fine-grained soil classified by USCS as OH, OL, CH, CL, CL-ML, MH, and ML for "Undisturbed" soil borings. Likewise, Standard Penetration Tests (SPT) shall be performed on all cohesionless soils such as sands or gravels. Where 3-inch Undisturbed Soil Borings are used, CPTs shall be used for the General Type Soil Boring.

In Coastal Louisiana, rotary wash drilling with hydraulically pushed thin-walled tube samplers is the commonly accepted method of obtaining undisturbed samples. The disadvantages of using open-drive samplers include the potential for obtaining samples with disturbed material or soil cuttings from a poorly cleaned borehole, which can be non-representative of overall sample characteristics. Open-drive thin-walled tube samplers may cause excess pressure over the sample during the drive causing the sample to be pushed aside within the sampler, and may not be effective in reducing the pressure over the sample during withdrawal, both of which result in poor quality and sample recovery. Hvorslev (1949) does not recommend open-drive samplers be used for undisturbed sampling of soft or loose soils for those aforementioned reasons.

Open-drive Shelby Tube sampling may also be used to retrieve samples considering the level of investigation necessary, soil conditions, and engineering judgment.

Sampling Intervals

Sampling intervals are recommended to be consistent with the current standard of practice, which consists of continuous sampling of the top 20 feet regardless of boring type.

Drill Advancement Methods

Soil borings shall be advanced by rotary wash with a side or upward discharging bit. Borehole stability shall be maintained and soil cuttings shall be displaced to the surface through the use of a bentonite slurry during borehole advancement.

The following mud rotary guidelines shall be implemented to reduce soil sample disturbance:

- Limit excess down pressure or excess drill penetration rates to reduce damage to clay formations.
- Drilling penetration shall be limited to 1-foot depth of advancement per minute, which limits clay "balling" during the removal of soil cuttings.
- The drill interval shall be re-washed or cleaned if the drill pump experiences an excessive pump surge or pressure increase during the drilling of an advancement interval. The pump shall be operated at the lowest pressure possible to keep the bit clean and to flush cuttings to the surface.
- Do not drill by quickly raising and lowering the drill stem. Cleaning of an advancement interval shall be done in a controlled slow manner to prevent excessive pressures.
- Once the borehole is completed to the sample interval the pump shall be run for a sufficiently long time period to flush and clean the base of the borehole. This time may vary

and should be based on experience and physical measurements of the borehole cleanliness prior to sampling.

- Borehole cleanliness shall be measured with a weighted tape, and shall be free of soil cuttings prior to taking a sample; otherwise the borehole shall be re-cleaned prior to taking the sample.

Borehole drilling mud shall consist of the following:

- Maximum Mud Weight 72 lbs/ft^3 per ASTM D 4380, 1984: R 2012 Standard Test Method for Density of Bentonitic Slurries. Virgin mud shall be used once mud weights exceed 72 lbs/ft^3.
- Mud Viscosity as measured by a Marsh Funnel shall be maintained within 60 to 70 seconds per API RP 13B-1 (2003).

Mud properties shall be continuously monitored and measured frequently during the drilling and sampling process.

2.2.3.1 Undisturbed Sampling

The 3-inch diameter relatively undisturbed clay and silt strata samples are obtained using thin-walled Shelby Tubes with a fixed-piston type sampler. This sampler and sample tube system consists of a piston that is connected to the piston rod (sometimes referred to as the inner rod). Within the sampler head, a locking mechanism (double-thread locking block type) allows the piston to be locked to the head in the down position (just inside the cutting edge of the sample tube), preventing the piston from falling out of the sample tube when in a vertical position. The piston rods or inner rods are then threaded inside the drill pipe during the lowering of the sampler to the sample interval. Once in position to sample, the piston is unlocked from the sampler head and the inner rod is fixed to the drilling rig. The steps used in obtaining the undisturbed sample using a fixed-piston sampler are as follows:

- A 30- to 36-inch and 54-inch long Shelby tube(s) is slipped over the piston and connected to the header with four bolts, for 3-inch and 5-inch samplers, respectively.
- The piston is positioned at the bottom of the tube (just inside the cutting edge) and locked (double-thread locking block type).
- The piston rod is connected to the smaller inner rods within outer drill rods.
- Once the piston is released (double-thread locking block type), the top of the inner rod is held in position (mechanically fixed to the drill rig) while the outer rod is pushed downward using the drill rig (i.e., the piston remains at the top of the sampling interval during the push).
- After pushing the Shelby tube in one continuous push to obtain the sample, the inner rod is removed from the outer drill rod and the sampler and Shelby tube are removed from the borehole.
- The push shall be measured and marked, and be approximately 6 inches less than the sample tube length.
- Prior to retrieval, the drill rod and attached sampler shall be rotated to shear the sample from the underlying formation.
- Once at the ground surface, the piston sampler, while still attached to the sample tube, is placed horizontally on a table, saw horses, or appropriate work surface.

- The vacuum breaker screw is removed to release the vacuum on the sample.
- The sample will then be field classified by a qualified geologist, geotechnical engineer, or a soil technician and prepared for transport to the laboratory.

Shelby tubes shall meet the requirements of ASTM D 1587-08(2012) e1 – Standard Practice for Thin-walled Tube Sampling of Soils for Geotechnical Purposes. In cases of less cohesive or leaner materials, such as silts, straight-sided Shelby tubes with inside clearance ratios of approximately zero may be considered based on actual field recovery and sampling experience of the geotechnical engineer.

The Shelby tube is pushed using the drill rig to obtain the sample and then removed from the borehole. All sampling shall be from a "clean" borehole. Potential very stiff to hard clays may be present in the upper overburden soils; therefore, if needed to ensure adequate recovery, Split-Spoon Sampling (SPT) may be considered during the field exploration. The soil recovered is field classified by a geologist or geotechnical engineer.

For sample intervals where little to no recovery is encountered, the sample interval shall be drilled out or cleaned to the bottom of the sampled or pushed interval. Then, a standard penetration test (SPT) will be taken, regardless of soil type. In the event back-to-back samples with less than 25 percent recovery occurs, the sample interval in question will be re-drilled and sampled. At a minimum, the zone should be over sampled by one sample above to one sample below.

To identify any re-drilled locations, an "R" shall be incorporated in the last string of the boring identification.

Drill cuttings or softened material from the top of the sample and a portion of the material at the bottom of the sample shall be removed by a soil lathe. Field classifications shall be determined from the bottom material removed for placement of the expandable packer. The top softened materials shall be discarded. Prior to capping the recovery shall be recorded by the soil logger. Shelby tubes shall then be sealed with expandable packers and capped, taped and labeled prior to shipment to the laboratory for extrusion. Labels shall clearly identify the soil boring name, the sample name and interval, direction of sample push, and date sampled. Labels shall be conspicuous and shall be located on the tube. The top cap should also be labeled with the boring name and sample name.

The prepared samples will then be transported from the drill site, typically on a daily basis, to the laboratory using special care to reduce disturbance of the soil sample.

The samples will be kept "shaded" during hot weather periods with a tarp or canopy until they are picked up for daily transport. The samples shall be secured vertically and cushioned to reduce sample disturbance. The samples are then transported to a laboratory for moisture content determination, classification, and testing.

2.2.3.2 Shelby Tube Storage, Extrusion and Shipment

<u>Transportation</u>

ASTM D 4220/D 4220M-14 – Standard Practices for Preserving and Transporting Soil Samples allows various modes of transportation and shipping containers that vary depending on the type of soil testing being performed and the sensitivity of the soil being tested. Based on the ASTM D 4220

groupings, a majority of the soils in Coastal Louisiana would be classified as "Group D" soils because of their sensitivity to disturbance for consolidation, strength, and other more sophisticated soil testing. Following the "Group D" classification, the most common transportation technique is to place the sealed samples in the back of a vehicle in a wood shipping container that provides cushioning and insulation to reduce vibration, shock, and protect against extreme heat for each sample within the shipping container. However, the amount of cushioning material recommended in ASTM D 4220 varies from the recommendations provided in EM 1110-1-1804.

Because reducing sample disturbance is paramount to yield quality samples for shear strength testing, EM 1110-1-1804 should be followed for sample transport.

Soil samples should be transported to the laboratory as quickly as feasible. EM 1110-1-1804 states samples must be stored snuggly to prevent rolling and bumping, and protected from changes in moisture content, shock, vibration, temperature extremes and chemical changes during transport. If samples are transported in tubes, a 1/2-inch to 3/4-inch thick marine plywood box with 3 to 6 inches of cushioning material between the samples, and bottom and sides of the container, or similar container, is recommended.

Extrusion

Though field extrusion is beneficial to reduce disturbance caused by soil-tube adhesion and friction during extrusion, the risk of disturbance from handling soft cohesive soils in the field and transport is increased. This is especially true with the very soft to soft soils typically found in Coastal Louisiana. For this reason, it is recommended that tube samples be sealed in the field and extruded in the laboratory. Field extrusion is recommended when cohesive samples are medium stiff or stiffer in consistency and can be handled and transported with less potential disturbance.

A hydraulic sample jack should be used to extrude a tube sample in one continuous uniform push. Mechanical and pneumatic jacks should not be used for extrusion.

Where it is anticipated extrusion is causing significant additional disturbance to the soil samples for testing, two methods may be used to help reduce disturbance. Cut the tube adjacent to the sampling location or use a hypodermic tube to feed a thin wire along the soil-tube interface and rotate the wire around the perimeter of the tube to break the bond.

Extruded samples shall be carefully caught within a core catcher to reduce disturbance, sample breakage, or bending the sample.

Preservation

After extrusion of the sample, samples should be wrapped in plastic wrap and stored in air-tight containers. It is recommended the designated tests be performed as soon as possible. When tests cannot be performed immediately following extrusion, samples should be waxed. Medium stiff or stiffer soils may, alternatively, be preserved by wrapping in plastic wrap, aluminum foil, and placed in a plastic bag. Because of the quality of pore water in some samples, they may have chemical reactions to the aluminum foil, which may affect the sample quality.

Samples to be waxed should follow:

- Wrap the sample in plastic wrap and wrap in aluminum foil;

- Place sample in two plastic bags (for easier removal of the inner bag with the sample); and,
- Place bagged sample in concrete cylinder mold or similar container and cover with molten wax (wax temperature shall not exceed 180° F).
- Minimum wax thickness above the sample is 0.4 inch or 10 mm.

Storage

Samples shall be temporarily stored within a temperature controlled laboratory environment until testing is complete. Samples slated for long-term storage shall be preserved and stored in a humid room. If mold or fungus growth, or slow chemical changes are anticipated, the samples should be properly refrigerated in accordance with EM 1110-1-1804.

Sample Schedule upon Sample Retrieval

- Extrusion should be performed as soon as feasible, at a minimum within 5 days of field sampling.
- Moisture Contents and Extrusion Logs should be performed within 7 days of field sampling.
- Samples not designated for Index or Physical tests shall be waxed within 7 days of extrusion.
- Index and physical tests such as Atterberg limits, Grain Size analysis, Hydrometer, Percent Passing U.S. No. 200 Sieve, Organics, Unconfined Compression, Unconsolidated-Undrained Triaxial Tests, Consolidated-Undrained Triaxial Tests, Direct Shear, Consolidation, and Permeability tests shall begin within 15 days of field sampling.

The remaining usable portions of specimens shall be waxed as recommended above at the time of test specimen preparation.

2.2.4 Standard Penetration Tests (SPT) with Split-Spoon Sampling

Where cohesionless strata are encountered, the Standard Penetration Test (SPT) shall be performed, ASTM D 1586-11 - Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. The SPT consists of a standard split-barrel sampler with a 2-inch outer diameter, 1.50- or 1.38 –inch inner diameter, and 18 to 24 inches in length, excluding the driving shoe. The SPT sample is collected by driving a total of 18 inches, using a 140-pound hammer falling 30 inches.

The SPT test is performed by recording the blows for three consecutive 6-inch intervals, with the first interval being a seating interval. The test is considered complete if any of the following occurs during the test:

- Sampler moves less than 1-inch in 10 blows;
- Total blows exceed 50 for any 6-inch interval; or
- Total blows exceed 100.

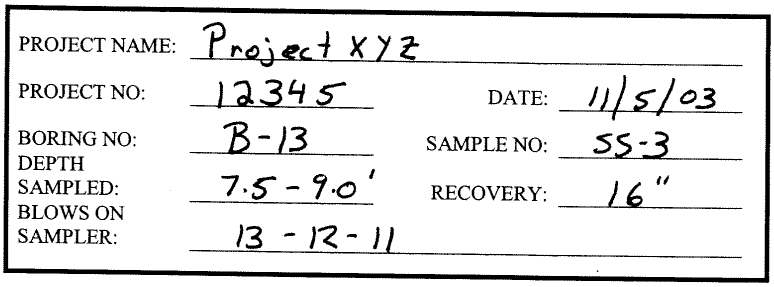
The SPT value is determined by adding the number of blows for the last two 6-inch test intervals, or recording a value of 50 and the inches driven for the termination interval, if the test is completed

prior to driving 18 inches. The test intervals shall be clearly measured and marked to a stationary reference for the soil logger.

CPRA prefers the use of an automatic trip hammer, to maintain consistency of hammer fall. However, if a safety hammer and cathead are used, a maximum of 1-3/4 wraps (CCW cathead rotation) and 2 ¼ wraps (CW cathead rotation) are allowed. Prior to using a safety hammer, the 30-inch fall shall be measured and clearly marked. Regardless of the hammer type, hammers used and/or hammer used by individual drillers shall be calibrated for energy at least on a yearly basis. Rig /operator calibration shall follow ASTM D 4633-10 – Standard Test Method for Energy Measurement for Dynamic Penetrometers. Hammer efficiency information as well as the date of hammer calibration should be reported on the soil boring log.

SPT samplers shall be equipped with a "core catcher" with an appropriate stiffness for every test. Plastic bags or glass jars with screw-on lids shall be used to retain representative split-spoon sample(s). Samples with multiple representative portions shall be split and sampled separately, with sampling interval depths described on each sample jar or bag, and described on the boring log.

Disturbed soil samples placed into sample containers should be labeled to indicate: project name, project number, blows per 6 inches of penetration (or blow for penetrations less than 6 inches), date of sampling, boring number, sample number, top and bottom elevations for the sample. In addition, if jars are used, the lid should also be marked with project number, boring number, sample number, blow counts, and top and bottom elevation of the sample. Examples for labeling disturbed samples are illustrated below in Figure B-2.



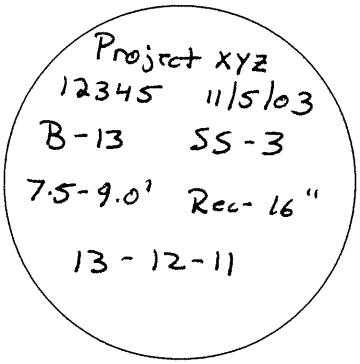


Figure B-2: Example SPT sample container label.

Representative sample portions shall be saved in sample containers for each sample interval. Within a sample interval where lenses or layers are evident, multiple samples shall be taken from a single spilt-spoon sample. Multiple samples should be noted with an alphabetic suffix on the sample number SS-2a, SS-2b, etc. The samples should be labeled in alphanumeric sequence with increasing depth. For a given sample interval, sample A should be from the top 6 inches of the sample, B from the middle, and C from the bottom 6 inches.

If the jar samples are to be temporarily stored on-site, they shall be protected from the weather, including direct sunlight, heat and freezing.

2.2.5 Cone Penetrometer Test (CPT) Procedure

CPTs are to be used as a General Type soil boring. CPTs should be done according to ASTM D 5778-12 – Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils, and the following.

<u>CPT Equipment</u>

Items to consider for CPTs, tooling, push rods, and the thrust vehicle include:

- Cones, either 10 cm^2 or 15 cm^2 Type 2 Piezocones, should have a net area ratio closest to 0.80, ratios of 0.60 to 0.90 may be acceptable for the level of the work being considered. The 10 cm^2 cone is the preferred standard per ASTM D 5778-12.
- Cones should include tip, friction, inclinometer, and pore water pressure sensors or channels. It is also recommended the cone include a temperature sensor.
- Cones shall be properly calibrated in accordance with ASTM D 5778 -12 and shall include a net area ratio curve.
- Thrust equipment minimum dead weight reaction may vary depending on the sensitivity, soil types, equipment type, and surface conditions.
- Thrust equipment shall include a hydraulic rig leveling system which is connected to a rigid frame to prevent flexure of the reaction system.
- The CPT provider must be able to meet the criteria of ASTM D 3740.

<u>CPT General Procedure</u>

The CPT rig is positioned over the location of the sounding and the leveling jacks are lowered to raise the machine mass off the rig's suspension system. The hydraulic rams of the penetrometer thrust system are set to as near vertical as possible by adjusting the leveling jacks. Once the rig is set level, the following procedures are performed:

- Power up the penetrometer tip and data acquisition system according to the manufacturer's recommendations, typically 15 to 30 minutes prior to use.
- Measure the average diameter of the tip and sleeve to be sure that the sleeve is slightly larger than the tip (approximately 0.01-inch), and both are within ASTM D 5778-12.
- CPT cones will be de-aired on the bench of the truck vertically, and the porous element will be assembled under the de-airing fluid without applying a vacuum (standard method).

- Obtain initial zero readings for the cone in an unloaded condition at a temperature as close as possible to existing ground conditions.
- Record on the project data sheet other pertinent information such as Project Number, Date and Time, Cone Identification, and starting point of each test.
- CPT operator will record the initial and final zero readings on his daily logs/data sheet for each sounding. The percent change should be recorded for the tip, sleeve, and the pressure transducers.
- Set steel casing, as needed, for cone rod support.
- Advance the cone into the soil at a rate of approximately 2 centimeters per second. The depth, tip resistance, sleeve friction, and probe inclination are recorded at 2-centimeter long intervals.
- During the progress of sounding, monitor tip and sleeve forces continuously for signs of proper operation. It is helpful to monitor other indicators such as ram pressure or probe inclination to ensure that damage will not occur if highly resistant layers or obstructions are encountered. Probe inclination is a particularly useful indicator of imminent danger to the system.
- Soundings should be advanced to the planned tip depth or refusal.
- Pore pressure dissipation tests should be obtained at the direction of the Geotechnical Engineer.
- At the end of a sounding, extract the penetrometer tip, obtain a final set of zero readings from the unloaded cone prior to cleaning, and check them against the initial zero readings. Record initial and final baselines on all documents related to the sounding.
- Inspect the cone assembly after each push for damage to the components or seals and replace parts as needed. Clean the cone per manufacturer's recommendations.
- Raw CPT data, the Operator's daily log, preliminary processed data and CPT plots will be available, and submitted weekly to CPRA.
- If CPTs are being conducted in conjunction with a "calibration soil boring," the CPT shall be conducted prior to the Undisturbed type soil boring.
- When CPTs are included in a subsurface investigation, calibration soil borings should be performed for every approximately 10 CPTs, or based on engineering judgment.

<u>CPT Quality</u>

The CPT data collection operator shall be directly supervised by a qualified engineer, geologist, or geoprofessional. Readings from the cone penetrometer are recorded in digital format with a digital data acquisition system. The calibration information is stored in the cone. This information is read from the cone before each test and recorded in the data files. Zero readings are recorded before and after each test automatically and stored in the data file. The cone should be checked for cross-talk between tip and the friction sleeve prior to starting the test and at the test completion. Zero and final readings should be recorded by the operator in his log as well. The raw data files should include the calculated initial / final baseline percent difference, which should be less than 2 percent for the tip, sleeve, and the pressure transducers. The cone and sleeve device units will be cleaned after each sounding. No "rotating wheel on the CPT rod" shall be used for the CPT depth encoder.

On a regular basis, the field data are transmitted electronically or by overnight mail to the appropriate office, where it is processed, reviewed, and finalized. The original unprocessed data is stored on large capacity, limited access storage medium where it is kept indefinitely for future reference as confidential records.

The integrity of the measurements shall be checked and verified to ensure that the logs generated are as accurate as possible. Rod spikes, which are generated naturally when the pushing is stopped such as when adding rods while advancing the sounding, are identified and edited out. These are displayed as negative spikes on the field generated CPT plot and when left uncorrected, may affect data integrity when further processing is done on the data.

The edited CPT plot is examined to ensure there are no problems with the data, such as negative values for tip or friction, cross talk between the tip and friction, or friction ratios which are excessive.

Prior to the release of the final report, the entire set of data will be reviewed by the CPT Manager. In this process, the reviewer conducts a thorough assessment of the data set checking its consistency and accuracy.

<u>CPT Data Presentation and Interpretation</u>

Data developed from a CPT shall follow ASTM D 5778 and include the following information:

- Ground surface elevation
- Raw tabular data including depth, cone tip resistance (q_c, psf), friction (f_s, psf), pore pressure (u_2, psf), inclinometer (x, y, feet), and temperature in electronic format.
- Calibration information
- Cone type and size
- Net area ratio
- Initial and final baselines, including percent difference calculation for the tip, sleeve, and pore pressure transducers
- Interpreted data files utilizing programs such as CONEPLOT, RAPID CPT, or others.
- Undrained shear strengths (psf) may be determined by N_U, N_C, N_K, or the N_{KT} methods, see Table B-2, based on engineering judgment
- Drained shear strengths shall be interpreted by engineering judgment
- Data plots including corrected total cone tip resistance($q_t = q_c + u_2(1-a)$), frictional sleeve resistance (f_s), pore pressure (u_2), friction ratio (FR%), and with interpreted soil classifications based on methods such as those illustrated by Robertson et. al (1986) for non-normalized behavior or Robertson et. al (1990) for normalized soil behavior
- CPT soil classifications shall be interpreted with USCS classifications based on ASTM D 2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) and ASTM D 2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) at paired CPT/Boring calibration locations

Table B-2: Undrained Shear Strength from CPT Data*.

Method	Undrained Shear Strength (psf)
N_U	$S_U = (u_2 - u_o) / N_U$
N_{KT}	$S_U = (q_t - \sigma_{vo}) / N_{KT}$
N_K	$S_U = (q_C - \sigma_{vo}) / N_K$
N_C	$S_U = q_t / N_C$

Note: * Equations as shown in the RapidCPT Manual. Where σ_{vo} is the total overburden.

2.2.6 Borehole Abandonment

Boreholes, CPTs, auger holes, and/or vane shear tests shall be grouted full-depth in accordance with the State of Louisiana regulations or these recommendations, whichever is more stringent. Boreholes should be grouted with 24 hours of completion, with the top three feet backfilled with native soil cuttings. Grout backfill shall be tremie grouted and should consist of a 3 to 8 percent (by weight) cement-bentonite grout. Listed in Table B-3 are typical cement-bentonite mixes.

Table B-3: Suggested One Sack Cement-Bentonite Grout Mixes

Mix (% bentonite by weight)	Bentonite (lbs)	Water (Gallons)	Grout Weight (lbs/ft^3)	Yield per Sack (ft^3)
3	2.0	7.0	14.4	1.45
4	3.8	7.8	14.1	1.54
5	4.7	8.5	13.8	1.64
6	5.6	9.1	13.5	1.74
7	6.6	9.8	13.3	1.83
8	7.5	10.5	13.1	1.93

2.3 Laboratory Requirements

At a minimum, laboratories will be required to meet ASTM E329 Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection, which among other items states in Section 9.4.1 that a lab shall demonstrate evidence of meeting requirements set forth in Specification ASTM E329 through accreditation in the field of its operation by a recognized accreditation authority. For example AASHTO Materials Reference Laboratory (AMRL) accreditation for the specified laboratory tests would meet this requirement. CPRA prefers USACE validated laboratories be used for testing in accordance with ER 1110-1-261 and ER 1110-1-8100.

2.4 Laboratory Moisture/Classification Log

All samples are to be classified in accordance with the Unified Soil Classification System (USCS) classification system following ASTM D 2487-11 and D 2488-09a as applicable. At a minimum, the moisture content of each sample shall be determined. Samples are considered to be in 1-foot intervals for cohesive materials from Shelby Tube samplers and a maximum of 18-inch intervals for SPT samples. Moisture content tests shall be in accordance with ASTM D 2216-10 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. This data is recorded on a laboratory moisture/classification log sheet.

2.5 Logging and Reporting of Soil Borings, CPT Logs, and Lab Tests

The results of field soil borings and laboratory tests will be shown and furnished by the geotechnical contractor. The borings will be reported and furnished as a stratified plotted boring log. In addition, the logs will be furnished and labeled with the project number and boring type characters as specified on the proposed boring plan. For example, soil borings, CPT's, and vibracores taken for project number CS-95 should be labeled CS95B-1 for soil borings, CS95C-1 for CPT's, and CS95V-1 for vibracores. The original "field log" for each boring will be recorded by the field logger, maintained by the geotechnical contractor as needed for analysis purposes. All strength data shall be reported in pounds per square foot (psf).

2.6 Laboratory Testing Frequency and Methods

Laboratory testing is anticipated to be assigned at the following frequency, pending actual materials encountered and engineering judgment:

Table B-4: Laboratory Testing Frequency and Methods.

Test	Frequency	Method	Comments
Moisture Content and USCS Classification	On all samples.	ASTM D 2216-10, D 2487-11, D 2488-09a	None
Atterberg Limits	On representative samples of foundations soils, borrow areas, and for every consolidation, UCT, UU, and unit weight test specimen.	ASTM D 4318-10	Method B
Unconfined Compression Tests (UCT)	On representative samples of ECD foundation soils from Undisturbed or General type soil borings. Minimum of 1 every 5 feet of drilling, limited to the upper 10 feet of foundations soils.	ASTM D 2166 / D 2166M-10	1.4-inch, 2.8-inch diameter specimens; trimmed from the center of 3-inch samples. Specimens shall be sheared to reach a constant axial load at 15 percent strain, otherwise shall be sheared to 20 percent strain, per EM 1110-2-1906.
Unconsolidated Undrained (UU or Q) Triaxial Test	On representative samples of ECD foundation soils from Undisturbed or General type soil borings. Minimum of 1 every 5 feet of drilling.	ASTM D 2850-03a (2007)	1.4-inch, 2.8-inch diameter specimens; trimmed from the center of 3-inch samples. Specimens shall be sheared to reach a constant axial load at 15 percent strain,

Test	Frequency	Method	Comments
			otherwise shall be sheared to 20 percent strain, per EM 1110-2-1906.
Consolidation Test	On representative samples of foundation soils from Undisturbed soil borings. Assigned considering project, loads, primary consolidation zone, and engineering judgment. Pending the size of the project and the material within the zone of consolidation, the number of tests shall be enough to adequately characterize the consolidation properties for settlement evaluation.	ASTM D 2435-11	Test Method B Minimum 2.0-inch diameter specimens, and Minimum D/H ≥ 2.5
Low Stress Consolidation Test for Marsh Fill Material	On representative/composite borrow area samples which should consider; borrow area orientation, dredging sequence, depth of cut, and soil variability.	Different from regular Oedometer tests for stiffer soil in terms of initial sample preparation, low starting load and size of load increments. This procedure is a combination of the USACE Method presented in EM-1110-2-5027 and the standard one-dimensional consolidation test ASTM-2435. This method was presented in Pedersen, R.C. (2001). Model Offshore Soil Deposit: Design, Preparation, and Characterization. M.S. Thesis, University of Texas at Austin.	None

Test	Frequency	Method	Comments
Settling Column Test for Marsh Fill Material	On representative/composite borrow area samples which should consider; borrow area orientation, dredging sequence, depth of cut, and soil variability.	USACE Method presented in EM-1110-2-5027	None
Gradation/ Hydrometer	On representative samples of borrow area. If any gradation does not obtain D_{10} size, a hydrometer test will need to be performed.	ASTM D 421-85 (Reapproved 2007) and D 422-63 (Reapproved 2007)	None
P_{200} Test	On representative samples of foundation and borrow area soils.	ASTM D 1140-00 (Reapproved 2006)	None
Organic Content	On representative cohesive samples with organics. All materials classified as OH, OL or Pt should be tested for organics.	ASTM D 2974-13	None
Unit Weight Tests	Test is included as part of consolidation, UCT, and UU tests. Borrow Area.	ASTM D 7263-09	Included with consolidation, UCTs, and UUs.
Specific Gravity	On each consolidation Test. Borrow Area.	ASTM D 854-14	None
Salinity and Sulfate Tests	As appropriate for Borrow Area Soil Borings.	ASTM D 4542 (Salinity)	None

2.7 Shear Strengths and Consolidation Data

2.7.1 Unconfined Compression Tests

Unconfined compression tests (UCTs) samples obtained from 3-inch diameter tubes are allowed. Specimens with 1.4-inch or 2.8-inch diameter shall have a height to diameter ratio between 2.0 to 2.5. Specimens should be trimmed from the center of the sample for testing, in accordance with ASTM and USACE guidelines. This practice will help minimize specimen disturbance. It is recommended EM 1110-2-1906 be followed for specimen shearing (i.e. samples are sheared to reach a constant axial load or 20 percent strain).

2.7.2 Triaxial Compression Tests

For triaxial shear tests, samples obtained from 3-inch diameter tubes are allowed. Specimens with 1.4-inch or 2.8-inch diameter shall have a height to diameter ratio between 2.0 to 2.5. Specimens should be trimmed from the center of the sample for testing, in accordance with ASTM and USACE guidelines. The specimens shall be tested under a confining pressure with the maximum confining pressure at least equal to the maximum normal pressure expected in the field with the project in place. The confining pressure may also be considered based on the overburden stress, the existing pore pressures, the consolidation state such as under-, normally, and over-consolidated, and the anticipated future load or stress increase. These confining pressures should be selected by the geotechnical engineer based on experience and engineering judgment. Triaxial shear tests shall be run for unconsolidated-undrained (UU) or Q-test stress state. A minimum of 1-point, or 1 specimen is recommended to estimate the shear strength envelope.

It is recommended EM 1110-2-1906 be followed for specimen shearing where samples are sheared at a rate of 1.0 percent per minute until a constant axial load is reached or 20 percent strain.

2.7.3 Low Stress Consolidation Tests for Marsh Fill Material

This test is performed on a representative composite borrow area specimen(s) prepared from borrow area borings (the composite specimen is also used for the settling column test). The low stress consolidation test is different from regular Oedometer tests for stiffer soil in terms of initial sample preparation, low starting load, and size of load increments. The test procedure is a combination of the USACE Method presented in EM-1110-2-5027 and the standard one-dimensional consolidation test ASTM-2435. The composite sample is diluted to 3 to 4 times the liquid limit to prepare free flowing slurry. The specimen should be loaded in the following increments; 0.001, 0.005, 0.01, 0.025, 0.05, 0.1, 0.25 and 0.5 tons per square foot (tsf) unless otherwise specified. Complete details of the recommended test procedures can be found in EM-1110-2-5027 Appendix D.

2.7.4 Settling Column Test for Marsh Fill Material

This test is performed on a representative composite borrow area specimen(s) prepared from borrow area borings. The sample preparation and test procedure is presented in USACE EM-1110-2-5027. The test is performed to predict the settling behavior of the marsh fill placed within the marsh creation area. Figure B-3 shows the typical settling column apparatus and an interface height versus time plot that is generated as an output. First, pilot testing should be run in order to determine the ideal slurry concentration. It is recommended that the full size column testing is performed on two slurry concentrations (High and Low) for each composite sample. This data should then be used to predict the initial void ratios and subsequent fill settlement.

Settling column data should also be utilized to calculate the bulking factor and the cut to fill ratio. The sediment losses during dredging and fill placement should be included in these calculations. USACE EM-1110-2-5025 provides guidance on dredging and dredge material disposal.

In order to prepare a composite sample, certain criteria should be followed which closely matches the predicted dredging activities during construction. The criteria may include things like borrow

area orientation, dredging sequence, depth of cut (including total allowed cut), types of soil and variability of soil both horizontally and vertically.

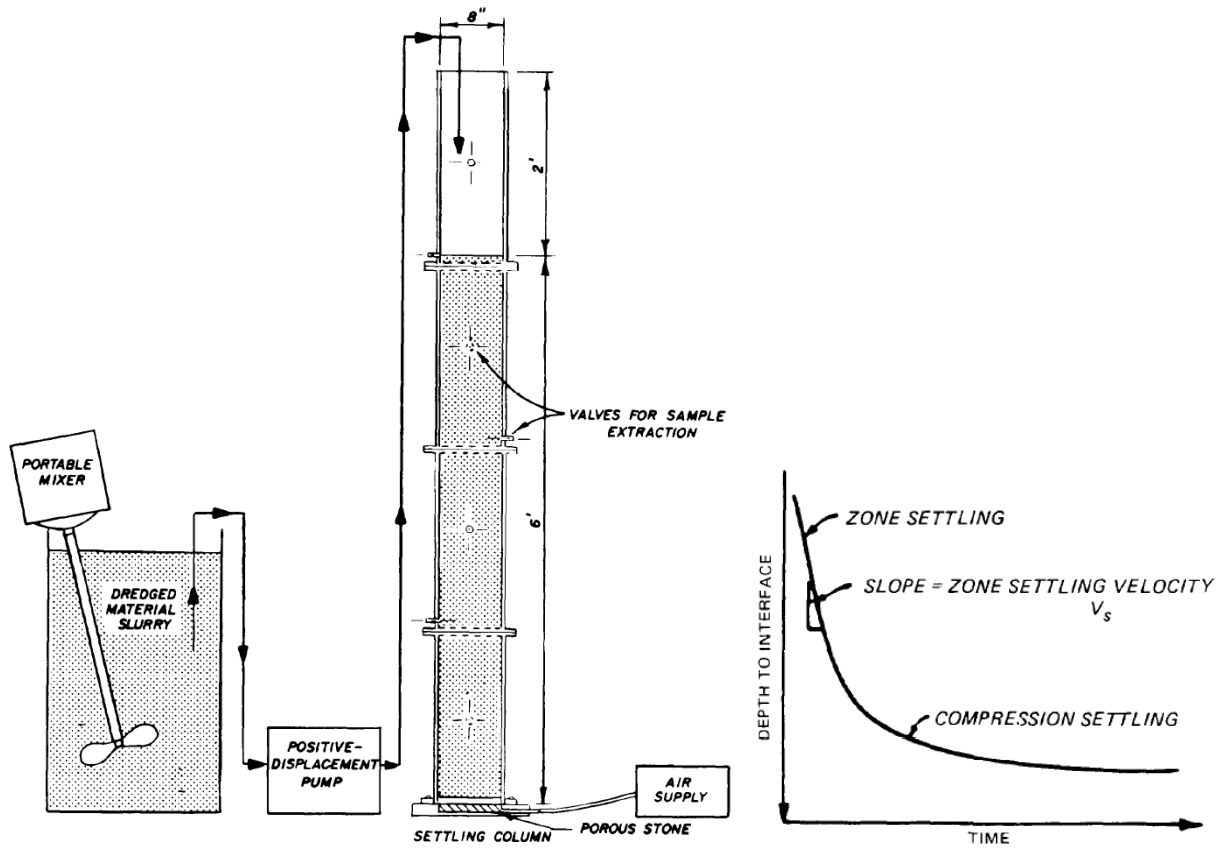


Figure B-3: Settling Column Apparatus and Conceptual Plot of Interface Height vs. Time (USACE EM-1110-2-5027, Fig-3-3 & 3-4, pg. 3-14 & 3-15).

2.7.5 Consolidation Tests

Specimens trimmed from 3-inch thin-walled tube samples may be used. Minimum specimen size of 2.0-inches may be used provided the diameter to height ratio is greater or equal to 2.5. Consolidation tests shall follow ASTM D 2435-11 Test Method B. The specimen should be loaded in the following increments; 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, and 16.0 tons per square foot (tsf) unless otherwise specified. Deformation readings should be recorded at increments of 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0 and 30.0 minutes, and at 1, 2, 4, 8 and 24 hours. If primary consolidation has not completed within the first 24 hours of testing, testing will be extended in 24 hour increments until primary consolidation has completed.

EM 1110-2-1906 provides guidance for the estimation of the preconsolidation pressure (P'c) and EM 1110-1-1904 provides guidance for the compression index (Cc), and the recompression index (Cr). EM 1110-1-1904 Chapter 3, including Figure 3-12 and Table 3-6, and NAVFAC (1986) Figure 4 page 7.1-144, shown in these Geotechnical Standards in Figure B-4, may be used for laboratory test data interpretations. It is recommended the interpretation of the preconsolidation pressure,

the compression and recompression indices be based on the Casagrande graphical construction method. Additional methods are available to estimate the preconsolidation pressure and compression indices and may be used based on engineering judgment and data correlations with soil index properties. Differences in P'c, Cc, Cr, may arise between software based interpreted values within commercially available consolidation test software and the Casagrande graphical procedure. Therefore, the consolidation test data shall be evaluated and interpreted by the geotechnical engineer. Regardless, design calculations shall clearly document the methodologies of data interpretation.

EM 1110-2-1906 does not provide guidance for estimation of the effective overburden pressure (σv') or the coefficient of consolidation (Cv). The effective overburden pressure is typically calculated from the basic soil mechanics principles of effective stress.

The coefficient of consolidation may be interpreted from the laboratory consolidation test data within the range of anticipated design pressures. However, the time rate of settlement in the field can often be orders of magnitude different than estimated from laboratory tests. Experience in Coastal Louisiana suggests the rate of settlement in the field is often quicker than calculated by using the laboratory estimated coefficient of consolidation. This is typically because of the non-homogeneity of the in-situ soils in comparison to the laboratory sample, which allows for quicker pore pressure dissipation or because shorter drainage paths exist in-situ. The Naval Facilities Engineering Command (NAVFAC) Design Manual 7.01 provides a figure (see Figure B-4) to estimate the coefficient of consolidation based on the soil's liquid limit, which has been used in local practice.

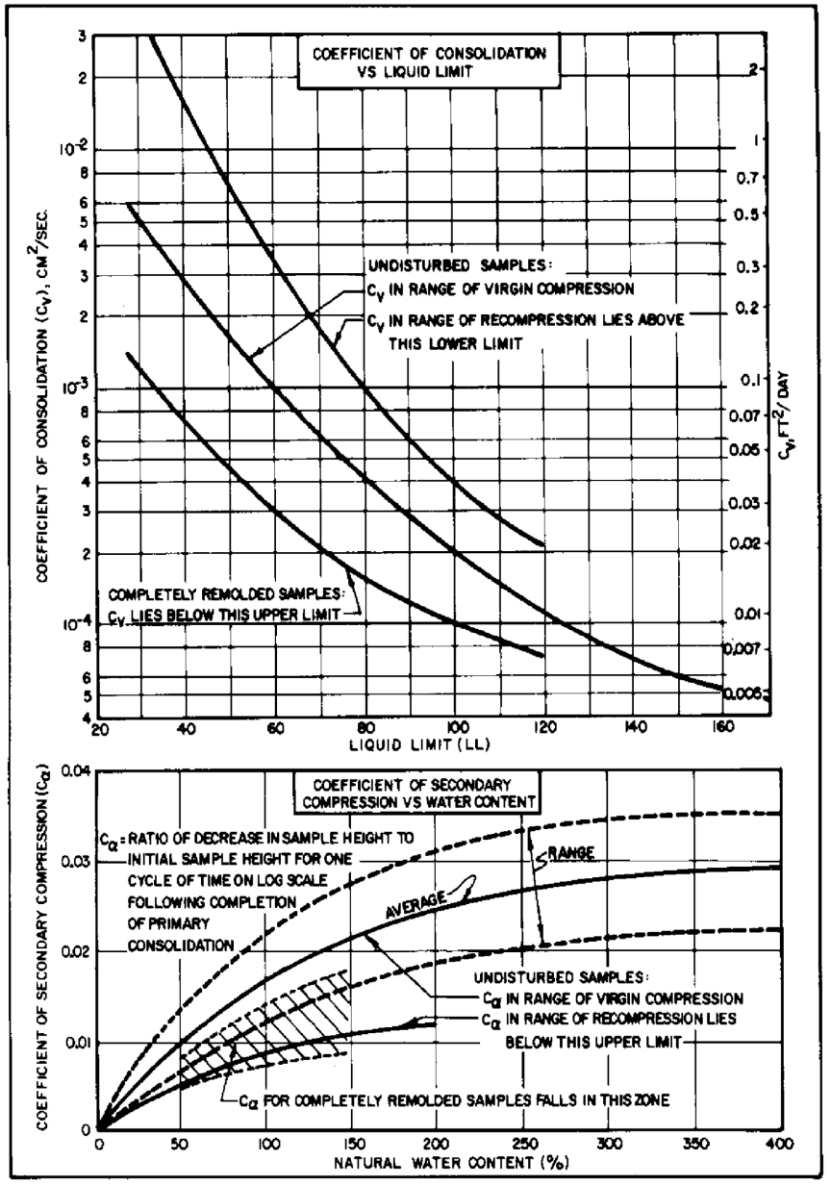


Figure B-4: Correlations for Consolidation Characteristics of Silts and Clays (NAVFAC DM 7.01, Fig-4, pg. 7.1-144).

3.0 ECD Design and Design Parameters

3.1 ECD Geometry

A minmum 5-foot crest width, minimum 1V:4H side slopes and a minimum 20 feet bench offset from the edge of borrow area are required. The ECD crest elevation varies depending on the construction marsh fill elevations and foundation soil behavior. Typical values of the ECD and borrow area are presented in Table B-5. However, the final ECD template shall be determined by analyzing the section for slope stability for various cases that have been outlined in Figure B-5.

EARTHEN CONTAINMENT DIKE (ECD) AND BORROW AREA GEOMETRIES TABLE											
			ECD				ECD BORROW			ECD DESIGN	
ECD #	MC AREA #	CMF EL. (FT.)	CREST WIDTH W (FT.) (5 ft. min.)	CREST EL. H (FT.) (1 ft. to 2 ft. min. freeboard above Max CMF EL.)	SIDE SLOPES $1:X_1$ (X_1 = 4 to 6)	BENCH OFFSET B (FT.) (20 ft. min.)	BOTTOM WIDTH (FT.) (varies)	MAX. CUT EL. C (FT.) (Typ. -8.0 to -10.0 ft.)	SIDE SLOPES $1:X_2$ (X_2 = 2 to 4)	CASE NO.	Stability Analyses FOS (FOS min=1.20)

Table B-5: ECD and borrow area geometries table for stability analyses. Typical and minimums values are shown in parentheses (This is a typical summary table for the GER)

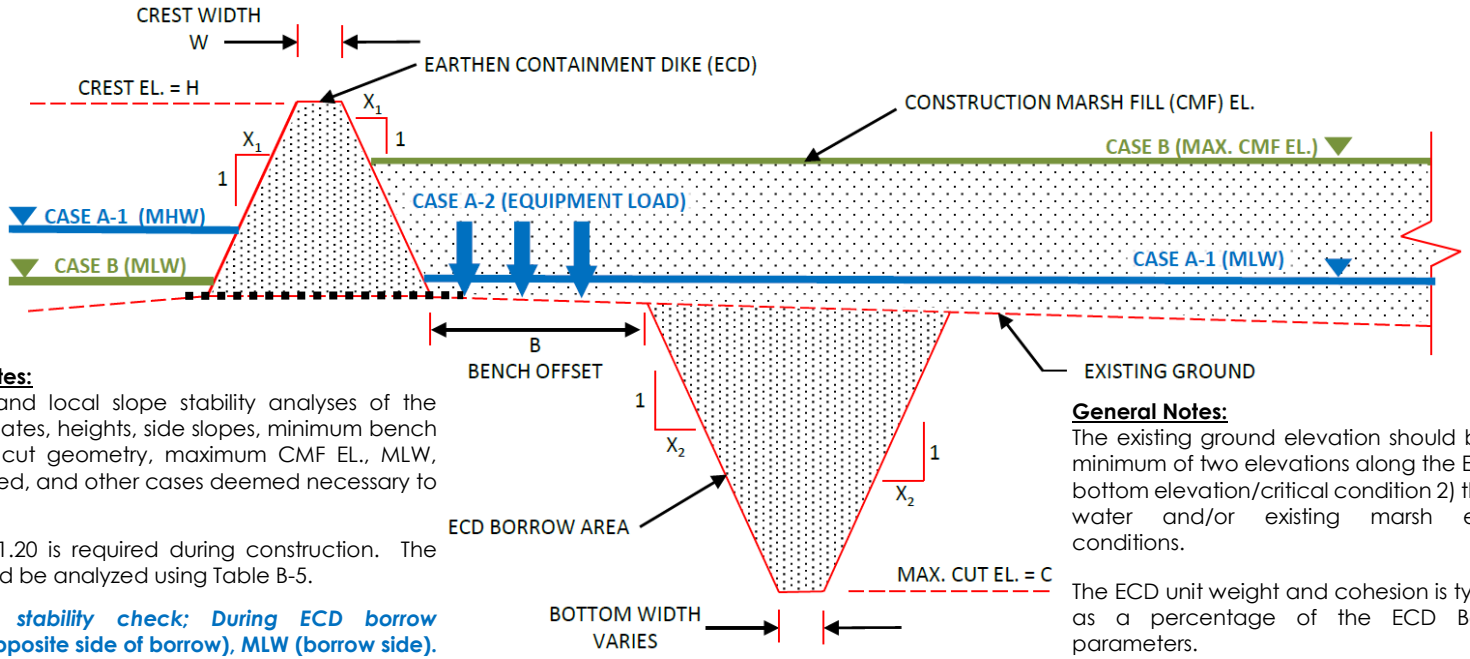


Figure B-5: Typical ECD Template.

Stability Analyses Notes:

Conduct a global and local slope stability analyses of the proposed ECD templates, heights, side slopes, minimum bench offset, borrow area cut geometry, maximum CMF EL., MLW, multi-lift CMF if required, and other cases deemed necessary to ensure ECD stability.

A minimum FOS of 1.20 is required during construction. The following cases should be analyzed using Table B-5.

CASE A-1: Global stability check; During ECD borrow excavation; MHW (opposite side of borrow), MLW (borrow side).

CASE A-2: Local stability check; During ECD borrow excavation; Distributed load from excavation equipment, MLW (borrow side).

CASE B: Dredged Material placed to CMF EL.; CMF (max. elevation), MLW (opposite side of borrow).

General Notes:

The existing ground elevation should be analyzed at a minimum of two elevations along the ECD; 1) the lowest bottom elevation/critical condition 2) the average open water and/or existing marsh elevation/general conditions.

The ECD unit weight and cohesion is typically expressed as a percentage of the ECD Borrow Area soil parameters.

A distributed load of 260 psf is typically used based on large marsh hoe/marsh buggy equipment. The ECD is constructed in several lifts.

A geosynthetic reinforcement fabric may be utilized to achieve the minimum FOS.

3.2 Typical ECD Fill Parameters

Table B-6: Typical ECD Soil Parameters

Soil Type	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle, ϕ (deg.)
*Uncompacted Clay (CH, CL))	80-100	100-200	0
*Uncompacted Organic Clay & Peat (OH, Pt)	50-80	60-100	0

*Note: The ECD Unit Weight and Cohesion are typically expressed as a percentage of the ECD Borrow Area soil parameters.

Table B-7: Typical Values for Silts, Sands, and Riprap Parameters (HSDRRSDG 2012)*.

Soil Type (per USCS)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle, ϕ (deg.)
Silt (undrained)	117	200	15
Silty Sand	122	0	30
Poorly graded Sand	122	0	33
Riprap	132	0	40

*Note: Typical values Taken from HSDRRSDG (2012) Table 3.3

Design values may vary from the typical values when site specific information is available. ECD fill properties are typically expressed as a percentage of the borrow area soils and requires engineering judgment. These values will be further refined after completion of the ongoing research.

3.3 Shear Strength Lines

Shear strength lines should be developed and plotted for all applicable cross sections and/or soil reaches along the ECD alignment. Shear strength lines should be based on the results of current or new subsurface data, but should consider any available historic soil information previously used for stability analysis.

A line indicating the ratio of cohesion (undrained shear strength) to effective overburden pressure (C/P') of 0.22 to 0.24, historically observed in Coastal Louisiana, should also be superimposed on the shear strength plots. Calculate the change in stress with depth using an appropriate method.

Shear strength lines should be plotted by elevation and include at a minimum the following:

- Design shear strength lines (psf);
- Historic shear strength lines (if applicable) (psf);
- Ground water elevation;
- C/P' line = 0.17 and 0.22 (psf);

- UU (solid symbols) and UC (hollow symbols) test results (psf);
- Moisture contents;
- Unit weights (pcf);
- Correlated CPT shear strengths (psf);
- Generalized stratigraphy (USCS); and
- Corresponding friction angles for sand and silt layers.

The design shear strength line (psf) generally should be selected considering laboratory shear strengths, CPT shear strength estimates, and engineering judgment.

Because of the quantity of data points, the design should color code the data points by soil boring and CPT, and present the data in 11" x 17" landscape format. An example Shear Line plot is illustrated in Figure B-6.

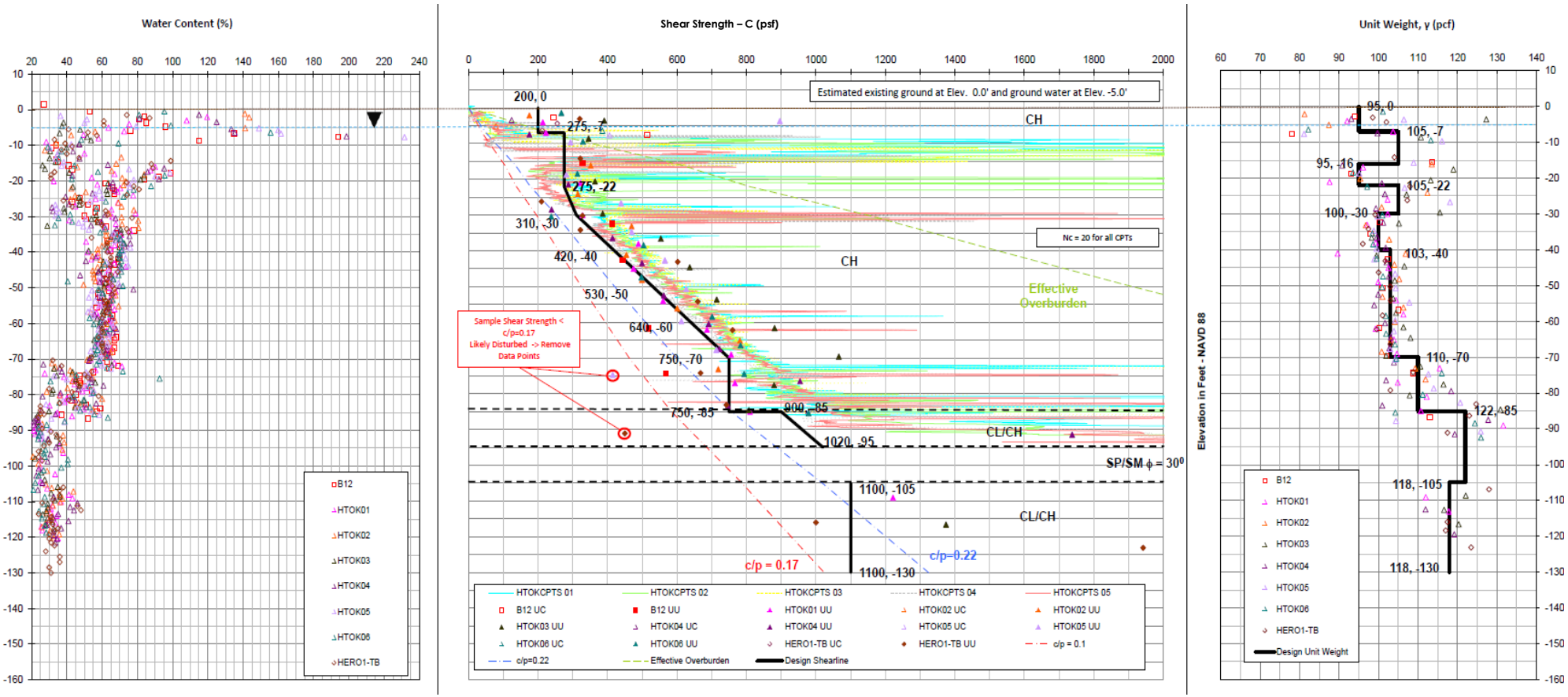


Figure B-6: Example Design Shear Strength Line Plot.

Undrained shear strengths may be estimated from CPT data by one of the methods presented in Table B-2 of these guidelines.

Typically, without any calibration, N_c values of 18 to 21 are used for southern Louisiana soils (Brandon et. al., 2010). However, when a CPT is done near the location of an undisturbed boring, calibration of the undrained shear strength is possible. The undrained shear strengths from the undisturbed borings can be compared with the measured CPT tip resistance, so a "calibrated" N_c value can be back calculated. A fair amount of scatter is possible with this approach, but overall selected N_c values below 10 and above 30 are not typically considered.

3.4 ECD Stability Factors of Safety (FOS)

ECD should be checked for a global and local slope stability failure with various heights, side slopes, minimum bench offset, borrow area cut geometry, maximum CMF elevation, MLW, multi-lift CMF if required, and other cases deemed necessary to ensure stability. The existing ground elevation should be analyzed at a minimum of two elevations along the ECD; one that represents the lowest bottom elevation/critical condition and the other that represents the average open water and/or existing marsh elevation/general conditions. The ECD unit weight and cohesion are typically expressed as a percentage of the ECD Borrow Area soil parameters. Based on a large marsh hoe/marsh buggy equipment during construction, a distributed load of 260 psf is typically used in Case A-2 and placed on the bench offset location. The ECD is constructed in several lifts to account for strength gain and constructability sequencing. A geosynthetic reinforcement fabric may also be utilized to achieve the minimum FOS.

Based on background research and stability evaluations, the minimum slope stability FOS's (during construction), the water level conditions, and the analysis cases listed in Table B-8 are recommended for use on Coastal Louisiana marsh creation and restoration projects.

Table B-8: Recommended Design Slope Stability Factors of Safety.

Case[1]	Water Level and Site Conditions	Required Min. FOS[2]
CASE A-1: Global stability check (Left to Right)	During ECD borrow excavation; MHW (opposite side of borrow), MLW (borrow side)	1.2
CASE A-2: Local stability check (Borrow area stability)	During ECD borrow excavation; Distributed load from excavation equipment, MLW (borrow side).	1.2
CASE B: Global stability check (Right to Left)	Dredged Material placed to CMF EL.; CMF (max. elevation), MLW (opposite side of borrow).	1.2

Notes: [1] Refer to Figure B-5.

[2] Spencer's Method used for analysis.

3.5 Consolidation Settlement

3.5.1 Stress Estimation Theory and Methodology

Consolidation settlement analysis of an ECD, a dune, a breakwater, or other proposed restoration feature, requires an estimate of the existing effective stress and proposed effective stress increase. In the case of multiple lifts, the use of superposition to estimate stresses is often used to account for the load shape and magnitude. Otherwise, overestimation of the stress increase could occur for various lifts.

Common methods used to estimate the stress in a soil mass in the form of 2-D or 3-D solutions include: the 2:1 Method, Boussinesq, and Westergaard theories. Further discussion on these stress estimation theory's and methods are available in soil mechanics publications, and in EM 1110-1-1904 Appendix C. More advanced methods or software such as finite element may better account for untypical load shapes.

The effective stress and stress increase estimation should be based on engineering judgment and experience, and will not be addressed in detail within these guidelines. Stress changes can also be estimated with commercially available software, such as USACE's CSETT settlement software, and Settle3D.

Consolidation settlement analysis for the marsh fill area is a combination of marsh fill material settlement and subgrade settlement. The subgrade settlement is due to the marsh platform loading. However, settlement in the marsh fill happens when marsh fill particles go through Discrete Settling, Flocculent Settling, Zone Settling and Compression Settling. All these processes have been addressed in details in USACE EM 110-2-5027. Settlement based on stress changes within the marsh fill can be estimated with commercially available software, such as USACE's PSDDF.

3.5.2 Consolidation Settlement Calculations

Consolidation Settlement calculations shall address marsh fill settlement, immediate settlement of foundation soils, consolidation settlement of foundation soils, shrinkage and consolidation settlement of the ECD, and regional subsidence. The time rate of settlement estimate is also required for evaluation of the ECD and marsh fill design grade over the construction duration and design life of the project, respectively.

Consolidation settlement shall also be calculated for proposed restoration project features such as dunes and breakwaters, and for any other feature imposing a load on the foundation soils.

Similar to the design shear lines, consolidation parameters for each soil reach shall be developed from consolidation data and should include the following at a minimum:

- Void ratio versus elevation and/or depth;
- Compression (CC) and re-compression (CR) indices (or Ratios strain based); and,
- Overconsolidation ratios.
- Consolidation data correlations to index properties versus elevation and/or depth

Within normally consolidated zones moisture contents may be utilized to estimate the void ratio and the compression index.

In Coastal Louisiana, soils are typically normally consolidated, however based on engineering judgment and site geology, some soil profiles may need to be considered as overconsolidated, particularly in the upper zones and within the upper portions of deeper geologic formations. EM 1110-1-1904 Settlement Analysis provides settlement theory and methodologies for the calculation of settlement. Settlement calculations are typically presented as hand written calculations or in spreadsheet format, but commercially available settlement software, such as PSDDF, CSETT or Settle3D may also be used for providing output reports.

3.5.3 Settlement Curves

The Constructed Marsh Fill (CMF) Elevation design should consider the future design grade based on the marsh inundation criteria discussed in the Marsh Creation Design Guidelines, Appendix D. These criteria are based on project specific information. The marsh elevation at the end of the design life will provide guidance for determining the initial construction elevation of the marsh fill. Ideally the marsh fill total settlement calculations are run with 3 to 4 iterations and the time rate settlement data may be used to produce a graph such as the one presented in Figure B-7. If the target marsh elevation at the end of design life cannot meet the project goals, a multiple lift marsh fill schedule may be an option to the designer.

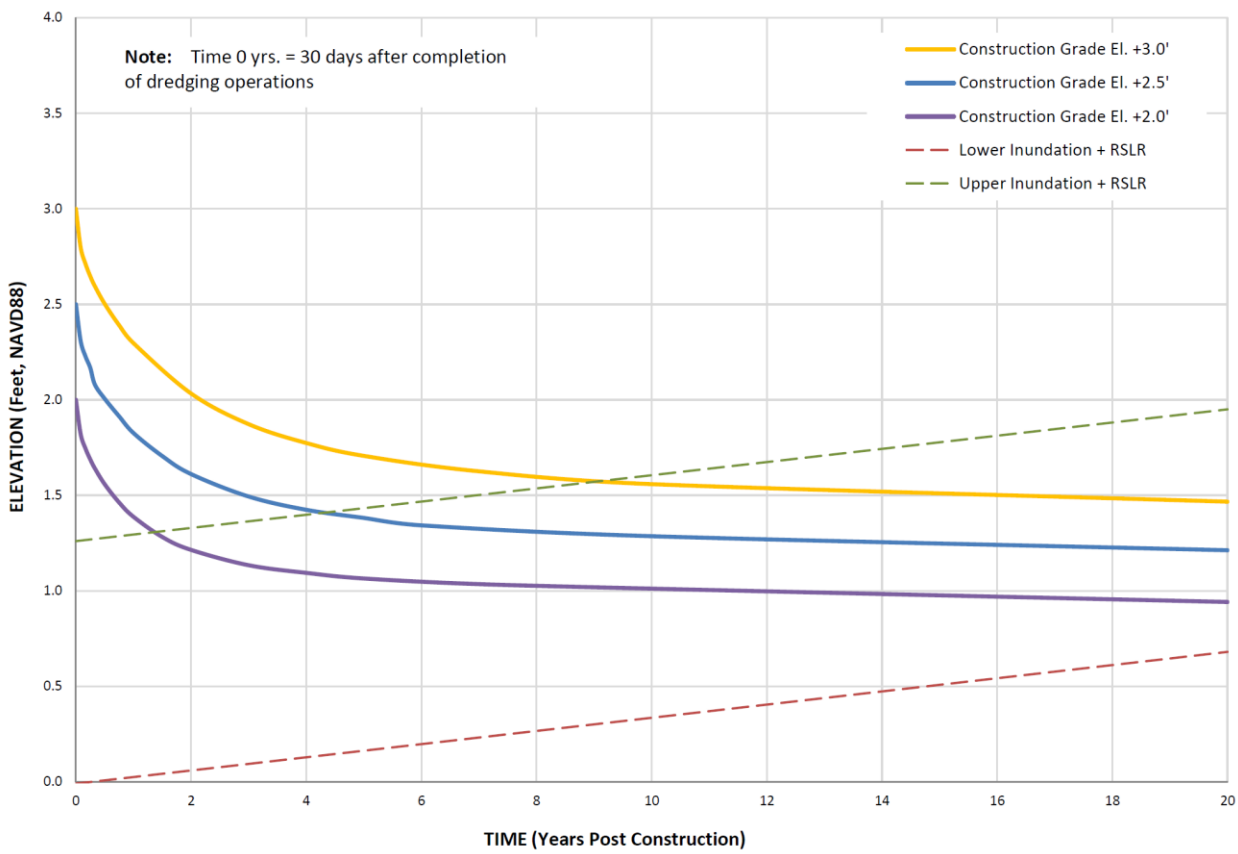


Figure B-7: Estimated Marsh Fill Total Settlement Curves.

Geotechnical instrumentation may be warranted during construction to validate the estimated consolidation parameters. This data may be utilized to adjust the estimated marsh fill schedule based on the observed actual time rate and magnitude of settlement. Additional field sampling of the dredge slurry material may also be warranted as deemed appropriate by the design engineer and geotechnical engineer.

4.0 Slope Stability Design Criteria

4.1 Slope Stability Modes of Failure

The following slope stability mode should be evaluated for an ECD.

End-of-Construction Stability Analysis:

Short-term (Undrained or End-of-Construction) stability analysis applies to the stability evaluation during construction or at the end-of-construction (EOC). Effective stress analysis with drained shear strength parameters are typically used for free-draining soils, and total stress analysis with undrained shear strength parameters are typically used for low-permeability soils. Soil with a hydraulic conductivity greater than 10^{-3} cm/s (3.3×10^{-5} ft/s) is commonly considered free-draining (Duncan and Wright 2005). Triaxial unconsolidated-undrained shear strength testing data (UU or Q test) and/or CPTs are used to estimate the total Mohr-Coulomb failure envelope, which for saturated clays is typically represented by a straight line with $\varphi = 0°$ (internal angle of friction, phi) . However, the Mohr-Coulomb envelope may have a curved portion when the soils are not saturated, as illustrated in EM 1110-2-1906 Figure 14b.

4.2 Required Factor of Safety

Experience and engineering judgment is cited for using a minimum FOS of 1.2 utilizing undrained strength and total stresses. Global stability is to be checked by Spencer's Method, a force-moment equilibrium method. Both block and circular searches with optimization should be performed to identify the critical failure surface. Optimization subdivides the traditional slip surface, circular or wedge, into a series of straight line segments. The process then starts at the entry point and randomly varies this point until the lowest FOS is found, then proceeds to adjust the next point along the straight line segments. This process is repeated until all points have been varied along the slip surface, generally resulting in a lower FOS than traditional circular or wedge surfaces. Once the critical failure surface has been identified, two stratums above and two stratums below the critical surface should be checked using the optimized block search method to demonstrate the critical failure surface has been bracketed. Both manual and automatic tension crack searches should be utilized to estimate the tension crack (if required) depth to eliminate tension from the upper slices. Regions used to model the soil strengths and unit weights, which can vary horizontally and vertically within each layer as needed.

Required FOS's for various cases, may be found in Section 3.4.

4.3 Optimization Methodology

As discussed above, critical slip surfaces should be optimized with commercially available software. The selected number of optimized surface iterations, convergence criterion, and

number of points along the critical slip surface should be selected such that the FOS is consistent toward the end of the iterations. If the FOS is still dropping, then the number of iterations or other optimization inputs may need to be adjusted to reach a consistent FOS towards the end of optimized slip surface.

During the optimization procedure, there may be a tendency to form convex surfaces along the critical failure surface. Considerable debate exits relating to the admissibility of convex slip surfaces. When performing optimization, inter-slice forces should be reviewed for unrealistic tension forces. Adjustments to the default optimization settings may be required to prevent tension from occurring in the critical slip surface.

4.4 Tension Crack Methodology

Tension cracks should be used when tension appears in the critical slip surface as determined by evaluation of the free body diagram or force polygons of individual slices. The following are ways tension may appear in the analysis:

- Negative inter-slice forces (i.e. normal side forces on the slice are not compressive, or shear side forces on the slice are negative);
- Negative total or effective normal forces on the bases of the slices;
- Line of thrust (or inter-slice forces resultant) acts outside the slice boundaries; and/or,
- The shear resistance resultant force switches direction (from a resisting to a driving force).

Some commercially available software packages may specifically indicate an error code when a tension crack may be needed. Regardless, the engineer should examine the slice forces in the output file to determine the need for a tension crack. If geotextile reinforcement is present in the ECD section, care should also be taken when using a program's automatic search for tension crack feature as this may result in a tension crack which extends through the geotextile in a reinforced ECD. As a result of a tension crack passing through the reinforcement, the fabric capacity would be ignored which is not appropriate.

EM 1110-2-1902 gives guidance on tension cracks for stability analysis. Generally, finding the appropriate tension crack depth is an iterative process of performing several analyses with various tension crack depths from relatively shallow to relatively deep. Ultimately the goal is to find the shallowest tension crack at which tensile stresses in the slices are removed from the analysis.

5.0 ECD Erosion Protection

The design of an erosion protection system for ECD's exposed to wave forces for the construction duration should be evaluated by the design engineer if deemed appropriate due to risk. The design should be based on proven recent project experience and design methodologies in the USACE Coastal Engineering Manual, or other approved and published technical document.

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ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils.

ASTM D 698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort.

ASTM D 854 - Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.

ASTM D 1140 - Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve.

ASTM D 1556 - Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method.

ASTM D 1586 - Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

ASTM D 1587 - Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes.

ASTM D 2166 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

ASTM D 2216 - Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock Mass.

ASTM D 2435 - Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.

ASTM D 2487 - Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

ASTM D 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

ASTM D 2850 - Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.

ASTM D 2974 - Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils.

ASTM D 4220 / D 4220M - Standard Practices for Preserving and Transporting Soil Samples.

ASTM D 4318 - Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

ASTM D 4380 - Standard Test Method for Density of Bentonitic Slurries.

ASTM D 4542 - Standard Test Method for Pore Water Extraction and Determination of the Soluble Salt Content of Soils by Refractometer.

ASTM D 4633 - Standard Test Method for Energy Measurement for Dynamic Penetrometers.

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