Bolivar Beach Access Improvement Phase I
24-099-012-E660

Final Report
May 2026

Prepared By:

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Project Description:
Bolivar Peninsula is 30 miles long but has limited beach access amenities. Galveston County gained ownership of hundreds of lots on the beach side of State Highway 87 because of a post-Hurricane Ike buyout program. The County-owned lots have restricted construction regulations; however, minimal construction activities are allowed. In 2019, Galveston County constructed a large restroom facility near the Noble Carl area and now many families have chosen this area for their daily beach activities. Because of the popularity of this facility, the County hopes to construct another park that is available to the public but also hurricane ready.

Subrecipient will use CMP Cycle 28 funds to design and complete the permitting for several amenities at a Bolivar Beach Park. The amenities include: a dune walkover, one mile of degraded granite trail, a permanent ADA-compliant restroom, an educational kiosk, and six picnic tables with removable shade structures. The park amenities will be hurricane ready. The restroom will meet the design standards for the gulf-facing location and the removable shade structures will be designed to withstand the everyday wind coming off the Gulf and will be removed if Galveston County is within the path of a projected storm. The Subrecipient will ensure the educational kiosk includes information on native wildlife and fish, ways to protect the Gulf of Mexico waters, and endangered species and what to do if sighted. The Project will be constructed using CMP GOMESA Cycle 29 funds.

Task 1 Summary: Survey and Design

Galveston County needed to obtain a Texas Antiquities Permit from Texas Historical Commission (THC). The County solicited engineering services through a Request for Qualifications (RFQ) in accordance with state and local standards and award a contract to the selected engineer. The engineer, Huitt Zollars completed archeological and geotechnical surveys as well as obtained a Texas Antiquities Permit, all were submitted for review and approval. Huitt Zollars began designs for a dune walkover, one mile of degraded granite trail, a permanent ADA-compliant restroom, and six picnic tables with removable shade structures. The technical specifications and 50% construction plans were submitted to the GLO for approval. While reviewing the 50% plans and the mitigation plan and after extensive discussions and careful consideration, Galveston County decided to discontinue the Bolivar Beach Pocket Park project. This decision was influenced by several unforeseen challenges, including the cost associated with elevating the restroom, the expanded footprint of the dune walkover, and the need to remove planned shade structures due to proximity restrictions near the vegetation line.

Task 2 Summary: Permits

Due to Galveston County's decision not to move forward with the Bolivar Beach Access Improvement Phase I project no permits were obtained other than the Antiquities Permit.

Task 3 Summary: Project Monitoring and Reporting

The County prepared and submitted monthly reports, deliverables, and requests for reimbursement as required in the contract, to CMPReceipts@GLO.TEXAS.GOV up to the date that the decision was made not to move forward with the project. Monthly progress reports and requests for reimbursement were due to CMPReceipts@GLO.TEXAS.GOV on the 10th day of every month of the year starting with December 10, 2024. The final report summarized the work completed under each task Antiquities Permit, Geotechnical Survey, and 50% plans submitted for the project.

Please see attached permit, survey and plans for the project.

Antiquities Permit, Geotechnical Survey, and 50% Plans

Bolivar Beach Access Improvement Phase I

24-099-012-E660

State of Texas

TEXAS ANTIQUITIES COMMITTEE

Archeology Permit # 32708

This permit is issued by the Texas Historical Commission, hereafter referred to as the Commission, represented herein by and through its duly authorized and empowered representatives.The Commission, under authority of the Texas Natural Resources Code, Title 9, Chapter 191, and subject to the conditions hereinafter set forth, grants this permit for:

Intensive Survey

To be performed on a potential or designated landmark or other public land known as:

Title: Bolivar Beach Access Improvement Project
County: Galveston
Location: 1728 Highway 87

Owned or Controlled by: (hereafter known as the Permittee):

Galveston County
722 Moody Ave
Galveston, TX 77550

Sponsored by (hereafter known as the Sponsor):

Galveston County
722 Moody Ave
Galveston, TX 77550

The Principal Investigator/Investigation Firm representing the Owner or Sponsor is:

Arlan Kalina
Cypress Environmental Consulting
10605 Grant Road
Houston, TX 77070

This permit is to be in effect for a period of:

5 Years and 0 Months

And will expire on:

1/14/2031

During the preservation, analysis, and preparation of a final report or until further notice by the Commission, artifacts, field notes, and other data gathered during the investigation will be kept temporarily at:

Cypress Environmental Consulting

Upon completion of the final permit report, the same artifacts, field notes, and other data will be placed in a permanent curatorial repository at:

Center for Archaeological Research

Scope of Work under this permit shall consist of:

An intensive pedestrian archaeological survey that meets or exceeds the State Archeological Survey Standards for Texas. This includes subsurface shovel testing of pedestrian survey transects and mechanical testing in appropriate alluvial areas. For details, see scope of work submitted with permit application.

This permit is granted on the following terms and conditions:

1. This project must be carried out in such a manner that the maximum amount of historic, scientific, archeological, and educational information will be recovered and preserved and must include the scientific, techniques for recovery, recording, preservation and analysis commonly used in archeological investigations.All survey level investigations must follow the state survey standards and the THC survey requirements established with the projects sponsor(s).
2. The Principal Investigator / Investigation Firm, serving for the Owner/ Permittee and / or the Project Sponsor, is responsible for insuring that specimens, samples, artifacts, materials and records that are collected as a result of this permit are appropriately cleaned, and cataloged for curation.These tasks will be accomplished at no charge to the Commission, and all specimens, artifacts, materials, samples, and original field notes, maps, drawings, and photographs resulting from the investigations remain the property of the State of Texas, or its political subdivision, and must be curated at a certified repository.Verification of curation by the repository is also required, and duplicate copies of any requested records shall be furnished to the Commission before any permit will be considered complete.
3. The Principal Investigator / Investigation Firm serving for the Owner/ Permittee, and / or the Project Sponsor is responsible for the publication of results of the investigations in a thorough technical report containing relevant descriptions, maps, documents, drawings, and photographs.A draft copy of the report must be submitted to the Commission for review and approval. Any changes to the draft report requested by the Commission must be made or addressed in the report, or under separate written response to the Commission. Once a draft has been approved by the Commission, one(1) printed, unbound copy and one bound copy of the final report containing at least one map with the plotted location of any and all sites recorded and two copies of the report in tagged PDF format shall be furnished to the commission. One PDF copy must include the plotted location of any and all sites recorded and the other should not include the site location data.An electronic copy of the completed Abstracts in Texas Contract Archeology Summary Form must also be submitted with the final report to the Commission.
4. If the Owner / Permittee, Project Sponsor or Principal Investigator / Investigation Firm fails to comply with any of the Commission's Rules of Practice and Procedure or with any of the specific terms of this permit, or fails to properly conduct or complete this project within the allotted time, the permit will fall into default status.A notification of Default status shall be sent to the Principal Investigator/ Investigation Firm and the Principal Investigator will not be eligible to be issued any new permits until such time that the conditions of this permit are complete or, if applicable, extended.
5. The Owner/ Permittee, Project Sponsor, and Principal Investigator/ Investigation Firm, in the conduct of the activities hereby authorizes, must comply with all laws, ordinances and regulations of the State of Texas and of its political subdivisions including, but not limited to, the Antiquities Code of Texas; they must conduct the investigation in such a manner as to afford protection to the rights of any and all lessees or easement holders or other persons having an interest in the property and they must return the property to its original condition insofar as possible, to leave it in a state which will not create hazard to life nor contribute to the deterioration of the site or adjacent lands by natural forces.
6. Any duly authorized and empowered representative of the Commission may, at any time, visit the site to inspect the fieldwork as well as the field records, materials, and specimens being recovered.
7. For reasons of site security associated with historical resources, the Project Sponsor(if not the Owner/ Permittee), Principal Investigator, Owner, and Investigation Firm shall not issue any press releases, or divulge to the news media, either directly or indirectly, information regarding the specific location of, or other information that might endanger those resources, or their associated artifacts without first consulting with the Commission and the State agency or political subdivision of the State that owns or controls the land where the resource has been discovered.
8. This permit may not be assigned by the Principal Investigator/ Investigation Firm, Owner / Permittee, or Project Sponsor in whole, or in part to any other individual, organization, or corporation not specifically mentioned in this permit without the written consent of the Commission.
9. Hold Harmless: The Owner/ Permittee hereby expressly releases the State and agrees that Owner / Permittee will hold harmless, indemnify, and defend(including reasonable attorney's fees and cost of litigation) the State, its officers, agents, and employees in their official and/or individual capacities from every liability, loss, or claim for damages to persons or property, direct or indirect of whatsoever nature arising out of, or in any way connected with, any of the activities covered under this permit.The provisions of this paragraph are solely for the benefit of the State and the Texas Historical Commission and are not intended to create or grant any rights, contractual or otherwise, to any other person or entity.
10. Addendum: The Owner/Permittee, Project Sponsor and Principal Investigator/Investigation Firm must abide by any addenda hereto attached.

Upon a finding that it is in the best interest of the State, this permit is issued on 1/14/2026

Brad Jones
Archeology Division Director

Joseph Bell
Executive Director

GEOTECHNICAL STUDY
PROPOSED DUNE WALKOVER AND FACILITIES
BOLIVAR BEACH, POCKET PARK
GALVESTON COUNTY, TEXAS

PROJECT NO. 25-578E

TO

HUITT-ZOLLARS, INC.
HOUSTON, TEXAS

BY

GEOTECH ENGINEERING AND TESTING

SERVICING

TEXAS, LOUISIANA, NEW MEXICO, OKLAHOMA, ARIZONA

www.geotecheng.com

SEPTEMBER 2025

TEXAS BOARD OF PROFESSIONAL ENGINEERS
REGISTRATION NUMBER F-001183

Huitt-Zollars, Inc.
10350 Richmond Avenue, Suite 300
Houston, Texas 77042

Project No. 25-578E
Report No. 1
Report Type: 4/12/N/D/HE/SP/FL/PL
September 10, 2025

Attention: Mr. Chad Nesvadba

GEOTECHNICAL STUDY
PROPOSED DUNE WALKOVER AND FACILITIES
BOLIVAR BEACH, POCKET PARK
GALVESTON COUNTY, TEXAS

Gentlemen:

Submitted here are the results of Geotech Engineering and Testing (GET) geotechnical study for the proposed Park Dune walkover and facilities at the above-referenced location. This study was authorized by Ms. Katherine A. Mears, P.E., with Huitt-Zollars, Inc. on August 13, 2025.

SUMMARY

- The subsoil and groundwater conditions of the project site were evaluated by conducting two (2) soil borings, Borings B-1 and B-2, at the request of the client. Results of laboratory testing data and our engineering analyses are summarized below:

  Based on Boring B-1 and B-2 the soils can be grouped into two (2) major strata with depth limits and characteristics as follows:

Stratum No.	Range of Depth, ft.	Soil Description*
I	0 – 13	SILTY SAND (SM), medium dense, dark brown, brown, light gray, with root fibers to 13', gravels
II	13 – 30	FAT CLAY (CH), very soft to stiff, dark brown, light gray, with root fibers to 15', ferrous nodules, calcareous nodules, gravels

- The project site has a potential vertical rise of 1.0-inch.
- Our short-term field exploration indicates that groundwater was encountered in Borings B-1 and B-2 at depths ranging from 8-ft to 10-ft during and after 0.33-hour of drilling. The groundwater may rise during the wet season.
- We understand that the proposed dune walkover will be supported either on drilled footings, helical piles, or a driven piles system. The decision as to what foundation type to be used should be made by the structural engineer, owner or the builder.
- If a drilled footings is used for the proposed dune walkover, then the recommendations on drilled shaft type foundation presented in Section 6.3 shall be used.
- If a helical piles system foundation is used for the proposed dune walkover, detailed information can be found in Section 6.4 of this report.
- If a driven piles (timber or prestressed concrete) system foundation is used for the proposed dune walkover, detailed information can be found in Section 6.5 of this report.
- We understand that the proposed restroom building will be supported either on helical piles system, spread footings, or a floating slab type foundation. The decision as to what foundation type to be used should be made by the structural engineer, owner, or the builder.
- If a helical piles system foundation is used for the proposed restroom building, detailed information can be found in Section 7.3 of this report.
- The spread footing type foundation should be placed at a depth of 3-ft below existing grade. The allowable net bearing capacities of the spread footing type foundation are presented in Sections 7.4 of this report.
- If a floating slab type foundation is used for the proposed restroom building, the slab should be designed in accordance with our recommendations provided in Section 7.6 of this report.
- We understand that the proposed walking loop and picnic tables are planned for this site. The proposed walking loop will be constructed with crushed granite. Our recommendations on pavement sections are provided in Section 8.0 of this report.

1.0 INTRODUCTION

It is planned to construct Park Dune walkover and facilities at Bolivar Beach, Galveston County, Texas.

The scope of the work includes evaluating the subsoil and groundwater conditions and providing recommendations for the proposed facilities. The project description is as follows:

Facility	Description
Dune Walkover	We understand that the proposed structure will be supported on drilled footings, helical piles system, or a piling type foundation.
Restroom Building	We understand that the proposed building will be supported on helical piles system, spread footings, or a floating slab type foundation.
Walking Loop and Picnic Tables	We understand that the crushed granite pavement will be used for the paving area. The paving will be subjected to pedestrian loading with limited access by light maintenance vehicles. The length of the walking loop is not known at this time.

The structural loading was not available at the time of this study. This study was performed to evaluate the subsoils and groundwater conditions and to provide recommendations for suitable foundation type, depth and allowable loading.

This report presents the results of our field explorations and laboratory testing together with recommendations for the design and construction of the proposed building. Our recommendations for pavement sections are provided in Section 7.0 of this report.

2.0 FIELD EXPLORATION

At the request of the client, the soil conditions were explored by conducting two (2) soil borings (B-1 and B-2) located approximately as shown on Plate 1. The number of borings and depths were specified by the client. The boring schedule is as follows:

Structure	Boring No.	Depth, ft
Dune Walkover	B-1	30
Restroom Building	B-2	20

In Boring B-1, soil samples were obtained continuously at boring location from the ground surface to 10-ft and at five-ft intervals thereafter to the completion depth of the boring at 30-ft. In Boring B-2, soil samples were obtained continuously at the boring location from the ground surface to 10-ft and at five-ft intervals thereafter to the completion depth of the boring at 20-ft. The cohesive soils were sampled in general accordance with the ASTM D 1587, using a Shelby tube sampler.

Cohesionless soils were generally sampled with a split-spoon sampler driven in general accordance with the Standard Penetration Test (SPT), ASTM D 1586. This test is conducted by recording the number of blows required for a 140-pound weight falling 30 inches to drive the sampler 12 inches into the soil. Driving resistance for the SPT, expressed as blows per foot of sampler resistance (N), is tabulated on the boring logs.

Soil samples were examined and classified in the field, and cohesive soil strengths were estimated using a calibrated hand penetrometer. This data, together with a classification of the soils encountered and strata limits, is presented on the log of borings, Plates 2 and 3. A key to the log terms and symbols is given on Plate 4.

The borings were drilled dry, without the aid of drilling fluids to more accurately estimate the depth to groundwater. A wet rotary technique was used to drill Borings B-1 and B-2 to the completion depth once the groundwater was encountered. Water level observations made during and after drilling are indicated at the bottom portion of the individual boring logs.

3.0 LABORATORY TESTS

3.1 General

Soil classifications and shear strengths were further evaluated by laboratory tests on representative samples of the major strata. The laboratory tests were performed in general accordance with ASTM Standards. Specifically, ASTM D 2487 is used for classification of soils for engineering purposes.

3.2 Classification Tests

As an aid to visual soil classifications, physical properties of the soils were evaluated by classification tests. These tests consisted of natural moisture content tests (ASTM D 4643), and Atterberg limit determinations (ASTM D 4318, Method B). Similarity of these properties is indicative of uniform strength and compressibility characteristics for soils of essentially the same geological origin. Results of these tests are tabulated on the boring logs at respective sample depths.

3.3 Strength Tests

Undrained shear strengths of the cohesive soils, measured in the field, were verified by calibrated hand penetrometer tests, unconfined compressive strength tests (ASTM D 2166) and torvane tests. Natural water content and dry unit weight were determined routinely for each unconfined compressive strength test. These test results are also presented on the boring logs.

3.4 Soil Sample Storage

Soil samples tested or not tested in the laboratory will be stored for a period of seven days subsequent to submittal of this report. The samples will be discarded after this period, unless we are instructed otherwise.

4.0 GENERAL SOILS AND DESIGN CONDITIONS

4.1 Site Conditions

The project site and the surrounding areas are generally flat with a topographic variation of less than three-feet. Currently, the project site is vacant and covered with grass. Project site pictures were taken during our field exploration and are presented on the cover page and Plate 5.

4.2 Soil Stratigraphy

Subsurface soil appears to be uniform across the site. Details of subsurface conditions at each boring location are presented on the respective boring logs. In general, the soil stratigraphy for the proposed facilities is shown in the following report sections:

4.2.1 Dune Walkover (Boring B-1)

Based on Boring B-1, the soils can be grouped into two (2) major strata with depth limits and characteristics as follows:

Stratum No.	Range of Depth, ft.	Soil Description*
I	0 – 13	SILTY SAND (SM), medium dense, dark brown, brown, light gray, with root fibers to 13', gravels
II	13 – 30	FAT CLAY (CH), very soft to stiff, dark brown, light gray, with root fibers to 15', ferrous nodules, calcareous nodules, gravels

* Classification in general accordance with the Unified Soil Classification System (ASTM D 2487)

4.2.2 Restroom Building Area (Boring B-2)

Based on Boring B-2, the soils can be grouped into two (2) major strata with depth limits and characteristics as follows:

Stratum No.	Range of Depth, ft.	Soil Description*
I	0 – 13	SILTY SAND (SM), medium dense, dark brown, brown, light gray, with root fibers to 13', gravels
II	13 – 20	FAT CLAY (CH), very soft, dark brown, light gray, with root fibers to 15', ferrous nodules, calcareous nodules, gravels

* Classification in general accordance with the Unified Soil Classification System (ASTM D 2487)

4.3 Soil Properties

Soil strength and index properties and how they relate to foundation design are shown in the following report section:

4.3.1 Dune Walkover (Boring B-1)

Stratum No.	Soil Type	PI(s)	SPT	Soil Expansivity	Soil Shear Strength, tsf
I	Fill: Silty Sand (SM)	–	20 – 29	Non-Expansive	–
II	Fat Clay (CH)	40	2	Expansive	0.46 – 0.62

Legend: PI = Plasticity Index
SPT = Standard Penetration Test

4.3.2 Restroom Building Area (Boring B-2)

Stratum No.	Soil Type	PI(s)	SPT	Soil Expansivity	Soil Shear Strength, tsf
I	Fill: Silty Sand (SM)	–	23 – 27	Non-Expansive	–
II	Fat Clay (CH)	67	2	Highly Expansive	–

Legend: PI = Plasticity Index
SPT = Standard Penetration Test

Legend: PI = Plasticity Index

4.4 Water-Level Measurements

The soil borings were dry augured to evaluate the presence of perched or free-water conditions. The level where free water was encountered in the open boreholes during the time of our field exploration is shown on the boring logs. Our groundwater measurements are as follows:

Boring Number(s)	Groundwater Depth, ft. at the Time of Drilling	Groundwater Depth, ft. at 0.33-Hour Later
B-1 and B-2	8	8

Fluctuations in groundwater generally occur as a function of seasonal moisture variation, temperature, groundwater withdrawal and future construction activities that may alter the surface drainage and subdrainage characteristics of this site.

An accurate evaluation of the hydrostatic water table in the relatively impermeable clay and low permeable sands/silts requires long term observation of monitoring wells and/or piezometers. It is not possible to accurately predict the pressure and/or level of groundwater that might occur based upon short-term site exploration. The installation of piezometers/monitoring wells was beyond the scope of our study. We recommend that the groundwater level be verified just before construction if any excavations such as construction of drilled footings/underground utilities, etc. are planned.

We recommend that GET be immediately notified if a noticeable change in groundwater occurs from that mentioned in our report. We would be pleased to evaluate the effect of any groundwater changes on our design and construction sections of this report.

5.0 POTENTIAL VERTICAL MOVEMENT

Vertical movement of expansive foundation soils is commonly referred to in terms of the Potential Vertical Rise (PVR, Ref. 1) that can occur due to changes in soil moisture content. Accepted methods of estimating PVR includes the use of empirical relationships and the results of laboratory Atterberg limits and moisture content tests.

We computed the Potential Vertical Rise at this site. A PVR of about 1.0-inch can be expected during the life of the structure.

6.0 DUNE WALKOVER FOUNDATION RECOMMENDATIONS

6.1 General

It is planned to construct a dune walkover structure at the above-referenced location. We understand that the dune walkover structure will be supported on either drilled footings, helical piles, or a driven piles system.

The objectives of this study were to evaluate the soil conditions and develop foundation recommendations for the proposed dune walkover including foundation type, depth, allowable drilled footings capacities, and allowable driven pile capacities. At the request of the client, one (1) soil boring (B-1) was drilled to a depth of 30-ft for conducting field and laboratory tests for the proposed dune walkover. The results of the drilling and laboratory test data were analyzed to develop foundation design criteria. Our engineering analyses and recommendations for the proposed dune walkover structure are provided in the following report sections.

6.2 Foundation Type

Foundations for the proposed dune walkover should satisfy three independent design criteria. First, the maximum design pressure exerted at the foundation level should not exceed the allowable net bearing pressure based on an adequate factor of safety with respect to soil shear strength. Secondly, the magnitude of total and differential settlements under sustained foundation loads must be such that the structure is not damaged, or its intended use impaired. Thirdly, the footings should resist uplift forces due to structural loads.

We understand that the proposed structural loads will be supported on either drilled footings, helical piles, or a driven piles system. The structural loads are not available at this time.

The use of drilled footings at this site may be expensive due to presence of cohesionless soils. **Drilled footings should be constructed, using a slurry method of construction**. This may make drilled footings more expensive than helical piles. The structural engineer may want to design the foundation system using helical piles, in addition to the drilled footings. There should be cost comparison between drilled footings, using a slurry method of construction and helical piles.

The decision as to what foundation type to be used should be made by the structural engineer, homeowner or the builder. Our recommendations for these foundation types are presented in the following report sections.

6.3 Drilled Footings Type Foundation

6.3.1 Axial Capacity of Drilled Shaft Type Foundations

We evaluated the vertical capacity of the drilled shaft for the proposed structure. Our design recommendations are independent of shaft size to permit comparison and consideration of various shaft sizes and penetrations. This data is shown on Plate 6, in the form of accumulative allowable static friction resistance (F), and allowable point bearing component (E), as a function of shaft tip penetration below the original ground level. Equations for the estimation of compression and tension capacity of the shaft and determination of dimensions are shown below the curve together with an illustrative example. The allowable shaft capacity curves include a factor of safety of 2.0 for skin friction and 3.0 with respect to end bearing.

Shaft spacing should not be less than three diameters of the shaft; otherwise, the drilled shaft capacities would require adjustment to account for group effects. It should, however, be noted that this shaft spacing is a minimum requirement and should be increased accordingly to the anticipated drilled shaft group action.

Relatively small spacings may require accounting for block-type failure mechanism, whereas individual shaft failure may occur at larger spacings. In drilled shaft groups, the zone of influence from an individual drilled shaft may intersect with other shafts depending on the shaft spacing. Such a group of drilled shafts with center-to-center spacing of less than 3 diameters of the shafts, may be considered as a block. Furthermore, the sum of the nominal bearing resistance of the shaft group is less than the sum of the individual nominal resistances of each shaft in the group.

The efficiency of a drilled shaft group is calculated by the equation below:

$$\eta_g = \frac{R_{Block}}{\sum_{i=1}^{n} R_{n,i}} \leq 1$$ ....FHWA-NHI-10-016, Equation 14.2

Where: η_g, is the efficiency of the drilled shaft group

$R_{n,i}$, is the individual drilled shaft nominal resistance.

R_{Block}, is the resistance of the block.

The nominal resistance of the block can be estimated using the equation below:

$$R_{Block} = f_{max}.\,[2.D.(Z+B)] + q_{max}.\,(Z.B)$$ ....FHWA-NHI-10-016, Equation 14.3

Where: D, Z, and B are the depth, length, and width of the block, respectively.

f_{max}, the nominal unit side resistance of the block is conservatively computed as if the peripheral surface of the block is a drilled shaft.

q_{max}, base resistance of the block computed using the appropriate procedure for cohesive materials.

Based on the field and laboratory testing data, it is our opinion that the drilled shaft should be designed and constructed as follows:

- Use a straight shaft foundation.

- Based on our current groundwater observations, the drilled shaft excavations may encounter groundwater. Any water inflow must be pumped out immediately using a sump pump. The drilling contractor must be prepared for this condition.

- Due to (a) the presence of cohesionless soils and groundwater, (b) potential seasonal variations in groundwater depth, (c) variations in the subsoils stratigraphy and strengths, and (d) corresponding potential caving problems, **a slurry method of construction will be required for the drilled shafts installations.**

Due to potential variability of the on-site soils and potential groundwater fluctuations, we recommend that the four corner piers be drilled first to better evaluate the constructability of the depth and bell to shaft ratios recommended herein. Once this information is field verified, all other piers need to be constructed accordingly.

We recommend the placement of tension steel in the drilled footings to resist uplift loads due to structural loads. The uplift capacity of the drilled footings should be calculated based on the capacity curve shown on Plate 6.

6.3.2 Lateral Capacity of Drilled Footings

We understand that the proposed structure will be subject to sustaining lateral loads. The induced lateral loads shall be resisted by the drilled footing foundation. The stress-strain response of soil in lateral loading is commonly referred to as the '*P-y*' response of the soil and is described by a load-deflection curve, where the y-axis is load '*P*' and the x-axis is deflection '*y*'. The shape of the curve varies depending on soil type and the type of loading. The '*P-y*' curves can be developed using a computer program LPILE and the soil parameters under the assumption of the groundwater table at the ground surface are presented in Plate 7.

6.3.3 Depth of Scour

Our shaft capacity curve assumes a scour depth of 5-ft. The scope of our work did not include scour analysis.

6.4 Helical Pile

The structural loads can also be supported on a helical pile system founded at a minimum depth of 30-ft below the existing grade. The helical pile system should be designed on the basis of design procedure, outlined in the "Basic Helical Screw Pile Design," (Ref. 4)

In general, the cost of Helical Pile System will be less than the cost of drilled footings, installed using casing or slurry method of construction. Furthermore, the construction time is significantly reduced. Further information on design of helical pile system can be obtained from Geotech Engineering and Testing web site (www.geotecheng.com), under "Publications, Guidelines".

The ultimate pile capacity can be computed from the following:

$P = \Sigma A_H (9c)$ Piles in Clays

$P = \Sigma A_H (qN_q)$ Piles in Sands

Where: P = Ultimate Pile Capacity, lbs
ΣA_H = Sum of Helical Plate Areas, ft^2
$q = \gamma h/2$ = Soil Overburden Pressure to Mid-Plate Depth, psf
γ = Soil Unit Weight of 60 pcf
h = Depth of Helical Piles, ft
c = Cohesion of Soil, psf
N_q = Bearing Capacity Factor for Granular Soil

A factor of safety of 2.0 is recommended in calculating ultimate helical piles capacity. We recommend that the helical plates be separated at a distance of three plate diameters. Furthermore, the structural engineer should also check for buckling, using a soil modulus of subgrade reaction (k). Buckling can usually be a problem in soft soils. One way to reduce the potential for buckling is to install the helical shafts inside a 12-inch diameter, 10-ft deep hole which is backfilled with concrete after helical pile installation. The helical pile should be placed at a distance of at least five largest plate diameters between each other to reduce group action. Pile spacing that is closer than 5 largest plate diameters will result in axial capacity reduction.

We recommend the following design parameters (Ref. 5) for this project:

c = 1,000 psf
k = 50 pci
N_q = 15

The helical pile should be designed to resist the "punch-through" failure in areas where soft soils encounter below strong soils. We recommend a distance of greater than five times the diameter of the lowest helical plate exists from the soft soils to prevent the helical piles from puncturing through into the soft soil stratum.

Some of the helical pile contractors are as follows:

Company Name	Telephone No.	Contact Person
Ram Jack Foundations	832-208-1455	Mr. Wesley Highnote
R.L. Nelson Construction Foundation Repair	713-473-2382	Ms. Ann Nelson
Rock Solid Helical Pile, LLC	713-417-9053	Mr. Ward Taylor
Du-West Foundation Repair	713-473-7156	Mr. Jim Dutton
Olshan Foundation Repair	713-213-1900	Mr. Chris Cates

Geotech Engineering and Testing does not endorse or advocate for any of these contractors. It is the responsibility of the owner or the contractors to evaluate the qualifications of these contractors.

6.5 Driven Pile Foundation

6.5.1 General

We understand that the proposed dune walkover will be supported on a driven pile foundation. Soil boring B-1 was drilled to a depth of 30-ft below the existing grade at the proposed location.

6.5.2 Axial Pile Capacity

We evaluated the axial capacity of driven piles for support of the proposed dune walkover structure. Our design recommendations are independent of pile size in order to permit comparison and consideration of various pile sizes and penetrations. The design data are presented on Plate 8 based on the results of Boring B-1 in the form of accumulative allowable unit shaft resistance, F, and allowable unit end bearing, E, as a function of pile tip penetration below the existing ground level. The pile capacity curves could be used to estimate the compression and uplift or tension capacity of the piles. Equations for the estimation of pile capacity and determination of dimensions are shown below the curve together with an illustrative example. The allowable pile capacity curves, presented on Plate 8, are referenced from the existing ground surface and include a safety factor of 2.0. Any length of pile needed above this referenced elevation should be added to the calculated lengths.

Pile spacing should not be less than three diameters of the pile; otherwise, pile capacities would require adjustment to account for group effects.

Design and Construction of Piles should be in general accordance with TxDOT Standard Specifications 404 Drilling Pile.

The pile capacity curves should be modified, depending upon where the pile is going to be located on the culvert crossing. The portion of the pile within the embankment should be subtracted from pile capacity calculations.

6.5.3 Lateral Capacity of Piles

We understand that the proposed dune walkover structure will sustain lateral (wind) loads. The induced lateral loads shall be resisted by the pile foundation. The stress-strain response of soil in lateral loading is commonly referred to as the '*P-y*' response of the soil and is described by a load-deflection curve, where the y-axis is load '*P*', and the x-axis is deflection '*y*'. The shape of the curve varies depending on soil type and the type of loading. The '*P-y*' curves and lateral load capacity can be developed using a computer program LPILE based on the design soil parameters. The design soil parameters based on the Boring B-1 drilled at this site and under the assumption of the groundwater table at the ground surface are presented on Plate 7.

6.5.4 Depth of Scour

Our pile capacity curve assumes a scour depth of 5-ft. The scope of our work did not include scour analysis.

6.6 Foundation Settlement

A settlement analysis was not within the scope of this study. It is anticipated that footings, grade beams and slabs designed using the recommended allowable bearing pressures will experience small settlements that will be within the tolerable limit for the proposed structure.

6.7 Vegetation Control

We recommend trees not be planted or left in place (existing trees) closer than half the canopy diameter of mature trees from the grade beams, typically a minimum of 20-ft. Alternatively, root barriers must be placed near the exterior grade beams to minimize tree root movements under the floor slab. This will minimize possible foundation movements as a result of tree root systems.

6.8 Foundation Maintenance

Long term performance of structures depends not only on the proper design and construction, but also on the proper foundation maintenance program.

A properly designed and constructed foundation may still experience distress from the vegetation and expansive soil which will undergo volume change when correct drainage is not established or incorrectly controlled water source, such as plumbing/sewer leaks, excessive irrigation, water ponding near the foundation becomes available.

Our general recommendations on foundation maintenance are presented in the article at the end of this report. More foundation maintenance information can be found at **Foundation Performance Association Document #FPA-SC-07-0** (Ref. 2).

6.9 Site Drainage

It is recommended that site drainage be well developed. Surface water should be directed away from the foundation soils (use a slope of about 5% in the grass within 10-ft of foundation). No ponding of surface water should be allowed near the structure.

In the event that sprinkler systems are used, we recommend that the sprinkler system be placed all around the structure to provide a uniform moisture condition throughout the year. This will reduce fluctuations in subsoil moisture and corresponding movement.

7.0 RESTROOM BUILDING FOUNDATION RECOMMENDATIONS

7.1 General

It is planned to construct a restroom building at the above-referenced location. We understand that the restroom building will be supported on either helical pile system, floating slab type foundation, or a spread footing.

The objectives of this study were to evaluate the soil conditions and develop foundation recommendations for the proposed restroom building including foundation type, depth, allowable drilled footings capacities, and allowable driven pile capacities. At the request of the client, one (1) soil boring (B-2) was drilled to a depth of 20-ft for conducting field and laboratory tests for the proposed dune walkover. The results of the drilling and laboratory test data were analyzed to develop foundation design criteria. Our engineering analyses and recommendations for the proposed dune walkover structure are provided in the following report sections.

7.2 Foundation Type

Foundations for the proposed restroom building should satisfy three independent design criteria. First, the maximum design pressure exerted at the foundation level should not exceed the allowable net bearing pressure based on an adequate factor of safety with respect to soil shear strength. Secondly, the magnitude of total and differential settlements under sustained foundation loads must be such that the structure is not damaged, or its intended use impaired. Thirdly, the footings should resist uplift forces due to structural loads.

We understand that the proposed structural loads will be supported on either deep foundations or shallow foundations. The deep foundation may consist of helical piles foundation system. The shallow foundation may consist of spread footings or a floating slab foundation. The structural loads are not available at this time.

The decision as to what foundation type to be used should be made by the structural engineer, homeowner or the builder. Our recommendations for these foundation types are presented in the following report sections.

7.3 Helical Pile

The structural loads can also be supported on a helical pile system founded at a minimum depth of 20-ft below the existing grade. The helical pile system should be designed on the basis of design procedure, outlined in the "Basic Helical Screw Pile Design," (Ref. 4)

In general, the cost of Helical Pile System will be less than the cost of drilled footings, installed using casing or slurry method of construction. Furthermore, the construction time is significantly reduced. Further information on design of helical pile system can be obtained from

Geotech Engineering and Testing web site (www.geotecheng.com), under "Publications, Guidelines".

The ultimate pile capacity can be computed from the following:

$P = \Sigma A_H (9c)$ Piles in Clays

$P = \Sigma A_H (qN_q)$ Piles in Sands

Where: P = Ultimate Pile Capacity, lbs
ΣA_H = Sum of Helical Plate Areas, ft^2
$q = \gamma h/2$ = Soil Overburden Pressure to Mid-Plate Depth, psf
γ = Soil Unit Weight of 60 pcf
h = Depth of Helical Piles, ft
c = Cohesion of Soil, psf
N_q = Bearing Capacity Factor for Granular Soil

A factor of safety of 2.0 is recommended in calculating ultimate helical piles capacity. We recommend that the helical plates be separated at a distance of three plate diameters. Furthermore, the structural engineer should also check for buckling, using a soil modulus of subgrade reaction (k). Buckling can usually be a problem in soft soils. One way to reduce the potential for buckling is to install the helical shafts inside a 12-inch diameter, 10-ft deep hole which is backfilled with concrete after helical pile installation. The helical pile should be placed at a distance of at least five largest plate diameters between each other to reduce group action. Pile spacing that is closer than 5 largest plate diameters will result in axial capacity reduction.

We recommend the following design parameters (Ref. 5) for this project:

c = 1,000 psf
k = 50 pci
N_q = 15

The helical pile should be designed to resist the "punch-through" failure in areas where soft soils encounter below strong soils. We recommend a distance of greater than five times the diameter of the lowest helical plate exists from the soft soils to prevent the helical piles from puncturing through into the soft soil stratum.

Some of the helical pile contractors are as follows:

Company Name	Telephone No.	Contact Person
Ram Jack Foundations	832-208-1455	Mr. Wesley Highnote
R.L. Nelson Construction Foundation Repair	713-473-2382	Ms. Ann Nelson
Rock Solid Helical Pile, LLC	713-417-9053	Mr. Ward Taylor
Du-West Foundation Repair	713-473-7156	Mr. Jim Dutton
Olshan Foundation Repair	713-213-1900	Mr. Chris Cates

Geotech Engineering and Testing does not endorse or advocate for any of these contractors. It is the responsibility of the owner or the contractors to evaluate the qualifications of these contractors.

7.4 Spread Footings Type Foundation

7.4.1 Allowable Bearing Pressure

Based on the results of field exploration, laboratory test data and bearing capacity theory, allowable net bearing pressures for spread footings for the proposed Restroom Building will be as follows:

Foundation Type	Minimum Depth, ft.(1),(2)	Bearing Soil Type	Allowable Net Bearing Pressures, psf (3)
Spread Footings:			
Strip	3	Loose to Medium Dense Cohesionless soils	$q = 348\,(H) + 224\,(B)$
Square or Circle	3	Loose to Medium Dense Cohesionless soils	$q = 548\,(H) + 134\,(B)$
Rectangle	3	Loose to Medium Dense Cohesionless soils	$q = 348\,(H) + 202\,(H)\,(B/L) + 224\,(B) - 80.6\,(B^2/L)$

Where: q = Allowable net bearing pressure (for dead + sustained live loading), psf.
H = Footing depth, ft.
B = Footing width, ft.
L = Footing length, ft

Notes:
1. With respect to existing grade.
2. The actual depth of the footing should be determined based on the structural loads.
3. Dead + sustained live loading.

The above equations include a factor of safety of 3.0 with respect to shearing failure. We recommend that the maximum allowable dead foundation loading (dead + sustained live load) be limited to 2,000 psf (for footing widths greater than four-feet) to limit footing settlements to tolerable levels (less than one-inch). The maximum allowable bearing pressure for dead loading should be reduced to 1,000 psf for footings with a width (B) of one-foot. The above-mentioned bearing capacities should be reduced by 50 percent if the subgrade soils become saturated or have potential for saturation. A minimum footing width of one-foot should be used in design. For total loading (dead + live load), the allowable net bearing pressures obtained from the above equations and the pressure limits should be increased by 50 percent.

Based on our current groundwater observations, footing excavations may encounter groundwater. Any water inflow must be pumped out using a sump-pump. The footing excavations should be free of loose material and water prior to concrete placements, and concrete should be poured as soon as possible.

Detailed observations of spread footing construction should be required by a qualified engineering technician to assure that the footings are (a) founded in the proper bearing stratum, (b) have the proper depth, (c) have the correct size, and (d) that all loose materials have been removed prior to concrete placement.

7.4.2 Lateral Capacity for Spread Footings

Lateral loads on spread footings may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.3 may be used between the foundation bottom or concrete slabs and the supporting soils. A factor of safety of 2.0 was used in the design. The resistance of natural soils or properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 90h pounds per square foot, where h is the depth below the top of the spread footings. A one-third increase in the passive value may be used for wind loads. The passive pressure and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance.

7.4.3 Uplift Capacity for Spread Footings

The ultimate uplift capability of a single spread footing can be estimated using the information shown on Plate 10 and the following empirical equation:

$$Q_T = W_F + W_S$$

Where: W_F = Weight of Foundation, pounds
W_S = Weight of Soil Wedge, pounds; Use Soil Unit Weight, γ' = 60 pcf
Q_T = Ultimate capacity of a footing, Pounds; Use Factor of Safety = 1.5

7.5 Floor Slabs Supported on Helical Piles or Spread Footings

The project site for the proposed restroom building has the potential for construction problems related to the surficial layer of silty sand soils. These permeable soils are underlain by relatively impermeable clays. Thus, due to poor site drainage, wet season or site geohydrology, water ponds (within the silty sand soils) on the clays and creates a "perched water table condition". The surficial silty sand soils become extremely soft when wet. Sometimes this condition may result in moisture migration such as water or vapor migration through the foundation slab. We recommend one of the following remedial measures to mitigate the problem:

1. Remove top two feet of surficial silty sand (Stratum I) soils from the floor slab areas, and five-ft beyond the building footprint and be replaced it with select structural fill in accordance with our "Site Preparation" section.

2. Leave the on-site surficial silty sand soils in place. Place a high-performance polyethylene moisture barrier over the soils to reduce the problem.

It is our opinion that alternative No. 1 will provide a lower risk of potential issues. The decision as to what measure to use should be made by the owner(s) after risks and rewards of the issues are explained to them.

We recommend that the upper eight-inch of subgrade soils in the floor slab areas be compacted to at least 95% standard Proctor density (ASTM D 698) at a moisture content within ±2% of optimum.

A bedding layer of leveling sand, one- to two-inches in thickness, may be planned under the slab for leveling purposes only. A layer of high-performance polyethylene moisture barrier should be used above the sands to prevent moisture migration through the slab.

7.6 Floating Slab Foundation

We understand that the structural loads will be supported either on a post-tensioned slab foundation (Ref. 6) or a conventionally reinforced slab (Ref. 6). The subsoils at this site is considered to be non-expansive so the type II lightly reinforced slab (Ref. 7) can be used at this site. The design methods developed in PTI Design Manual (Ref. 8) for slabs are applicable for subgrade with Plasticity Index (PI) equal or greater than 15 in the upper five-feet. Therefore, the post-tensioned slab design in this project may not follow the PTI Design Manual. Our recommendations for conventionally reinforced slab as well as the post-tensioned slab are presented below:

Minimum Grade Beam Depth Below the Final Grade	:	1.5-ft
Minimum Grade Beam Width	:	10-inch
Allowable Net Bearing Capacity		
Total (Dead + Live) Loading	:	1,350 psf
Dead + Sustained Live Loads	:	900 psf
Slab Subgrade Coefficient		
Slab-on-Vapor Sheeting over Sand	:	0.75
Depth of Deepest Root Fibers	:	15-ft
Edge Moisture Variation, e_m, feet		
Edge Lift	:	4.8
Edge Drop	:	9.0
Differential Swell, y_m, inches		
Edge Lift	:	0.8
Edge Drop	:	1.0
Effective Plasticity Index (PI)	:	21
Structural Fill Type	:	See Site Preparation Section
The Required Minimum Fill Undrained Shear Strength	:	1,000 psf
Support Index	:	0.96
Climatic Rating	:	26
Thornthwaite Moisture Index	:	18

Design Suction Envelope	:	Post-Equilibrium
Potential Vertical Rise (PVR)	:	1.0-inch

Grade beams proportioned in accordance with the above bearing capacity values will have a factor of safety of 3.0 and 2.0 with respect to shearing failure for dead and total loading, respectively. Footing weight below final grade can be neglected in the determination of design loading.

The allowable grade beam bearing pressures for the floating slab foundations should be reduced by 50 percent if the surficial sandy silt and sandy lean clay soils become saturated during the life of the structures.

The differential movement values presented in this report are based on climate-controlled soil conditions and are not valid when influenced by significant other conditions, such as trees, poor drainage, slope, cut and fill sections, etc.

The project site has the potential for construction problems related to the surficial layer of sandy silt and sandy lean clay soils. These permeable soils are generally underlain by relatively impermeable clays. Thus, due to poor site drainage, wet season or site geohydrology, water ponds (within the sandy silt and sandy lean clay soils) on the clays and creates a "perched water table condition". The surficial sandy silt and sandy lean clay soils become extremely soft when wet. Sometimes this condition may result in moisture migration (vapor) through the foundation slab. This condition can be reduced by using a layer of polyethylene moisture barrier or removing surficial sandy silt and sandy lean clay soils and replacing it with select structural fill. The structural fill should not extend beyond the structure footprint in accordance with our "Site Preparation" section.

A bedding layer of leveling sand, one- to two-inch in thickness, may be placed beneath the floor slab. A layer of high-performance polyethylene moisture barrier should be used above the sands to prevent moisture migration through the slab. The excavations for the grade beams should be free of loose materials prior to concrete placement.

Information was not available on whether fill will be used to raise site grade prior to slab construction. In the event that fill is placed on the site, specifications should require placement in accordance with our recommendations given in the "Site Preparation" section. Lack of proper site preparation may result in additional stress and inferior slab performance. The on-site soils, with the exception of sands and silts, free of root organics, can be used as structural fill under a post-tensioned slab foundation. Sands should not be used as fill materials at this site (with the exception of top two-inch of leveling sand under the slab).

7.7 Foundation Settlement

A settlement analysis was not within the scope of this study. It is anticipated that footings, grade beams and slabs designed using the recommended allowable bearing pressures will experience small settlements that will be within the tolerable limit for the proposed structure.

7.8 Vegetation Control

We recommend trees not be planted or left in place (existing trees) closer than half the canopy diameter of mature trees from the grade beams, typically a minimum of 20-ft. Alternatively, root barriers must be placed near the exterior grade beams to minimize tree root movements under the floor slab. This will minimize possible foundation movements as a result of tree root systems.

7.9 Foundation Maintenance

Long term performance of structures depends not only on the proper design and construction, but also on the proper foundation maintenance program.

A properly designed and constructed foundation may still experience distress from the vegetation and expansive soil which will undergo volume change when correct drainage is not established or incorrectly controlled water source, such as plumbing/sewer leaks, excessive irrigation, water ponding near the foundation becomes available.

Our general recommendations on foundation maintenance are presented in the article at the end of this report. More foundation maintenance information can be found at **Foundation Performance Association Document #FPA-SC-07-0** (Ref. 2).

7.10 Site Drainage

It is recommended that site drainage be well developed. Surface water should be directed away from the foundation soils (use a slope of about 5% in the grass within 10-ft of foundation). No ponding of surface water should be allowed near the structure.

In the event that sprinkler systems are used, we recommend that the sprinkler system be placed all around the structure to provide a uniform moisture condition throughout the year. This will reduce fluctuations in subsoil moisture and corresponding movement.

8.0 PAVEMENT SECTIONS

8.1 General

It is planned to construct the crushed granite pavement at Bolivar Beach, Galveston County, Texas. The paving will be subjected to pedestrian loading with limited access by light maintenance vehicles. The length of the walking loop is not known at this time. The pavement design in this study is generally in accordance with ASSHTO 1993 Guide of Design of Pavement Structure (Ref. 11) as well as Texas Department of Transportation (TxDOT) "Pavement Design Guide", 2011, Chapters 5 and 8 (Ref. 12). We propose three options for the proposed walking loop as follows:

1) Option No. 1: Using crushed granite on top of the existing subgrade without geogrid reinforcement.
2) Option No. 2: Using crushed granite on top of the existing subgrade with geogrid reinforcement.
3) Option No. 3: Using crushed granite on top of the lime-fly ash stabilized subgrade.

The recommended pavement thicknesses are presented in the following sections.

8.2 Crushed Granite Paving without Geogrid

This option does not consider geogrid reinforcement. The untreated onsite material will be used as the subgrade materials. However, a geotextile fabric separator should also be placed between the subgrade and aggregate base to minimize migration of the aggregate material into the underlying subgrade. The recommended pavement component thicknesses are summarized as follows:

		Thickness, inch
Aggreagte:	Crushed Granite Material (TxDOT Item 247, Grade 1, See Notes 1) Compact in 8" lifts to 100% of Maximum Modified Proctor Density (ASTM D 1557) at a Moisture content between optimum and ± 2% of optimum moisture.	12
Subgrade:	Compact to 95% of Maximum Standard Proctor Density (ASTM D 698) at a moisture content between optimum and ± 2% of optimum moisture.	8

Notes: 1. Reference Texas Department of Transportation Specifications (TxDOT).

8.3 Crushed Granite Paving with Geogrid

This option consists of using geogrid reinforcement (Tensar® TX5) on top of the existing subgrade. These geogrids can be used as reinforcement to reduce required pavement thickness and improve the overall performance of the granular pavement. These benefits depend upon proper installation of the geogrid materials as put forth in Tensar Installation Guide. The geogrid specification for Tensar TX5 can be obtained from the Tensar International Corporation. The geogrid layer should be placed between the subgrade and the crushed stone material layer. The recommended pavement component thicknesses are summarized as follows:

		Thickness, inch
Aggreagte:	Crushed Granite Material (TxDOT Item 247, Grade 1, See Note 1) Compact in 8" lifts to 100% of Maximum Modified Proctor Density (ASTM D 1557) at a Moisture content between optimum and ± 2% of optimum moisture.	8
Subgrade:	Compact to 95% of Maximum Standard Proctor Density (ASTM D 698) at a moisture content between optimum and ± 2% of optimum moisture.	8

Notes: 1. Reference Texas Department of Transportation Specifications (TxDOT).

Geogrid placement over the subgrade should be in general accordance with the "Installation Guide of Spectra Roadway Improvement System" provided by Tensar. The recommendations on geogrid placement are summarized as below:

1. Adjacent geogrid rolls are normally not connected to one another, particularly if fill is placed and spread. A notable exception is over very soft subgrades (CBR < 0.5) where nylon cable ties can be effective in helping maintain overlap dimensions. These ties are not considered structural connections, but rather construction aids.

2. Tensar TX5 Geogrids may be anchored in place to maintain overlaps and alignment over the coverage area. Before fully unrolling the geogrid, anchor the beginning of the roll, in the center and at the corners, to the underlying surface.

3. When aggregate fill is spread by pushing it over the TX5 Geogrid with heavy equipment, such as bulldozers, the shoving action may create a "wave" in the sheet of geogrid ahead of the advancing fill. Shoveled fill or pins can trap this wave and force the geogrid up into the aggregate layer where it can be damaged by the spreading equipment.

Further information on geogrid installation can be obtained from Tensar web site (www.tensarcorp.com), under "Roadway Improvement".

8.4 Crushed Granite Paving with Lime Stabilized Subgrade

This option consists of using crushed granite on top of the lime stabilized subgrade to support the traffic loads. The recommended pavement thicknesses are as follows:

		Thickness, inch
Aggregate:	Crushed Granite Material TxDOT Item 247, Grade 1, See (Notes 1 and 2), compact in 8" lifts to 100% of Maximum Standard Proctor Density (ASTM D 698) at a moisture content between optimum and ±2% of optimum.	10
Lime-fly Ash Stabilized Subgrade:	(2% Lime and 8% Fly Ash by dry weight TxDOT Specification Item 265 for lime-fly ash treated subgrade, Notes 1 and 2) Compact to 95% of Maximum Standard Proctor Density (ASTM D 698) at a moisture content between optimum and ±2%.	8

NOTES:

1. Reference Texas Department of Transportation Specifications (TxDOT).
2. Use 2% lime and 8% fly-ash by dry weight to stabilize the subgrade soils. The percentages are given as application by dry weight and are typically equivalent to about 10 pounds of lime and 40 pounds of fly Ash per square yard per 6-inch depth.

9.0 OTHER DESIGN CONSIDERATIONS

The potential foundation problems can be reduced by the incorporation of additional design features. Recommended items for consideration are outlined below:

1. Positive drainage should be maintained away from the foundation and pavement areas, both during and after construction.

2. Roof drainage should be collected by a gutter system and downspouts with discharge transmitted by pipe to a storm drainage system or to a paved surface where water can drain away without entering the soil.

3. Sidewalks should be sloped away from the building so that water is drained away from the structure. Water stops, mastic or other means of positive sealing of joints should be used to prevent water intrusion between joints.

4. Parking lots, streets and surface drainage should be sloped away from the building on all sides. Water should not be allowed to pond near the building, pavement or landscape areas.

5. Paving, if possible, should commence at the perimeter of the structural walls to limit moisture content change in floor slab areas.

6. Sand bedding should be specifically prohibited in pavement areas since these more porous soils can allow water inflow which can cause heave and strength loss in the subgrade soils.

7. Backfill for utility lines should consist of low plasticity clays or lime-treated clays. These soils should have a liquid limit of less than 40 and plasticity index (PI) between 12 and 20 and at least 35% of soil particles passing No. 200 sieve. Utility line entrance to under the buildings shall be sealed with bentonite.

8. We recommend that trees should not be planted or left in place (existing trees) closer than half the canopy diameter of mature trees from the grade beams, typically a minimum of 20-feet. Alternatively, root barriers must be placed near the exterior grade beams to minimize tree root movements under the floor slab. This will reduce the risk of possible foundation movement as a result of tree root systems.

9. We recommend that the sprinkler system be placed all around the structure to provide a uniform moisture condition throughout the year. This will reduce fluctuations in subsoil moisture and corresponding movements.

10. Long term performance of structures depends not only on the proper design and construction, but also on the proper foundation maintenance program. A properly designed and constructed structure may still experience distress from vegetation and soils which may undergo volume change when correct drainage is not established or an incorrectly controlled water source, such as plumbing/sewer leaks, excessive irrigation, water ponding near the foundation becomes available. More foundation maintenance information can be found at **Foundation Performance Association Document #FPA-SC-07-0** (Ref. 2).

10.0 CONSTRUCTION CONSIDERATIONS

10.1 Surface Water Drainage

In order to minimize ponding of surface water, site drainage should be established early in the project construction so that this condition will be controlled.

10.2 Site Preparation

The project site has the potential for construction problems related to the surficial layer of cohesionless soils. These permeable surficial soils are underlain by relatively impermeable clay soils. Thus, due to poor site drainage, wet season, or site geohydrology, water ponds on the clay soils create a "perched water table condition". The surficial cohesionless soils could become extremely soft when wet and must be stabilized, aerated, or replaced. In the event that these surficial soils become wet, they will experience rutting and pumping. Therefore, these soils may have to be improved. Our recommendations for subgrade improvements are presented in the earthwork section of this report. Our recommendations for site preparations in the floor slab and pavement area are summarized in the following report section:

1. In general, remove all vegetation, tree roots, organic topsoil, existing foundations, paved areas and any undesirable materials from the construction area. Tree trunks and tree roots under the floor slabs should be removed to a root size of less than 0.5-inch. We recommend that the stripping depth be evaluated at the time of construction by a soil technician.

2. Any on-site fill soils, encountered in the structure and pavement areas during construction, must have records of successful compaction tests signed by a licensed professional engineer that confirms the use of the fill and record of construction and earthwork testing. These tests must have been performed on all the lifts for the entire thickness of the fill. In the event that no compaction test results are available, the fill soils must be removed, processed and recompacted in accordance with our site preparation recommendations. Excavation should extend at least two-feet beyond the structure and pavement area. Alternatively, the existing fill soils should be tested comprehensively to evaluate the degree of compaction in the fill soils.

2. The subgrade areas should then be proofrolled with a loaded dump truck or similar pneumatic-tired equipment with loads ranging from 25- to 50-tons. The proofrolling serves to compact surficial soils and to detect any soft or loose zones. The proofrolling should be conducted in accordance with TxDOT Standard Specification Item 216. Any soils deflecting excessively under moving loads should be undercut to firm soils and recompacted. Any subgrade stabilization should be conducted after site proofrolling is completed and approved by the geotechnical engineer. The proofrolling operations should be observed by an experienced geotechnician.

4. Scarify the subgrade, add moisture, or dry if necessary, and recompact to 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor). The moisture content at the time of compaction of subgrade soils should be within ±2% of the Proctor optimum value. We recommend that the degree of compaction and moisture in the subgrade soils be verified by field density tests at the time of construction. We recommend a minimum of four field density tests per lift or one every 2,500 square feet of floor slab areas, whichever is greater.

5. Structural fill beneath the structure area may consist of off-site inorganic lean clays with a liquid limit of less than 40 and a plasticity index between 12 and 20 and at least 35% of soil particles passing No. 200 sieve. Other types of structural fill available locally, and acceptable to the geotechnical engineer, can also be used.

   These soils should be placed in loose lifts not exceeding eight-inch in thickness and compacted to 95% of the maximum dry density determined by ASTM D 698 (standard Proctor). The moisture content of the fill at the time of compaction should be within ±2% of the optimum value. We recommend that the degree of compaction and moisture in the fill soils be verified by field density tests at the time of construction. We recommend that the frequency of density testing be as stated in Item 4.

6. The backfill soils in the trench/underground utility and root excavation areas should consist of select structural fill, compacted as described in Item 4. In the event of compaction difficulties, the trenches should be backfilled with cement-stabilized sand or other materials approved by the Geotechnical Engineer. Due to the high permeability of sands and potential surface water intrusion, bank sands should not be used as backfill material in the trench/underground utility and tree root excavation areas.

7. In cut areas, the soils should be excavated to grade and the surface soils proofrolled and scarified to a minimum depth of six-inch and recompacted to the previously mentioned density and moisture content.

8. The subgrade and fill moisture content and density must be maintained until paving or floor slabs are completed. We recommend that these parameters be verified by field moisture and density tests at the time of construction.

9. We recommend that the site and soil conditions used in the structural design of the foundation be verified by the engineer's site visit after all of the earthwork and site preparation has been completed and prior to the concrete placement.

10.3 Suitability of On-Site Soils for Use as Fill

10.3.1 General

The on-site soils can be used as fill. There are typically three types of fill at a site. These fills can be classified as described in the following report sections.

10.3.2 Select Structural Fill

This is the type of fill that can be used under the floor slab, paving, etc. These soils should consist of lean clays, free of root organics, with liquid limit of less than 40 and plasticity indices between 12 and 20 and at least 35% of soil particles passing No. 200 sieve.

10.3.3 Structural Fill

This type does not meet the Atterberg limit requirements for select structural fill. This fill should consist of lean clays or fat clays. They can be used under a post-tensioned slab foundation or paving.

10.3.4 General Fill

This type of fill consists of sands and silts. These soils are moisture sensitive and are difficult to compact in a wet condition (they may pump). Furthermore, these soils erode easily. Their use is not recommended under the floor slabs or pavements. They can be used in the planter areas at least 5-ft away from the structure. They can also be used for site grading outside the structure and pavement areas.

10.3.5 Use of On-Site Soils as Fill

The on-site soils can be used as fill materials as described in the following sections:

		Use as Fill			
Stratum No.[1]	Soil Type	Select Structural Fill	Structural Fill	General Fill	Notes
I	Silty Sand (SM)	–	–	✓	2, 3
II	Fat Clay (CH)	–	✓	✓	2, 4

Notes:

1. See the soil stratigraphy and design conditions sections of this report for strata description.
2. All fill soils should be free of organics, roots, etc.
3. The on-site silty sand soils are moisture-sensitive and erode easily. These soils will pump when they get wet. Compaction difficulties will occur in these soils in a wet condition.
4. These soils, once lime modified (5% by dry weight), can be used as select structural fill.

10.4 Drilled Footings Installations

The drilled footings installations must be in accordance with the most recent version of American Concrete Institute (ACI) Reference Specifications (Ref. 9) for the construction of drilled piers (ACI 336.1). Furthermore, it should comply with U.S. Department of Transportation, drilled shafts construction procedures and design methods (Ref. 10).

The drilled footing excavations should be free of loose materials and water prior to concrete placements, and concrete should be poured immediately after drilling the holes.

Due to the potential variability of the on-site soils and potential groundwater fluctuations, we recommend that the four corner piers be drilled first to better evaluate the constructability of the depth and bell to shaft ratios recommended herein. Once this information is field verified, all other piers need to be constructed accordingly.

Detailed observations of pier construction should be required by a qualified engineering technician to assure that the piers are (a) founded in the proper bearing stratum, (b) have the proper depth, (c) have the correct size, and (d) that all loose materials have been removed prior to concrete placement.

10.5 Helical Pile Installations

Experience indicates that torque required to install a helical pile can be used to estimate its compressive capacity (Ref. 11). The contractor should screw the pile into ground to desired torque. Do not over-torque. Furthermore, grout can be placed if specified in the design and brackets can also be installed.

In general, the ultimate compressive capacity of helical pile can be estimated in the field, using a value of 9 to 10 times the value of the torque, for square base products. The ultimate compressive capacity will be 6 to 9 times of the field torque, if tubular products are used. The structural engineer should consult with the helical pile manufacturers for piles that can resist corrosion.

10.6 Driven Timber Pile Installation (If Used)

The selection of the proper hammer for the size, length, and weight of piles for the soil conditions and for the required load capacity or driving requirements can be guided by wave equation analyses. In some cases, hammer formulas have indicated satisfactory values only to have the pile subsequently fail.

A hammer with rated energy on the order of 20,000 ft-lbs should be capable of installing the driven timber piles. The pile capacity curves should be modified if driving aids (such as predrilling, jetting, etc.) are used.

Design and Construction of Timber Piles should be in general accordance with the Timber Pile Design and Construction Manual (Ref. 8) developed by Timber Piling Council and American Wood Preservers Institute.

10.7 Prestressed Concrete Pile Installation

The prestressed concrete pile is sensitive to tensile force during driving; therefore, it should be driven carefully. The prestressed concrete pile may be damaged during driving into the ground with blow count values greater than 120 blows per foot. The pile capacity curves should be modified if driving aids such as pre-drilling or jetting are used. A hammer with a rated energy of 60,000 ft-lbs should be capable of installing the prestressed concrete pile. However, it is the responsibility of the contractor to select the proper hammer type that will provide enough energy to drive the pile to the bearing.

10.8 Consideration for Pile Driving

Pile driving difficulties and pile cracking may occur if the correct pile-hammer system is not used. We recommend a one-dimensional wave equation analysis be performed as an aid in making proper equipment selection to minimize driving difficulties. Predrilling may be needed to facilitate the installation of piles. If predrilling is used, the drill bit size should not exceed 75% of the pile tip diameter or width. Predrilling should be limited to within 5-ft of the pile tip depth. The piles should then be driven with normal effort to grade.

We recommend all piles be driven in accordance with Texas Department of Transportation Standard Specification Item 404 Driving Piles. Furthermore, all piles shall be driven to the specified length and tip elevations.

10.9 Spread Footings Excavations

An excavation or trench which is three-ft or deeper must be protected by sheeting/bracing shoring or sloped. Based on soil strength data, temporary (less than 24 hours) open-trenched, non-surcharged, and unsupported excavations should be made on slopes of flatter than 1.5 (h):1 (v). Vertical cuts can be constructed, provided shoring and bracing are used for the excavation wall stability. Benched excavation can also be used with average slopes of about 1(h):1(v) and steps should not be higher than five-ft. In all cases, excavations should conform to OSHA guidelines. Flatter slopes may have to be used if large amounts of sand need to be excavated for deep installations. Specifications should require that no water be allowed to pond in the excavations. The surface slopes should be protected from deterioration and weathering if they are to be left open for more than 24 hours.

Excavations should be performed with equipment capable of providing a relatively clean bearing area. Excavation equipment should not disturb the soil beneath the design excavation bottom and should not leave large amounts of loose soil in the excavation.

Foundation excavations should be protected against any significant change in soil moisture content and disturbance by construction activity. If concrete is not poured the same day, the excavation is completed, we recommend placement of a thin seal slab over the base of the excavation.

10.10 Surface Water Drainage

In order to minimize ponding of surface water, site drainage should be established early in project construction so that this condition will be controlled.

10.11 Earthwork

10.11.1General

Difficult access and workability problems will most likely occur in the surficial cohesionless soils due to poor site drainage, wet season, or site geohydrology. Considering the soil stratigraphy, the construction of this project should be conducted during the dry season to avoid major earthwork problems. In the event the subgrade soils become wet and experience pumping problems, they can be improved by (a) improving drainage, (b) soil mixing, (c) opening up (scarifying) to dry up, (d) removing and replacing with dry cohesive soils, or (e) chemically modified or stabilizing the soils. These alternatives are discussed in the following report sections.

10.11.2 Improving Drainage

The project site drainage in the pumping soils can be accomplished by placing several shallow bleeder ditches (about 18-inch ±) in the surficial cohesionless soils. These bleeder ditches should be directed to a low area, such as a hole (detention pond) or another ditch in the lowest elevation area of the site. This will allow the surficial soils to drain the water and make the drying process faster. The hole/low area should not be under the building areas. The excess water can be pumped out of the hole and moved off-site.

10.11.3 Soil Mixing

The on-site cohesionless soils (silty sand, Stratum I) can be mixed with on-site cohesive soils (lean clay, Stratum II) to reduce subgrade pumping. GET can do a mix design to come up with soil mix percentages if this option is considered.

10.11.4 Subgrade Drying

The on-site wet soils can be opened up so that it would dry up. However, opening up the surficial cohesionless soils for drying purposes may not be practical, due to cyclic rainfall in the Gulf-Coast area.

10.11.5 Removal and Replacement

The surficial cohesionless soils can be removed and replaced with select structural fill. The actual depth of removal and replacement should be evaluated in the field, but it can be the whole thickness of surficial cohesionless soils. This procedure will include the removal of the surficial cohesionless soils, proofrolling and compacting the subgrade cohesive soils to a minimum of 95 percent standard proctor density (ASTM D 698). The site can then be backfilled with select structural fill, compacted to a minimum of 95 percent of standard Proctor density. The proofrolling should be in accordance with the site preparation section of this report. All of the fill soils should be placed and tested in accordance with the site preparation section of this report.

10.11.6 Modification/Stabilization

It is recommended that the on-site cohesionless soils be modified (to dry up), using 5 to 10 percent fly ash by dry weight in accordance with the City of Houston Specifications, Section 02337. The estimated amounts of fly ash per depth of modification are as follows:

Modification Depth, in.	Fly Ash Weight Range, lbs. per Square Yard. 5%	10%
6	23	45
12	46	90
18	69	135
24	92	180

We recommend that five percent fly ash be used if the surficial soils are relatively moist at the time of application. Higher levels (10 percent) of fly ash should be used if wet and soggy subgrade soils are encountered.

The subgrade soils should be removed to a depth of 24-inch (or more) below the existing grade. These soils should be stockpiled. The soils below a depth of 24-inch should be modified to a depth of 12-inch. These soils should be compacted to a minimum of 95 percent of standard Proctor density (ASTM D 698). The stockpiled soils should then be modified and replaced in six-inch lifts and compacted to 95 percent of maximum dry density as determined by ASTM D 698 at moisture contents within ±2 percent of optimum.

Due to poor drainage and the depth of the cohesionless soils, the depth of stabilization may be as deep as the depth of cohesionless soils. A test section can be implemented for this purpose. The subgrade soils should be modified in six-inch lifts and compacted within four hours of mixing and placement. All of the subgrade soils should be compacted to a minimum of 95 percent of the standard Proctor density at the moisture content of ±2 percent of optimum. The degree of compaction for the lifts, below a depth of 24-inch can be relaxed to 90 percent of maximum dry density to ease the construction procedures.

The subcontractor who will be doing the subgrade modification or stabilization should be experienced with stabilization procedures and methods. Furthermore, all of the earthworks on this project should be monitored by our geotechnician to assure compliance with the project specifications.

Once the subgrade is constructed, the soils at the top of the subgrade should be slicked and the subgrade needs to be crowned such that the all-surface water would drain away. No low areas should be left within the subgrade areas since these areas would hold water and destroy the subgrade structure.

10.12 Construction Surveillance

Construction surveillance and quality control tests should be planned to verify materials and placement in accordance with the specifications. The recommendations presented in this report were based on a discrete number of soil test borings. Soil type and properties may vary across the site. As a part of quality control, if this condition is noted during the construction, we can then evaluate and revise the design and construction to minimize construction delays and cost overruns. We recommend the following quality control procedures be followed by a qualified engineer or technician during the construction of the facility:

- Observe the site stripping and proofrolling.

- o Verify the type, depth, and amount of stabilizer.
- o Verify the compaction of subgrade soils.
- o Evaluate the quality of fill and monitor the fill compaction for all lifts.
- o Monitor and test the foundation excavations for, strength, cleanness, depth, size, etc.
- o Observe the foundation make-up prior to concrete placement.
- o Monitor concrete placement, conduct slump tests and make concrete cylinders.
- o Conduct after pour observations, including post-tensioned slab cable stress monitoring, if used.
- o Monitor installation of drilled footings, spread footings or helical piles, if used.
- o Conduct after construction site visit to evaluate the site landscaping, drainage and the presence of trees near the structure.

It is the responsibility of the client, to notify GET of when each phase of the construction is taking place so that proper quality control and procedures are implemented. More information regarding construction quality control can be found at the **Foundation Performance Association Documentation #FPA-SC-10-1** (Ref. 2).

11.0 RECOMMENDED ADDITIONAL STUDIES

We recommend the following additional studies be conducted:

1. This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. We recommend that GET be retained to review the plans and specifications to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted as intended.

2. **Conduct site characterization studies.** These studies will include the following separate studies:

   - o Phase I Environmental Site Assessment Study to evaluate the risk of contamination at the site.
   - o Review previous aerial photos of the project site.

- o Review site topography.
- o Conduct a site visit to look for drainage features, slopes, seeps, trees and other vegetation; fence lines, ponds, stock tanks; areas of fill, etc.

3. We recommend obtaining baseline micro-elevations of the floor slabs after floor covering is installed. This information will be valuable in the event of future foundation movements.

12.0 STANDARD OF CARE

The recommendations described herein were conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical engineering profession practicing contemporaneously under similar conditions in the locality of the project. No other warranty or guarantee, expressed or implied, is made other than the work was performed in a proper and workmanlike manner.

13.0 REPORT DISTRIBUTION

This report was prepared for the sole and exclusive use by our client, based on specific and limited objectives. All reports, boring logs, field data, laboratory test results, maps and other documents prepared by GET as instruments of service shall remain the property of GET. Reuse of these documents is not permitted without written approval by GET. GET assumes no responsibility or obligation for the unauthorized use of this report by other parties and for purposes beyond the stated project objectives and work limitations.

We appreciate the opportunity to assist on this project. Please call if there should be any questions.

Very truly yours,

GEOTECH ENGINEERING AND TESTING
TBPE Registration Number F-001183

Dmitry Steshenko

Dmitry Steshenko, Ph.D.
Project Manager

STATE OF TEXAS
SHAILESH NAMEKAR
131521
LICENSED
PROFESSIONAL ENGINEER
Shailesh "James" Namekar
09/10/2025

James Namekar, Ph D., P.E.
Chief Engineer

ND/JN/DAE/nd

Copies Submitted: (1) PDF Copy Email – Mr. Chad Nesvadba

14.0 ILLUSTRATIONS

15.0 REFERENCES

1. "Method for Determining the Potential Vertical Rise, PVR," State Department of Highways and Public Transportation, Test Method Tex 124-E, Austin, Texas.

2. Committee Papers from Foundation Performance Association (FPA), see FPA Website: http://www.foundationperformance.org/committee_papers.cfm.

3. Generalized Solutions for Laterally Loaded Piles", Matlock, H. and Reese, L.C., Journal of Soil Mechanics and Foundation Division, Proceeding of the American Society of Civil Engineers, SM5, October 1960, pp. 63-91.

4. "Basic Helical Screw Pile Design", Donald J. Clayton, P.E., Earth Contac Products, February 2005.

5. "Basic Guidelines for Design Helical Piers for Underpinning," A.B. Chance Company, January 1993 Bulletin 01-9202).

6. "Design of Post-Tensioned Slab-on-Ground", Post-Tensioning Institute, Phoenix, Arizona, Second Edition with June 2019.

7. "Design of Slab-on-Ground Foundation", Wire Reinforcement Institute, Findlay, Ohio, August 1981 and update March 1996.4

8. "Construction and Maintenance Procedures Manual for Post-Tensioned Slabs-on-Ground", 2nd Edition, Post-Tensioning Institute, Phoenix, Arizona, September 1998.

9. "Reference Specifications for the Construction of Drilled Piers (ACI 336.1) and Commentary (ACI 336.1R-98)", American Concrete Institute, Farmington Hills, Michigan.

10. Federal Highway Administration, US-DOT "Drilled Shafts; Construction Procedures and Design Methods", Publication FHWA-IF-99-025, 2010.

11. "AASHTO Guide for Design of Pavement Structures", American Association of State Highway and Transportation Officials, Washington, D.C, 1993.

12. Texas Department of Transportation (TxDOT) "Pavement Design Guide", 2011.

13. "Timber Pile Design and Construction Manual," Timber Piling Council and American Wood Preserves Institute, 2002.

PLAN OF BORINGS (boring locations are approximate)			NORTH
PROJECT: G/S for Proposed Pocket Park Dune Walkover and Facilities at Bolivar Beach, Galveston County, TX			
SCALE: NOT TO SCALE	DATE: SEPTEMBER 2025	PROJECT NO.: 25-578E	

LOG OF BORING NO. B-1

Sheet 1 of 1

Geotech Engineering and Testing
17407 US Highway 59
Houston, Texas 77396
Phone: 713-699-4000 Fax: 713-699-9200

PROJECT: G/S for the Proposed Dune Walkover and Facilities at Bolivar Beach
LOCATION: Galveston County, Texas
PROJECT NO.: 25-578E STATION NO.:
DATE: 8-29-25 COMPLETION DEPTH: 30.0 ft.

UNDRAINED SHEAR STRENGTH, tsf: ▲ HAND PENETROMETER; ■ TORVANE; ● UNCONFINED COMPRESSION; ○ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 0.5, 1.0, 1.5, 2.0, 2.5)

DEPTH, ft	SPT N-VALUE blows per foot	OVM, ppm	DESCRIPTION (ELEVATION: Existing Grade)	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SUCTION (pF)	DRY UNIT WEIGHT, pcf	PERCENT COMPACTION	PASSING/FAILING (P/F)
0			SILTY SAND (SM), medium dense, brown, light gray, with root fibers to 13', gravels - dark brown 1' to 13'	4				31				
	24											
5	27											
	26											
	23			18				19				
13			FAT CLAY (CH), very soft, dark brown, light gray, with root fibers to 15', gravels									
	2											
	2											
23			- firm 23' to 28'									
28			- stiff 28' to 30'	30	60	20	40					
30												

WATER OBSERVATIONS:
▽ : WATER ENCOUNTERED AT 8.0 ft. DURING DRILLING
▼ : WATER DEPTH AT 8.0 ft. AFTER 0.33-HOUR

DRY AUGER: 0 TO 8 ft.
WET ROTARY: 8 TO 30 ft.

DRILLED BY: Adrian (T)
LOGGED BY: Joseph

LOG OF BORING NO. B-2

GET GEOTECH ENGINEERING & TESTING
Geotech Engineering and Testing
17407 US Highway 59
Houston, Texas 77396
Phone: 713-699-4000 Fax: 713-699-9200

PROJECT: G/S for the Proposed Dune Walkover and Facilities at Bolivar Beach
LOCATION: Galveston County, Texas
PROJECT NO.: 25-578E STATION NO.:
DATE: 8-29-25 COMPLETION DEPTH: 20.0 ft.

DEPTH, ft	SPT N-VALUE blows per foot	OVM, ppm	SYMBOL	SAMPLES	DESCRIPTION ELEVATION: Existing Grade	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	SUCTION (pF)	DRY UNIT WEIGHT, pcf	PERCENT COMPACTION	PASSING/FAILING (P/F)	UNDRAINED SHEAR STRENGTH, tsf (▲ HAND PENETROMETER; ▮ TORVANE; ● UNCONFINED COMPRESSION; ○ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 0.5 1.0 1.5 2.0 2.5
0					SILTY SAND (SM), medium dense, brown, light gray, with root fibers to 13', gravels										
	24														
5	27					20				36					
	26														
	23														
10															
					FAT CLAY (CH), very soft, dark brown, light gray, with root fibers to 15', gravels										
	2														
15															
	2					25	92	25	67						
20															
25															
30															
35															

WATER OBSERVATIONS:
▽ : WATER ENCOUNTERED AT 8.0 ft. DURING DRILLING
▼ : WATER DEPTH AT 8.0 ft. AFTER 0.33-HOUR

DRY AUGER: 0 TO 8 ft.
WET ROTARY: 8 TO 20 ft.

DRILLED BY: Adrian (T)
LOGGED BY: Adrian (T)

OVM2 25-578E(B1-B2).GPJ OVM.GDT 9/10/25

KEY TO LOG TERMS AND SYMBOLS

UNIFIED SOIL CLASSIFICATIONS

Symbol	Material Descriptions
GW	WELL GRADED-GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GM	SILTY GRAVELS, GRAVEL-SAND SILT MIXTURES
GC	CLAY GRAVELS, GRAVEL-SAND CLAY MIXTURES
SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SP	POORLY GRADED SANDS, OR GRAVELLY SANDS, LITTLE OR NO FINES
SM	SILTY SANDS, SAND-SILT MIXTURES a
SC	CLAYEY SANDS, SAND-SILT MIXTURES b
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT
	FILL SOILS

TERMS CHARACTERIZING SOIL STRUCTURE

Slickensided	-	Having incline planes of weakness that are slick and glossy in appearance.
Fissured	-	Containing shrinkage cracks frequently filled with fine sand or silt: usually vertical.
Laminated	-	Composed of thin layers of varying colors and soil sample texture.
Interbedded	-	Composed of alternate layers of different soil types.
Calcareous	-	Containing appreciable quantities of calcium carbonate.
Well Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes.
Poorly Graded	-	Predominantly of one grain size, or having a range of sizes with some intermediate sizes missing.
Pocket	-	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	-	Inclusion less than ⅛-inch thick extending through the sample.
Seam	-	Inclusion ⅛- to 3-inch thick extending through the sample.
Layer	-	Inclusion greater than 3-inch thick extending through the sample.
Interlayered	-	Soils sample composed of alternating layers of different soil types.
Intermixed	-	Soil samples composed of pockets of different soil type and layered or laminated structure is not evident.

COARSE GRAINED SOILS (major portion retained on No. 200 Sieve): Includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Conditions rated according to standard penetration test (SPT)* as performed in the field.

Descriptive Terms	Blows Per Foot*
Very Loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	over 50

* 140 pound weight having a free fall of 30-inch

FINE GRAINED SOILS (major portion passing No. 200 Sieve): Include (1) inorganic or organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength as indicated by hand penetrometer readings or by unconfined compression tests.

Descriptive Term	Undrained Shear Strength Ton/Sq. Ft.	Descriptive Term	Blows Per Foot*
Very Soft	Less than 0.13	Very Soft	< 2
Soft	0.13 to 0.25	Firm	2 – 8
Firm	0.25 to 0.50	Stiff	8 – 15
Stiff	0.50 to 1.00	Very Stiff	15 – 30
Very Stiff	1.00 to 2.00	Hard	> 30
Hard	2.00 or higher		

NOTE: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above because of weakness or cracks in the soil. The consistency ratings of such soils are based on hand penetrometer readings.
* 140 pound weight having a free fall of 30-inch

SOIL SAMPLERS

- SHELBY TUBE SAMPLER
- STANDARD PENETRATION TEST
- AUGER SAMPLING

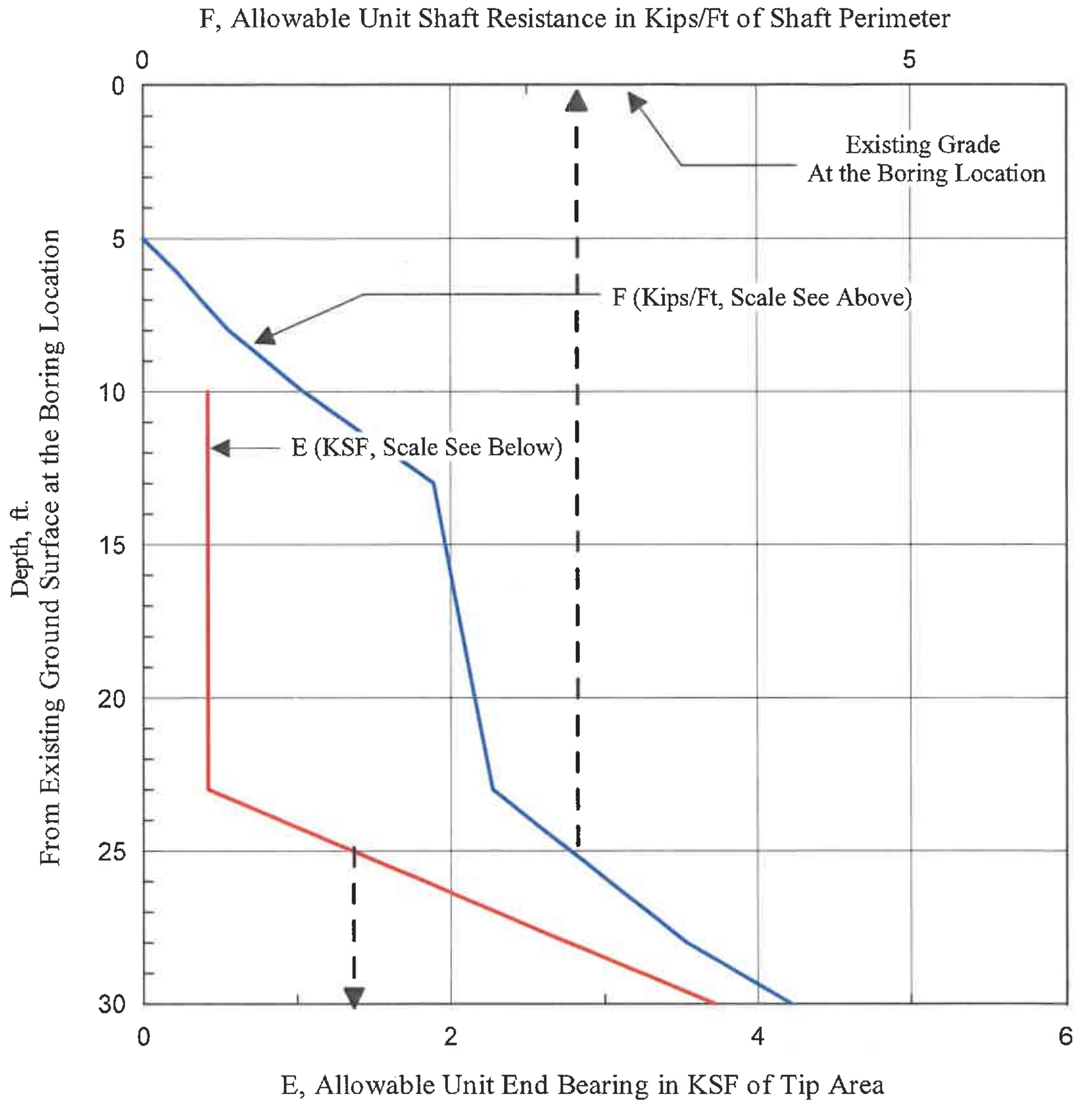
TERMS CHARACTERIZING ROCK PROPERTIES

VERY SOFT OR PLASTIC	Can be remolded in hand: corresponds in consistency up to very stiff in soils.
SOFT	Can be scratched with fingernail.
MODERATELY HARD	Can be scratched easily with knife; cannot be scratched with fingernail. Difficult to scratch with knife.
VERY HARD	Cannot be scratched with knife.
POORLY CEMENTED OR FRIABLE	Easily crumbled.
CEMENTED	Bounded Together by chemically precipitated materials.
UNWEATHERED	Rock in its natural state before being exposed to atmospheric agents.
SLIGHTLY WEATHERED	Noted predominantly by color change with no disintegrated zones.
WEATHERED	Complete color change with zones of slightly decomposed rock.
EXTREMELY WEATHERED	Complete color change with consistency, texture, and general appearance or soil.

PROJECT PICTURES
Project No. 25-578E

Note: The above picture(s) indicate a snapshot of the project and the surroundings. We request that the client review the picture(s) and make sure that they represent the project area. We must be contacted immediately if any discrepancy exists.

PROPOSED PROPOSED DUNE WALKOVER AND FACILITIES
DRILLED FOOTING CAPACITY CURVES - BORING B-1
DESIGN FACTORS F and E



DESIGN EXAMPLE

Drilled Footing (Straight Shaft)
Assumed Drilled Footing Diameter = 24 inches
P = 6.28 feet
A = 3.14 sq. feet
Assumed Drilled Footing Length = 25 feet
Q_c = 6.28*2.75+3.14*1.4
= 21.67 kips

Q_t = 1.0*6.28*2.75
= 12.10 kips

DESIGN EQUATIONS

Compression: $Q_c = PF + AE$

Tension: $Q_t = 0.7\ PF$

TERMS

P = Average shaft perimeter, feet
A = Tip area, square feet
F, E = Factors from curves at shaft depth
Q = Allowable capacity in kips
C, T = Subscripts denoting compression and tension capacity, respectively

Factor of Safety on F = 2
Factor of Safety on E = 3

Project No. 25-578E

GENERAL SOIL DESIGN PARAMETERS FOR LATERAL LOAD ANALYSIS (L-PILE)

(BASED ON BORING B-1)

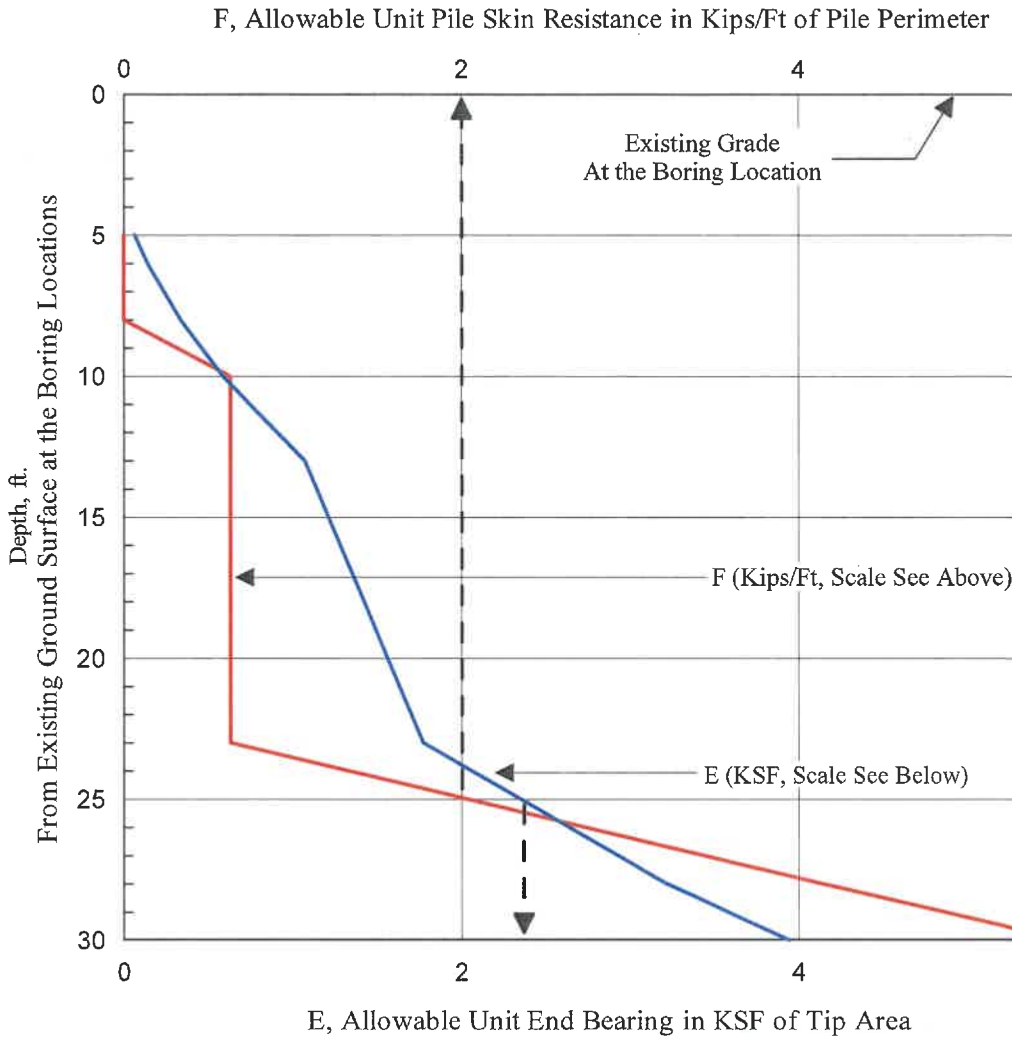
Soil Type	Range of Depth, ft.	Soil-Modulus Parameter (k), pci [1]	ε_{50} [2]	Effective Unit Weight (γ'), pcf	Undrained Shear Strength (C_u), psf	Angle of Internal Friction (ϕ), degree
SILTY SAND (SM)	0 – 2	25	–	62.0	–	28
SILTY SAND (SM)	2 – 8	90	–	62.0	–	30
SILTY SAND (SM)	8 – 13	60	–	62.0	–	30
FAT CLAY (CH)	13 – 23	30	0.02	62.0	–	–
FAT CLAY (CH)	23 – 28	100	0.01	62.0	920	–
FAT CLAY (CH)	28 – 30	200	0.007	62.0	1,240	–

(BASED ON BORING B-2)

Soil Type	Range of Depth, ft.	Soil-Modulus Parameter (k), pci [1]	ε_{50} [2]	Effective Unit Weight (γ'), pcf	Undrained Shear Strength (C_u), psf	Angle of Internal Friction (ϕ), degree
SILTY SAND (SM)	0 – 2	25	–	62.0	–	28
SILTY SAND (SM)	2 – 8	90	–	62.0	–	30
SILTY SAND (SM)	8 – 13	60	–	62.0	–	30
FAT CLAY (CL)	13 – 20	30	0.02	62.0	-	–

Notes: (1) for static loading only
(2) ε_{50} is the strain of clays at 50% of the maximum shear

CONCRETE / TIMBER DRIVEN PILE
DESIGN FACTORS F and E
BASED ON BORING B-1



DESIGN EXAMPLE

PILE
Assumed Pile Width = 12 inches
P = 4.00 ft.
A = 1.00 sq. ft.
Assumed Pile Length = 25 ft.
Q_c = (4.00)*(2.00)+(1.00)*(2.35) = 10.35 kips

Q_t = (0.7)*(4.00)*(2.00) = 5.6 kips

DESIGN EQUATIONS

Compression: $Q_c = PF + AE$

Tension: $Q_t = 0.7\ PF$

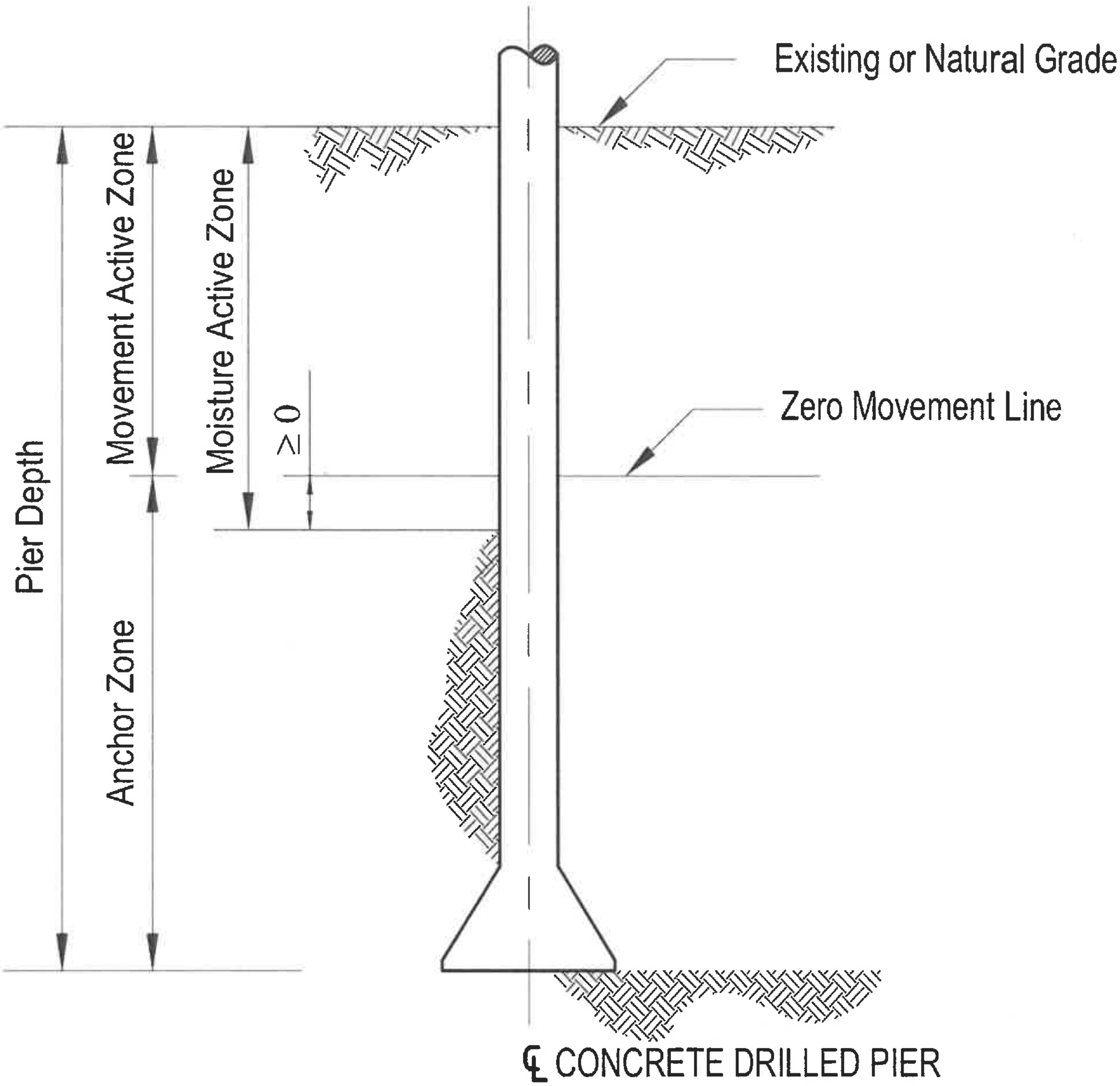
TERMS

P = Average pile perimeter, feet
A = Tip area, square feet
F, E = Factors from curves at tip penetration
Q = Allowable capacity in kips
C, T = Subscripts denoting compression and tension capacity, respectively

Factor of Safety on F = 2
Factor of Safety on E = 2

Project 25-578E

DRILLED FOOTING DEPTH IN EXPANSIVE SOILS



Definition

Pier Depth: See the Drawing.

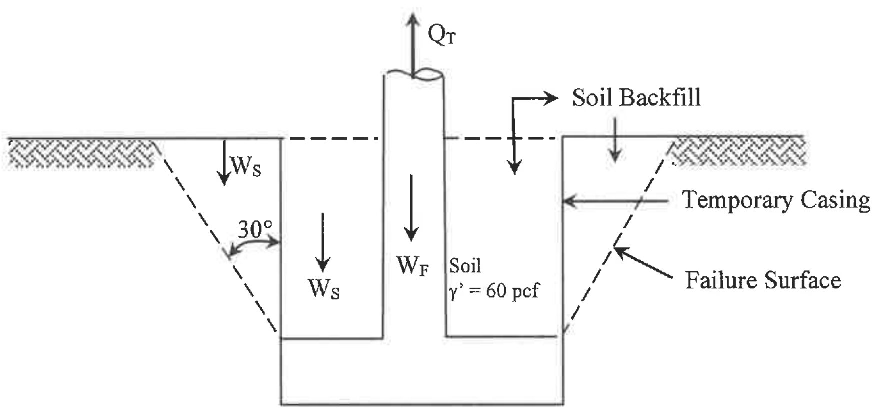
Moisture Active Zone: Depth of an active soil measured from the ground surface downward, wherein moisture fluctuations occur.

Movement Active Zone: Depth of an active soil measured from the ground surface downward where movement can occur due to volumetric moisture changes.

Zero Movement Line: The bottom of movement active zone.

Anchor Zone: Depth to anchor the footing such that it will be sufficient to resist uplift loads.

UPLIFT CAPACITY OF SPREAD FOOTING



Where, $Q_T = W_F + W_S$ (Use F.S = 1.5)
W_F = Weight of Foundation (Submerged)
W_s = Weight of Soil Wedge (Use Soil Unit Weight, $\gamma' = 60$ pcf)

GALVESTON COUNTY
BOLIVAR BEACH POCKET PARK

GILCHRIST, TEXAS
GALVESTON COUNTY

VICINITY MAP
KEY MAP 741Q
SCALE: 1" = 1000'

MARK A. HENRY
COUNTY JUDGE

DARREL APFFEL COMMISSIONER PRECINCT 1	**JOE GIUSTI** COMMISSIONER PRECINCT 2
STEPHEN HOLMES COMMISSIONER PRECINCT 3	**DR. ROBIN ARMSTRONG** COMMISSIONER PRECINCT 4

SHEET INDEX

JOB NO. R318065.01

FEBRUARY 2026

HZ HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston Texas 77042-4248
281.496.0066
www.huitt-zollars.com

PRELIMINARY

SURVEYED BY:
FB NO.

SHEET No. 01 OF 11 DWG No.

GENERAL NOTES:

1. THE CONTRACT SHALL CALL 811 TO IDENTIFY ALL THE EXISTING UTILITY 48 HOURS PRIOR TO START OF CONSTRUCTION.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ANY AND ALL DAMAGES TO THE EXISTING PUBLIC OR PRIVATE UTILITY LINES. ALL THE REPAIRS SHALL BE REPAIRED WITH NO COST TO THE OWNER.
3. ALL THE LOWER PRESSURE SEWER LINES WITH CROSSING UTILITIES INCLUDING WATER SHALL BE IN ACCORDANCE WITH TCEQ REGULATIONS.
4. ALL LOW PRESSURE SEWER LINES SHALL BE ENCASED AT ALL LOCATIONS FOR COUNTY ROAD CROSSING PER COUNTY ENGINEER REQUIREMENTS.
5. OPEN CUT PIPE INSTALLATION WILL NOT BE ALLOWED IN THE COUNTY MAINTAINED PAVING.
6. THE CONTRACTOR TO SUBMIT MANUFACTURER DATA FOR ALL PIPE, VALVES, AND FITTINGS PRIOR TO START OF CONSTRUCTION FOR ENGINEER APPROVAL.
7. THE CONTRACTOR TO FOLLOW UNDINE LLC STANDARD SPECIFICATIONS FOR THE LOW PRESSURE SANITY SERVICE LATERAL CONNECTIONS.
8. CONTRACTOR TO INSTALL ALL ENCASEMENT PIPING PER TXDOT REQUIREMENTS. SEE DETAILS ON DETAILS SHEET X.
9. CONTRACTOR TO COORDINATE CONSTRUCTION/TRAFFIC CONTROL PLANS WITH NEAREST TXDOT MAINTENANCE OFFICE AFTER APPROVAL OF PERMIT.
10. CONTRACTOR TO VERIFY THE EXACT LOCATION OF THE WATER MAIN WITH BOLIVAR PENINSULA SPECIAL UTILITY DISTRICT AT 409-684-3515 PRIOR TO START OF CONSTRUCTION.

SANITARY SEWER NOTES:

1. PIPE SHALL BE SDR26 OR SCHEDULE 40.
2. HANDLE, CUT, AND JOIN PIPE AS PER MANUFACTURERS RECOMMENDATIONS.
3. MANHOLE TO BE PRECAST CONCRETE PIPE, 48-INCH DIAMETER, CONFORMING TO ASTM C-76 FOR VERTICAL STRAIGHT WALL SECTIONS. USE O-RINGS FOR JOINTS. PROVIDE TAPERED CONE FOR TRANSITION FROM 48-INCH MANHOLE DIAMETER TO 24-INCH DIAMETER RING SECTION AT MANHOLE FRAME AND COVER. TAPERED CONES AND NECK RING TO BE REINFORCED CONCRETE IN ACCORDANCE WITH APPLICABLE PROVISIONS OF ASTM C-76. RING AT COVER TO BE ASTM A-48, CLASS NO. 20, CAST IRON. USE 23 1/2 INCH DIAMETER COVER WITH WORDING "SANITARY SEWER".
4. ALL WORK SHALL COMPLY WITH APPLICABLE LOCAL, STATE, AND FEDERAL REGULATIONS.
5. PROVIDE TRENCH SAFETY AS REQUIRED AND IN ACCORDANCE WITH APPLICABLE STATE AND FEDERAL REGULATIONS. A COPY OF THE GEOTECHNICAL INFORMATION OBTAINED BY THE OWNER IS AVAILABLE FROM THE OWNER UPON REQUEST.

WATER NOTES:

1. PIPE LESS THAN 1-1/2 INCHES IN DIAMETER SHALL HAVE A PRESSURE RATING OF NOT LESS THAN 160 PSI. USE SOLVENT WELD JOINTS.
2. PIPE EQUAL TO OR GREATER THAN 1-1/2 INCHES IN DIAMETER SHALL BE SDR 21, PRESSURE RATED CLASS 160 CONFORMING TO ASTM D2241. USE ELASTOMERIC GASKET JOINTS (ASTM D1869).
3. HANDLE, CUT, AND JOIN PIPE AS PER MANUFACTURERS RECOMMENDATIONS.
4. DISINFECT NEW WATER LINES BY APPLICATION OF STERILIZING AGENT. FLUSH LINE, GATHER BACTERIOLOGICAL SAMPLE, AND SUBMIT TO CITY OF HOUSTON HEALTH DEPARTMENT FOR TESTING.
5. ALL WORK SHALL COMPLY WITH APPLICABLE LOCAL, STATE, AND FEDERAL REGULATIONS.
6. PROVIDE TRENCH SAFETY AS REQUIRED AND IN ACORDANCE WITH APPLICABLE STATE AND FEDERAL REGULATIONS. A COPY OF THE GEOTECHNICAL INFORMATION OBSTAINED BY THE OWNER IS AVAILABLE FROM THE OWNER UPON REQUEST.
7. ALL FITTINGS SHALL BE PUSH-ON JOINTS WITH MEGA-LUG CONNECTIONS.

CROSSING DETAIL NOTES:

1. ALL SANITARY SEWERS CROSSING WATER LINES WITH A CLEARANCE BETWEEN 12 INCHES AND 9 FEET SHALL HAVE A MINIMUM OF ONE 18' JOINT OF 150 PSI DUCTILE IRON OR (GREEN) C900 PVC PIPE MEETING ASTM SPECIFICATION D2241 CENTERED ON WATER LINE. WHEN WATER LINE IS BELOW SANITARY SEWER, PROVIDE MINIMUM 2 FOOT SEPARATION.

THE STATE OF TEXAS
COUNTY OF GALVESTON

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

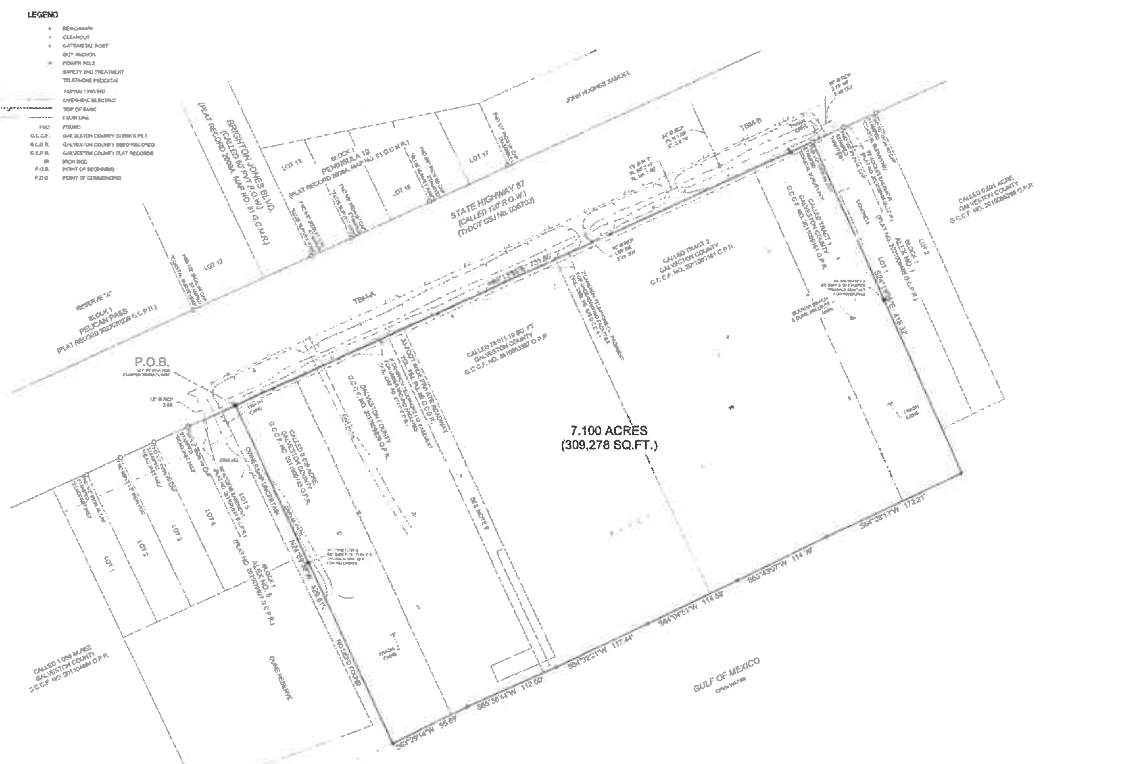
GENERAL NOTES

HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston Texas 77042-4248
281.496.0066
www.huitt-zollars.com

PROJECT NO.
R319065.01
DWG SCALE
HORIZONTAL : N/A
VERTICAL : N/A
DWG NO.
SHEET NO.
02 OF 11

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. MENDOSA
91092
Date: 2/10/25
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

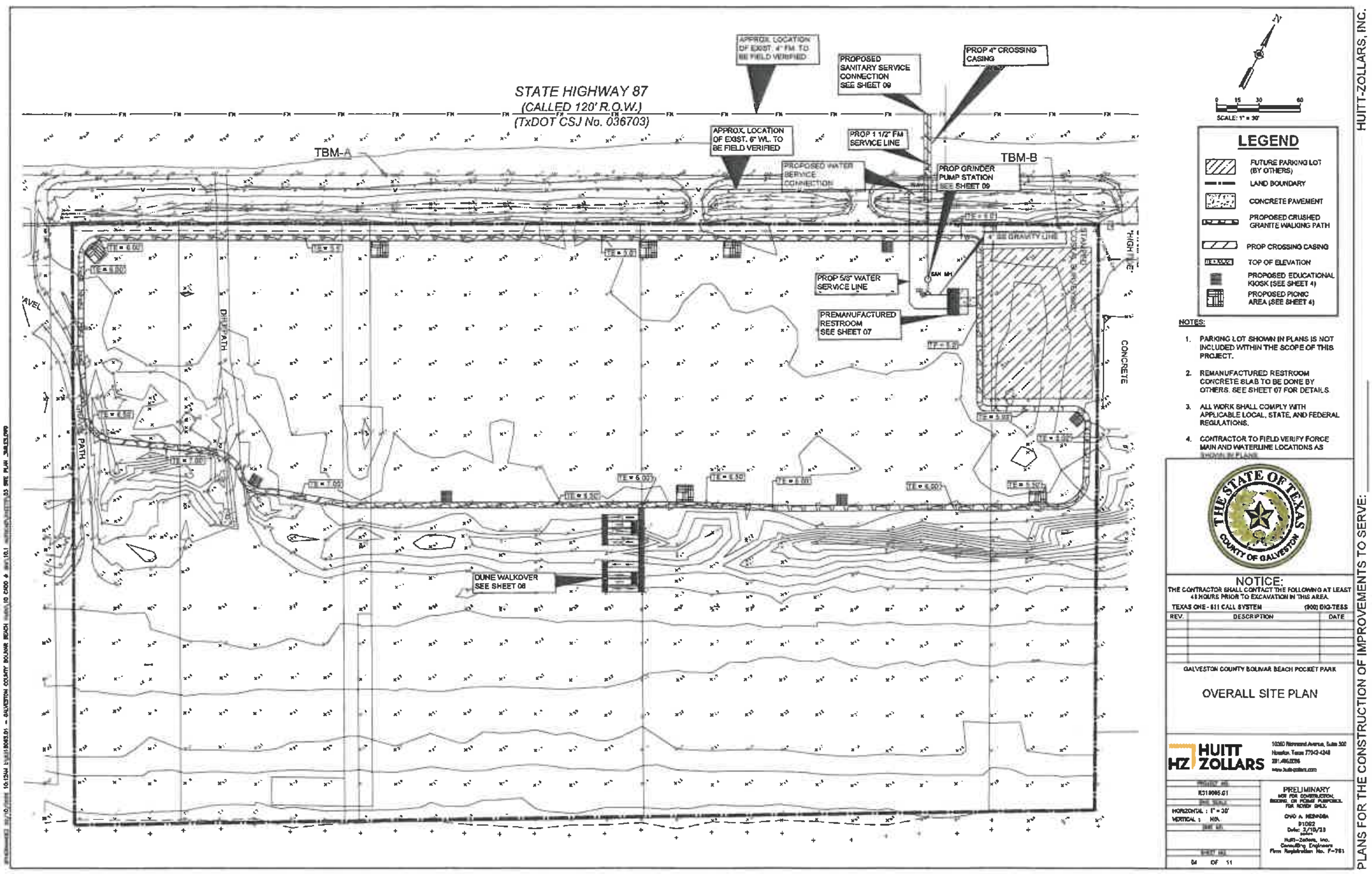
HUITT-ZOLLARS, INC.
PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



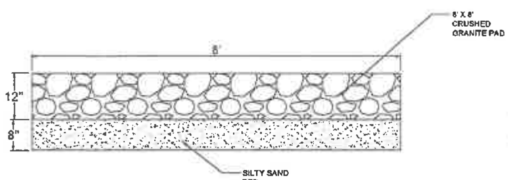
PLAT OF SURVEY W/TOPOGRAPHY
7.100 ACRES (309,278 SQ. FT.)

BOLIVAR BEACH POCKET PARK
SITUATED IN THE
ELIJAH FRANKS 1/3 LEAGUE, A-64
GALVESTON COUNTY, TEXAS

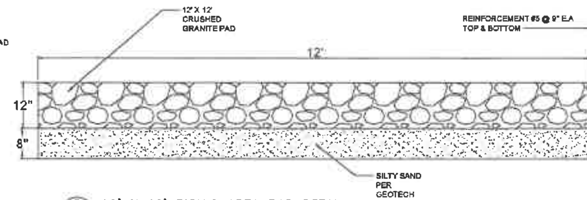
TEJAS SURVEYING



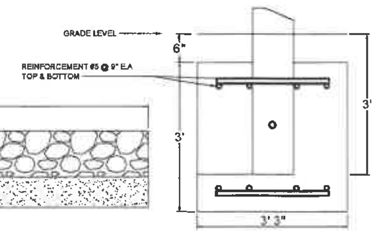
STATE HIGHWAY 87
(CALLED 120' R.O.W.)
(TxDOT CSJ No. 036703)
APPROX. LOCATION OF EXIST. 4" FM. TO BE FIELD VERIFIED
PROPOSED SANITARY SERVICE CONNECTION SEE SHEET 09
PROP 4" CROSSING CASING
APPROX. LOCATION OF EXIST. 6" WL. TO BE FIELD VERIFIED
PROP 1 1/2" FM SERVICE LINE
PROPOSED WATER SERVICE CONNECTION
PROP GRINDER PUMP STATION SEE SHEET 09
TBM-A
TBM-B
SAN MH
PROP 5/8" WATER SERVICE LINE
PREMANUFACTURED RESTROOM SEE SHEET 07
DUNE WALKOVER SEE SHEET 08
CONCRETE
SCALE: 1" = 30'
LEGEND
FUTURE PARKING LOT (BY OTHERS)
LAND BOUNDARY
CONCRETE PAVEMENT
PROPOSED CRUSHED GRANITE WALKING PATH
PROP CROSSING CASING
TOP OF ELEVATION
PROPOSED EDUCATIONAL KIOSK (SEE SHEET 4)
PROPOSED PICNIC AREA (SEE SHEET 4)
NOTES:
1. PARKING LOT SHOWN IN PLANS IS NOT INCLUDED WITHIN THE SCOPE OF THIS PROJECT.
2. REMANUFACTURED RESTROOM CONCRETE SLAB TO BE DONE BY OTHERS. SEE SHEET 07 FOR DETAILS.
3. ALL WORK SHALL COMPLY WITH APPLICABLE LOCAL, STATE, AND FEDERAL REGULATIONS.
4. CONTRACTOR TO FIELD VERIFY FORCE MAIN AND WATERLINE LOCATIONS AS SHOWN IN PLANS.
THE STATE OF TEXAS
COUNTY OF GALVESTON
NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS
REV. DESCRIPTION DATE
GALVESTON COUNTY BOLIVAR BEACH POCKET PARK
OVERALL SITE PLAN
HZ HUITT ZOLLARS
R318005.01
HORIZONTAL : 1" = 30'
VERTICAL : N/A
04 OF 11
PRELIMINARY
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761
HUITT-ZOLLARS, INC.
PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



1 8' X 8' KIOSK PAD DETAIL
N.T.S.



2 12' X 12' PICNIC AREA PAD DETAIL
N.T.S.



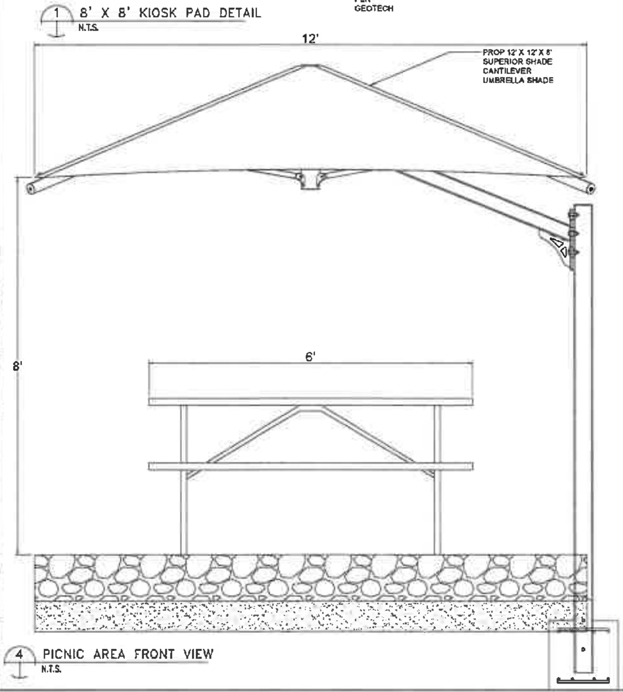
3 FOOTING DETAIL
N.T.S.

FOOTING NOTES:

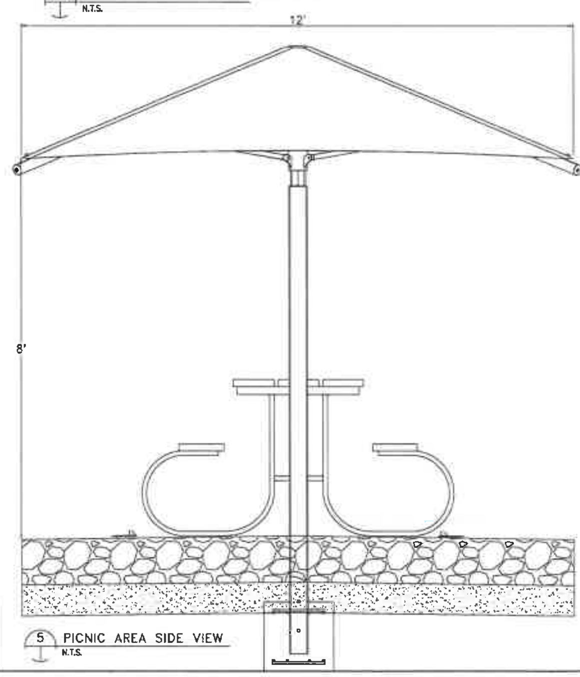
1. FOOTING DESIGN BASED ON 1500 PSF SOIL BEARING PRESSURE.

NOTES:

1. SEE SKYSPAN WEBSITE FOR FRAME AND CANOPY COLOR OPTIONS.
2. INSTALLATION TO BE COMPLETED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.
3. CONTRACTOR'S NOTE: FOR PRODUCT AND COMPANY INFORMATION VISIT WWW.CADDETAILS.COM/INFO AND ENTER REFERENCE NUMBER 4148-060.
4. THE DETAILS REGARDING THE PICNIC TABLES SHALL BE PROVIDED AND SPECIFIED BY THE OWNER.



4 PICNIC AREA FRONT VIEW
N.T.S.



5 PICNIC AREA SIDE VIEW
N.T.S.

THE STATE OF TEXAS
COUNTY OF GALVESTON

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

KIOSK PAD AND PICNIC AREA DETAILS

HZ HUITT ZOLLARS

10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4348
281.496.0066
www.huitt-zollars.com

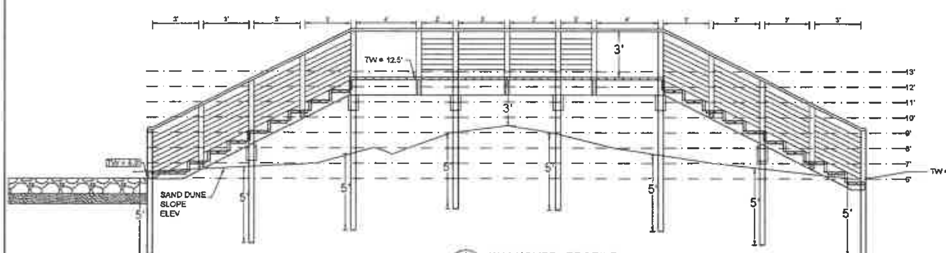
PROJECT NO.
R310065.01
DWG SCALE
HORIZONTAL : N.T.S.
VERTICAL : N.T.S.
DWG NO.

SHEET NO.
05 OF 11

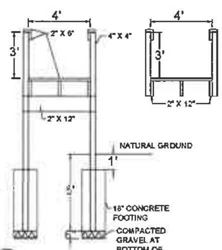
PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. BERNADSA
91092
Date: 2/10/20
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

HUITT-ZOLLARS, INC.

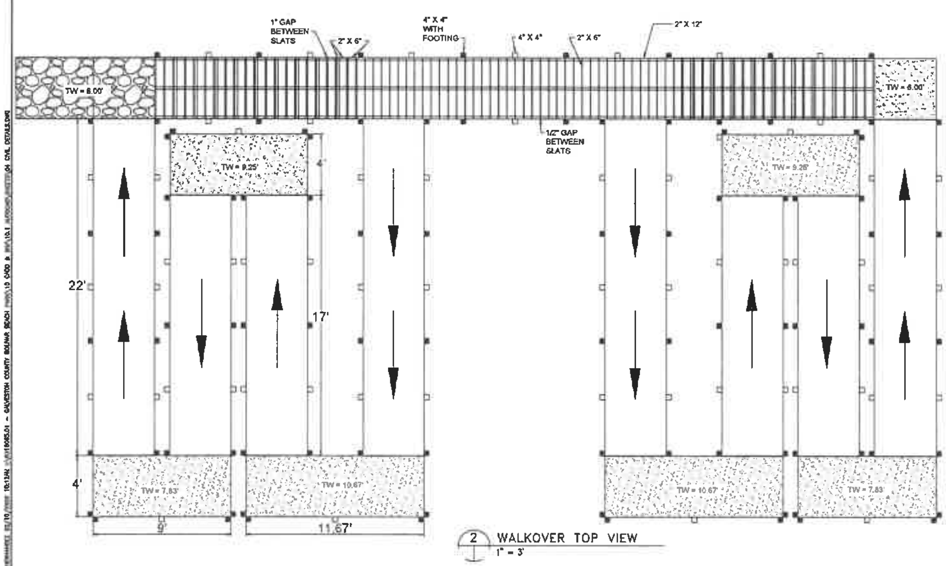
PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



1 WALKOVER PROFILE
1" = 3'



3 WALKOVER S
1" = 3'



2 WALKOVER TOP VIEW
1" = 3'

DUNE RAMP NOTES:

1. THE SAND LANDING ZONE CLOSEST TO THE BEACH SIDE REQUIRES A TEMPORARY FOOTPATH, WHICH CAN BE A REMOVABLE MAT OR AN APPROVED ALTERNATIVE METHOD OF SURFACE STABILIZATION THAT ACCOMMODATES THE REQUIREMENTS.
2. TO MEET COMPLIANCE WITH THE UTILIZATION OF A 1:12 SLOPE ON THE RAMP, RESTING ZONES WITH TEMPORARY FOOT PATHS WERE IMPLEMENTED TO ALLOW AMBULATORY AND PEOPLE IN WHEELCHAIRS TO REST.
3. THERE MUST BE 1/2" OF SPACE BETWEEN THE PLANKS ALONG THE WALKWAY TO ALLOW FOR BOTH VEGETATION UNDERNEATH THE WALKWAY TO RECEIVE RAINFALL AND A SMALL ENOUGH GAP WHERE IT WOULD NOT INHIBIT A WHEELCHAIR USER.
4. THESE REQUIREMENTS AND CRITERIA CAN BE FOUND WITHIN THE TEXAS DUNE PROTECTION MANUAL EDITION 6, GLO'S TEXAS BEACH ACCESSIBILITY GUIDE, AND TEXAS ACCESSIBILITY STANDARDS (TAS).

VEGETATION NOTES:

1. THE PLANTS THAT SHALL BE USED CAN BE IDENTIFIED AS BITER PENICUM AND SEA OATS, BUT ADDITIONAL PLANTS CAN BE USED WITHOUT THE PURPOSE OF STABILIZING THE SAND DUNES.
2. BETWEEN THE BITTER PANICUM AND SEA OATS, THERE SHALL BE AT LEAST 2-3 FEET OF SEPARATION BETWEEN THEIR CENTERS. THE RECOMMENDED PLANTING PERCENTAGE BETWEEN BOTH PLANTS IS 20% SEA OATS AND 80% BITTER PANICUM.
3. THE MOST OPTIMAL TRANSPORTATION DATES FOR SUCH PLANTS FALL IN THE MONTHS OF FEBRUARY, MARCH, OR APRIL.

DUNE WALKOVER NOTES:

1. THE LOWEST LEVEL OF THE WALKOVER MUST BE OF SUFFICIENT ELEVATION TO ACCOMMODATE THE EXPECTED INCREASES IN DUNE HEIGHT. AT A MINIMUM, THE LOWEST LEVEL OF A DUNE WALKOVER WITH A WIDTH OF FOUR FEET OR LESS SHOULD BE CONSTRUCTED AT A HEIGHT OF AT LEAST THREE FEET ABOVE THE HIGHEST POINT OF THE TALLEST DUNE CREST BENEATH AND IMMEDIATELY ADJACENT TO THE DUNE WALKOVER.
2. SPACE THE SLATS FORMING THE DECK OF THE WALKOVER 1/2 INCH APART SO THAT SUNLIGHT AND RAINFALL CAN PENETRATE TO VEGETATION BELOW AND SO THAT SAND WILL NOT ACCUMULATE ON THE DECK.
3. PLACE THE SUPPORTING PIERS AS FAR APART AS POSSIBLE ALONG THE LENGTH OF THE STRUCTURE. A DISTANCE OF AT LEAST SIX FEET BETWEEN PAIRS OF PIERS IS RECOMMENDED. IMPLANT THE PIERS AT LEAST THREE FEET IN THE GROUND TO ENSURE STABILITY. A DEPTH OF FIVE FEET OR MORE IS ADVISABLE TO ALLOW FOR EROSION AROUND THE PIERS DURING STORMS. INSTALL THE PIERS WITH A HAND AUGER OR POSTHOLE DIGGER RATHER THAN WITH A TRACTOR.
4. WALKOVER PIERS SHOULD NOT BE SET WITH CEMENT, SINCE THE USE OF CONCRETE TO STABILIZE DUNE WALKOVER PILINGS IS PROHIBITED. ANY DAMAGES TO THE DUNE AREA MUST BE AUTHORIZED BY THE LOCAL GOVERNMENT AND REPAIRED AS SOON AS POSSIBLE.
5. PROVIDING HANDRAILS ON BOTH SIDES OF THE WALKOVER IS RECOMMENDED AS A SAFETY MEASURE AND TO DISCOURAGE PEOPLE FROM JUMPING OFF INTO THE DUNES. RAILINGS ARE PARTICULARLY ADVISABLE ON PUBLIC WALKOVERS AND THOSE THAT ARE HIGH ABOVE THE GROUND. RAILINGS SHOULD BE AT LEAST THREE FEET HIGH.
6. THE DEVELOPMENT FOR THE DESIGN OF THE DUNE WALKOVER TOOK INTO CONSIDERATION THE PUBLIC USES OF THE WALKOVER, WHICH REQUIRES ADA AS WELL AS ANY OTHER LOCAL ACCESSIBILITY REQUIREMENTS TO BE MET. BECAUSE OF THIS, A SEPARATE RAMP SECTION THAT BEGINS AND ENDS AT BOTH LANDING ZONES AS WELL AS MEETING ALONG THE APEX OF THE WALKOVER SHALL BE ADDED.
7. WALKOVERS SHOULD BE INSPECTED ON A REGULAR BASIS AND PROMPTLY REPAIRED AS NEEDED. TO AVOID DAMAGE TO DUNES, WORKERS SHOULD ENTER THE DUNE AREA ON FOOT RATHER THAN BY VEHICLE.

LEGEND

- RESTING ZONE/TEMPORARY REMOVABLE FOOTPATH
- CRUSHED GRANITE
- RAMP (1:12 4' WIDE SLOPPED DOWNWARD PATH)
- TW = X.XX' TOP OF WALKWAY
- 4"X4" WITH FOOTING

THE STATE OF TEXAS
COUNTY OF GALVESTON

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

DUNE WALKOVER DETAILS

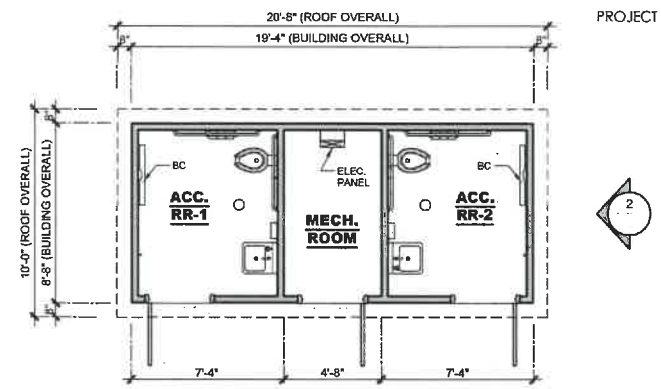
HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4248
281.496.0066
www.huitt-zollars.com

PROJECT NO. R318805.01
DWG SCALE
HORIZONTAL : 1" = 3'
VERTICAL : 1" = 3'

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. NEWBORA
91092
Date: 2/10/24
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

HUITT-ZOLLARS, INC.

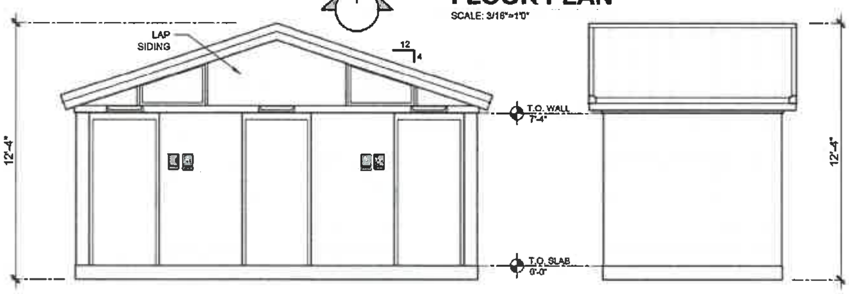
PROJECT REF#: 13296-1/23/2026-0



1

FLOOR PLAN

SCALE: 3/16"=1'0"



ELEVATION 1

SCALE:3/16"=1'-0"

ELEVATION 2

SCALE:3/16"=1'-0"

THIS CONCEPTUAL/ PRELIMINARY DESIGN AND THE 3D RENDERING IS AN ARTISTIC INTERPRETATION OF THE DESIGN. IT IS NOT MEANT TO BE AN EXACT RENDITION OF THE FINISH PRODUCT. SOME ITEMS MAY NOT BE STANDARD AND / OR SUBJECT TO CHANGE DURING PROJECT DEVELOPMENT

PUBLIC RESTROOM COMPANY
Building Better Places To Go.℠

BUILDING TYPE:	RESTROOM BUILDING	REVISION #: -	REVISION DATE: / DRAWN BY:	SHEET#: A-1
PROJECT:	BOLIVAR BEACH POCKET PARK GALVESTON COUNTY, TX	PROJECT #: 13296	START DATE: 1/23/2026 / DRAWN BY: EOR	MAX. PERSON / HOUR: 90 S

Ph: 888-888-2060 | Fax: 888-888-1448

~NOT FOR CONSTRUCTION ~ PRELIMINARY DESIGN DRAWING ONLY ~ DO NOT SCALE, DIMENSIONS PRESIDE

NOTES:

1. RESTROOM MANUFACTURER TO PROVIDE SLAB DESIGN BELOW PREFABRICATED RESTROOM.
2. REFER TO MANUFACTURER NOTES FOR LIMITATIONS REGARDING THE PREFABRICATED RESTROOM.

THE STATE OF TEXAS · COUNTY OF GALVESTON

NOTICE:
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TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

RESTROOM DETAILS

HZ HUITT ZOLLARS

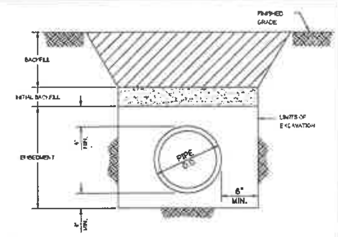
PROJECT NO. R318045.01

HORIZONTAL : N.T.S.
VERTICAL : N.T.S.

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. NERADA
81082
Date: 2/10/26
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

SHEET NO. 07 OF 11

PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



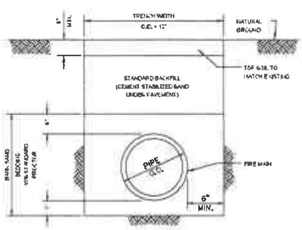
NOTES

1. BACKFILL SHALL BE NATIVE SOIL FREE OF DEBRIS COMPACTED TO 95% STANDARD PROCTOR DENSITY, EXCEPT AS REQUIRED UNDER PAVEMENT.
2. INITIAL BACKFILL SHALL BE UNIFORMLY GRADED MATERIAL (MAXIMUM SIZE 3" DIAMETER) PLACED IN 6" LIFTS AND COMPACTED TO 95% STANDARD PROCTOR DENSITY, EXCEPT AS REQUIRED UNDER PAVEMENT.
3. EMBEDMENT SHALL BE CEMENT STABILIZED SAND (1.5 SACKS PER CUBIC YARD) COMPACTED TO 95% STANDARD PROCTOR DENSITY.
4. UNDER PAVING OR WITHIN 3' OF PAVEMENT, THE INITIAL BACKFILL AND ALL BACKFILL UP TO THE PAVEMENT SUBGRADE SHALL BE CEMENT STABILIZED SAND (1.5 SACKS PER CUBIC YARD) COMPACTED TO 95% STANDARD PROCTOR DENSITY.
5. TRENCH SHORING IN ACCORDANCE WITH OSHA SHALL BE INSTALLED WHERE REQUIRED.
6. SOIL IN THE PIPE ZONE SHALL CONSIST OF NONWATERBEARING COHESIVE SOILS WITH A SHEAR STRENGTH OF 1000 PSF OR GREATER. WHEN WET SAND EXISTS IN THE PIPE ZONE, MODIFIED BEDDING SHALL BE INSTALLED.

SANITARY SEWER

BEDDING AND BACKFILL

N.T.S.



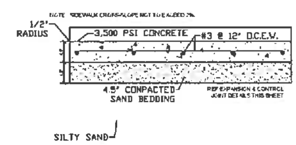
NOTES

1. BACKFILL SHALL BE NATIVE SOIL FREE OF DEBRIS PLACED IN LIFTS, 6" THICK OR LESS, COMPACTED TO 95% STANDARD PROCTOR DENSITY, EXCEPT AS REQUIRED UNDER PAVEMENT.
2. BACKFILL UNDER PAVEMENT AND PUBLIC STREETS SHALL BE CEMENT STABILIZED SAND (1.5 SACKS OF CEMENT PER CUBIC YARD OF SAND) COMPACTED TO 95% STANDARD PROCTOR DENSITY.
3. TRENCH SHORING IN ACCORDANCE WITH OSHA SHALL BE INSTALLED AS REQUIRED.

FIREWATER MAIN

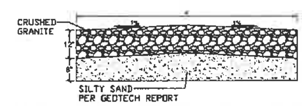
BEDDING AND BACKFILL

N.T.S.



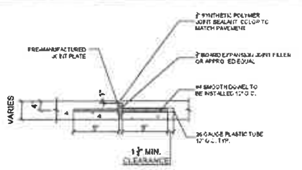
CONCRETE SIDEWALK DETAIL

N.T.S.



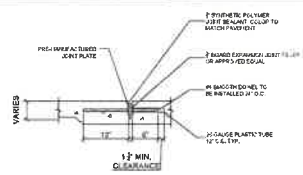
CRUSHED GRANITE WALKING PATH WITHOUT GEOGRID DETAIL

N.T.S.



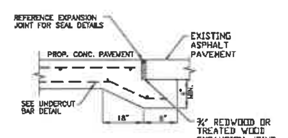
EXPANSION JOINT DETAIL

N.T.S.



SIDEWALK TO PAVEMENT SECTION

N.T.S.



PAVING HEADER DETAIL

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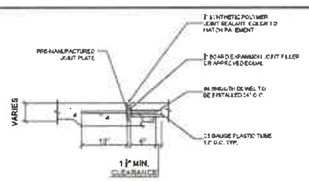
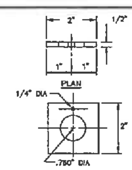


SIDEWALK TO EXISTING SIDEWALK SECTION

N.T.S.



JOINT PLATE DETAIL

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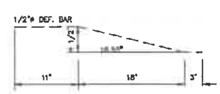


UNDERCUT BAR DETAIL

N.T.S.

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

PAVING DETAILS

HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4248
281.496.0066
www.huitt-zollars.com

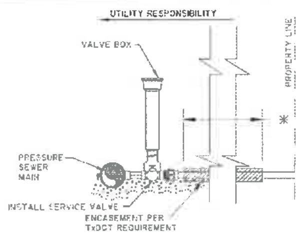
R318066.01
HORIZONTAL : N.T.S.
VERTICAL : N.T.S.

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. HERRERA
81082
Date: 2/10/26
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

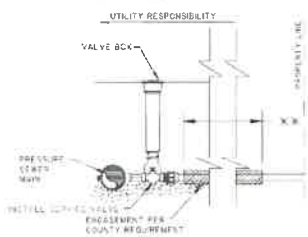
08 OF 11

HUITT-ZOLLARS, INC.

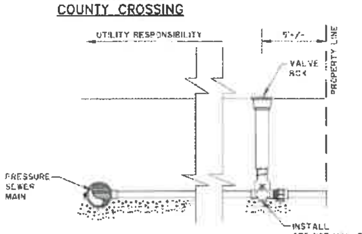
PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



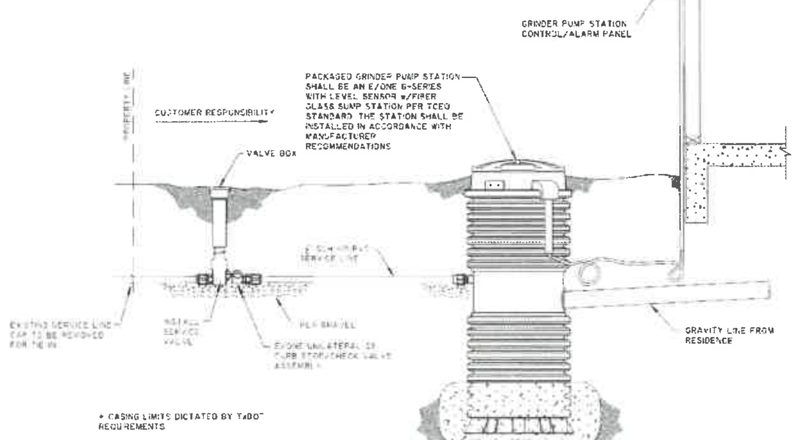
TxDOT CROSSING



COUNTY CROSSING



PRIVATE/RESIDENTIAL



* CASING LIMITS DICTATED BY TxDOT REQUIREMENTS

** CASING LIMITS DICTATED BY GALVESTON COUNTY REQUIREMENTS

NOTES

1. THE SERVICE LATERAL SIZE MAY VARY FROM 1½" TO 4" BASED ON DESIGN CRITERIA
2. TxDOT CONSTRUCTION PERMIT REQUIRED PRIOR TO START OF CONSTRUCTION FOR TxDOT ROAD CROSSING.
3. GALVESTON COUNTY ENGINEER CONSTRUCTION PERMIT REQUIRED PRIOR TO CONSTRUCTION FOR COUNTY ROAD CROSSING.
4. THE DEVELOPER SANITARY SEWER PLANS MUST BE APPROVED BY UNDINE, LLC PRIOR TO START OF CONSTRUCTION AND REQUEST FOR SERVICE

A&S Engineers, Inc.

U N D I N E , L L C

STANDARD SUBDIVISION
SERVICE CONNECTION

THE STATE OF TEXAS — COUNTY OF GALVESTON

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

SERVICE CONNECTION
DETAILS

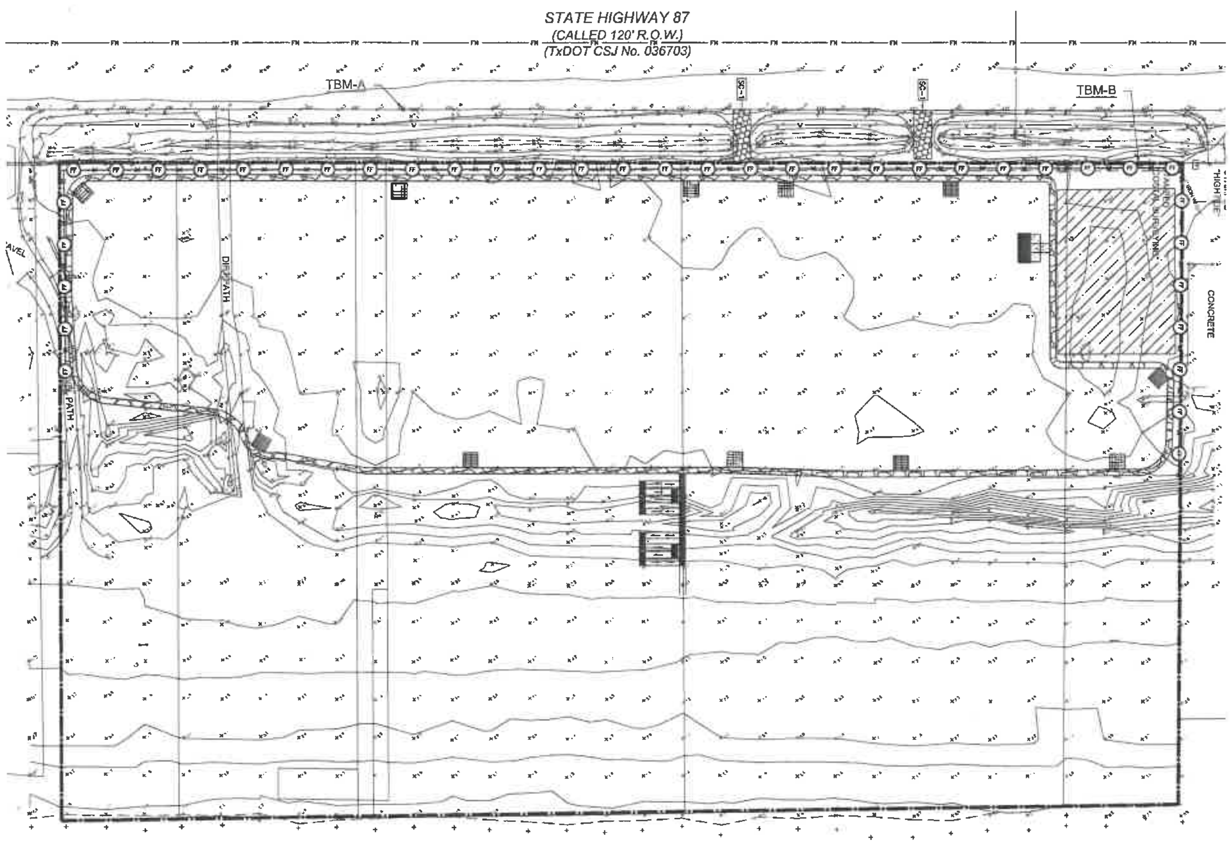
HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4248
281.496.0066
www.huitt-zollars.com

PROJECT NO.
R310065.01
SCALE
HORIZONTAL : N.T.S.
VERTICAL : N.T.S.
SHEET NO.
08 OF 11

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. HERNDON
91082
Date: 2/5/26
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

HUITT-ZOLLARS, INC.

PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



LEGEND

- FILTER FENCE
- SC-1 STABILIZED CONSTRUCTION ACCESS

NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

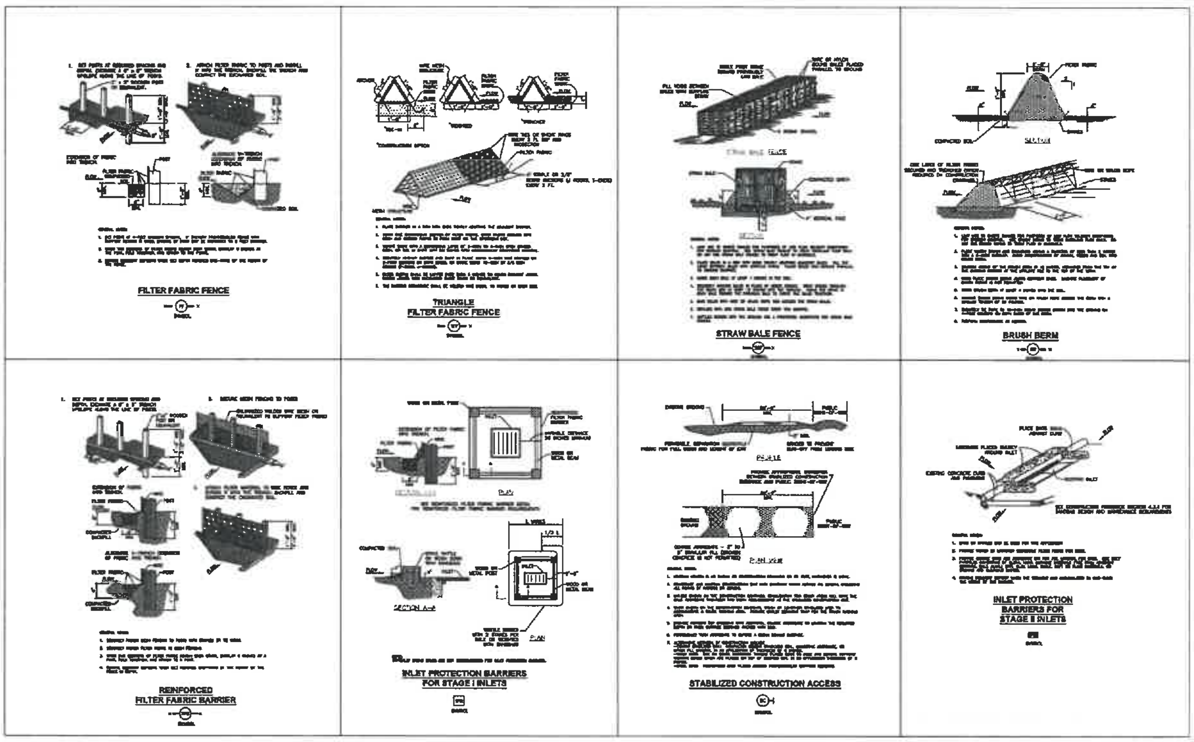
SWPPP

HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4344
281.496.0066
www.huitt-zollars.com

PROJECT NO. R310065.01
HORIZONTAL : 1" = 30'
VERTICAL : N/A
SHEET NO. 10 OF 11

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. NEWADBA
81092
Date: 2/6/24
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

HUITT-ZOLLARS, INC.
PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:



NOTICE:
THE CONTRACTOR SHALL CONTACT THE FOLLOWING AT LEAST 48 HOURS PRIOR TO EXCAVATION IN THIS AREA.
TEXAS ONE - 811 CALL SYSTEM (800) DIG-TESS

REV.	DESCRIPTION	DATE

GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

SWPPP DETAILS

HUITT ZOLLARS
10350 Richmond Avenue, Suite 300
Houston, Texas 77042-4248
281.496.0066
www.huitt-zollars.com

PROJECT NO.
R316065.01
DWG SCALE
HORIZONTAL : N/A
VERTICAL : N/A
DWG NO.
SHEET NO.
11 OF 11

PRELIMINARY
NOT FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES. FOR REVIEW ONLY.
CHAD A. HERMOSA
91092
Date: 1/8/25
Huitt-Zollars, Inc.
Consulting Engineers
Firm Registration No. F-761

PLANS FOR THE CONSTRUCTION OF IMPROVEMENTS TO SERVE:
HUITT-ZOLLARS, INC.

50% OPINION OF PROBABLE CONSTRUCTION COST
GALVESTON COUNTY BOLIVAR BEACH POCKET PARK

ITEM NO.	DESCRIPTION	MEAS. UNIT	QUANTITY	UNIT COST	ITEM COST
GENERAL WORK ITEMS					
01	Mobilization, Demobilization, Insurance and Furnish Performance and Payment Bonds	LS	1	$ 10,000.00	$ 10,000.00
02	Furnish, Implementation, and Management of the SW3P; including Concrete Truck Washout, In Accordance with the Plans and Specifications	LS	1	$ 5,000.00	$ 5,000.00
03	Site Restoration, including but not limited to Hydro-Mulch, In Accordance with the Plans and Specifications	LS	1	$ 7,000.00	$ 7,000.00
				GENERAL WORK ITEMS SUBTOTAL	$ 22,000.00

ITEM NO.	DESCRIPTION	MEAS. UNIT	QUANTITY	UNIT COST	ITEM COST
SITE WORK ITEMS					
04	Prefabed Restroom Building	LS	1	$ 265,000.00	$ 265,000.00
05	Concrete Slab	LS	1	$ 22,000.00	$ 22,000.00
06	4-1/2-Inch Concrete Sidewalk	SF	144	$ 10.00	$ 1,440.00
07	Crushed 4' Wide Granite Path, 12-Inch	SY	781	$ 40.00	$ 31,244.71
08	Crushed Granite Pads, 12-Inch	SY	132	$ 40.00	$ 5,262.22
09	Square Cantilever Shade Structure (12' x 12' x 8')	EA	6	$ 8,334.00	$ 50,004.00
10	Picnic Table	EA	6	$ 1,200.00	$ 7,200.00
11	Dune Walkover	LS	1	$ 50,000.00	$ 50,000.00
12	5/8-Inch Diameter Water Taps and Copper Service Line with Meter Box, Long Side	EA	1	$ 1,500.00	$ 1,500.00
13	5/8-Inch Water Service Line	LF	110	$ 25.00	$ 2,750.00
14	1 1/2-Inch FM Service Connection	EA	1	$ 1,800.00	$ 1,800.00
15	4-Inch Sanitary Sewer Gravity Line. With Cleanout	LF	27	$ 55.00	$ 1,463.00
16	1 1/2-Inch SCH 40 PVC Service Line	LF	116	$ 30.00	$ 3,474.60
17	E-One G-Series Packaged Pump Station	EA	1	$ 9,000.00	$ 9,000.00
18	4-Inch Casing for FM Service Line	LF	63	$ 67.00	$ 4,221.00
				SITE WORK ITEMS SUBTOTAL	$ 456,359.53
				GRAND TOTAL	$ 478,359.53
				CONTINGENCY (5%)	$ 23,917.98
				BASE BID TOTAL	$ 502,277.51

END OF SECTION

HZ HUITT ZOLLARS