DISCLAIMER OF LIABILITY

The Owner and Architect/Engineer disclaim any responsibility for the accuracy, true location and extent of the soils investigation that has been prepared by others. They further disclaim responsibility for interpretation of that data by Bidders. Report of the soils investigation is bound in this project manual for the Bidders' convenience only and IS NOT AND SHALL NOT CONSTITUTE PART OF THE BIDDING AND CONTRACT DOCUMENTS.

TOLUNAY-WONG
ENGINEERS

GEOTECHNICAL ENGINEERING STUDY
NEW PRE-ENGINEERED METAL BUILDING AND PARKING LOT
DEPARTMENT OF TEXAS PARKS AND WILDLIFE
POWDERHORN WILDLIFE MANAGEMENT AREA
PORT O'CONNOR, TEXAS

Prepared for:

Richter Architects
201 S. Upper Broadway
Corpus Christi, Texas 78401

Prepared by:

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June 24, 2025

Project No. 25.53.064 / Report No. 36529

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June 24, 2025

Richter Architects
201 S. Upper Broadway
Corpus Christi, Texas 78401

Attn: Ms. Maya Richter Hernandez, AIA
mrichter@richterarchitects.com

Ref: Geotechnical Engineering Study
New Pre-Engineering Metal Building and Parking Lot
Department of Texas Parks and Wildlife
Powderhorn Wildlife Management Area
Port O'Connor, Texas
TWE Project No. 25.53.064 / Report No. 36529

Dear Ms. Richter,

Tolunay-Wong Engineers, Inc. (TWE) is pleased to submit this report of our geotechnical engineering study for the above referenced project. This report contains a description of the field program and laboratory services performed for this study as well as soil boring logs. Also included in this report are our geotechnical design and construction recommendations for the proposed new pre-engineered metal building and parking lot.

We appreciate the opportunity to work with you on this phase of the project and look forward to the opportunity of providing additional services as the project progresses. If you have any questions or comments regarding this report or if we can be of further assistance, please contact us.

Sincerely,

TOLUNAY-WONG ENGINEERS, INC.
Texas Board of Professional Engineers Firm Registration Number F-000124

Justin Buchen
Geotechnical Staff Engineer

Don R. Rokohl, P.E.
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STATE OF TEXAS
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PROFESSIONAL ENGINEER
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JB/DRR/jb

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1 INTRODUCTION AND PROJECT DESCRIPTION

Introduction

This report presents the results of our geotechnical engineering study performed for the new pre-engineered metal building and parking lot at the Texas Department of Parks and Wildlife (TPWD) Powderhorn Wildlife Management Area near Port O'Connor, Texas. Our geotechnical engineering study was conducted in accordance with TWE Proposal No. P25-C008, dated January 16, 2025, and authorized by Ms. Maya Richter Hernandez, AIA on May 28, 2025 by way of executed proposal.

Project Description

We understand that the project will include a single-story pre-engineered building with an approximate 3,500 ft^2 footprint and a parking area at the TPWD Powderhorn Wildlife Management Area in Port O'Connor, Texas. Furthermore, we understand that the new building will be a metal frame and siding structure with a slab-on-grade foundation. As such, the foundation loads will be relatively light and are assumed to be up to 50 kips column loads and up to 3 kips per lineal foot wall loads. We assume the planned parking lot and driveway will carry only automobile and light truck traffic with exception of the route to the trash dumpster which will carry traffic from weekly heavy trash disposal truck(s).

2 PURPOSE AND SCOPE OF SERVICES

Our scope of services performed for the project consisted of:

1. Drilling two (2) soil borings to depths of 40-ft within the project site to evaluate subsurface stratigraphy and groundwater conditions;

2. Performing geotechnical laboratory tests on recovered soil samples to evaluate the physical and engineering properties of the strata encountered;

3. Providing geotechnical engineering design recommendations for suitable foundation systems to support the new pre-engineered metal building. Based on anticipated foundation loads, recommendations for a shallow foundation system will be included;

4. Providing estimated potential vertical rise (PVR) for the site and means of reducing the PVR to levels acceptable for shallow foundations and slab-on-grade construction. Geotechnical information regarding the use of slab-on-grade support for lightly loaded interior floor slabs in the building;

5. Providing geotechnical design recommendations for both flexible (asphalt) and rigid (concrete) pavement sections including subgrade preparation and required component thicknesses; and,

6. Providing geotechnical construction recommendations including site and subgrade preparation, excavation considerations, fill and backfill placement, compaction requirements, foundation installation, and overall quality control monitoring inspection and testing services.

Our scope of services did not include any environmental assessments for the presence or absence of wetlands or of hazardous or toxic materials within or on the soil, air, or water within this project site. Any statements in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions are strictly for the information of the Client. A geological fault study was also beyond the scope of our services associated with this geotechnical engineering study.

3 FIELD PROGRAM

Soil Borings

TWE conducted an exploration of subsurface soil and groundwater conditions at the project site on June 6, 2025, by drilling and sampling two (2) soil borings to depths of 40-ft below natural grade at the time of the field program. The soil boring locations are presented on TWE Drawing Nos. 25.53.064-1 in Appendix A of this report. Drilling and sampling of the soil borings were performed using all-terrain vehicle mounted drilling equipment. Our field personnel coordinated the field activities and logged the boreholes. The boring locations were marked at the site by TWE. The final latitude and longitude coordinates for each boring were determined using a handheld GPS unit and are presented on the boring logs in Appendix B of this report.

Drilling Methods

Field operations were performed in general accordance with *Standard Practice for Soil Investigation and Sampling by Auger Borings* [American Society for Testing and Materials (ASTM) D 1452]. The soil borings were drilled using all-terrain vehicle mounted drilling equipment with a rotary head. The boreholes were advanced using dry-auger and hollow stem drilling methods. Samples were obtained intermittently from existing ground surface to a depth of 12-ft, at the 13-ft. to 15-ft. depth interval, and at intervals of 5-ft. thereafter until the boring completion depths were reached.

Soil Sampling

Samples of cohesionless and semi-cohesionless soils were collected with the standard penetration test (SPT) sampler driven 18-in by blows from a 140-lb hammer falling 30-in in accordance with the *Standard Test Method for Standard Penetration Test (SPT) and Spilt-Barrel Sampling of Soils (ASTM D 1586)*. The number of blows required to advance the sampler three (3) consecutive 6-in depths are recorded for each corresponding sample on the boring logs. The N-value, in blows per foot, is obtained from SPTs by adding the last two (2) blow count numbers. The compactness of cohesionless and semi-cohesionless samples are inferred from the N-value. The samples obtained from the split-barrel sampler were visually classified, placed in moisture sealed containers, and transported to our laboratory.

The recovered soil sample depths with corresponding SPT blowcounts are presented on the boring logs in Appendix B.

Boring Logs

Our interpretation of the general subsurface materials and groundwater conditions at the project boring locations are included on the boring logs. Our interpretations of the soil types throughout the boring depths and the location of strata changes were based on visual classifications during field sampling and laboratory testing in accordance with *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D 2487)* and *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) (ASTM D 2488)*.

The boring logs include the type and interval depth for each sample along with their corresponding SPT blowcounts. The boring logs and a key to terms and symbols used on boring logs are presented in Appendix B.

Groundwater Measurements

Groundwater level measurements were attempted in the open boreholes during drilling. Water level readings were attempted in the open boreholes when water was first encountered and after a fifteen (15) minute period. The groundwater observations are summarized in Section 5.4 of this report entitled *"Groundwater Observations."*

4 LABORATORY SERVICES

A laboratory testing program was conducted on selected samples to assist in classification and evaluation of the physical and engineering properties of the soils encountered in the project borings. Laboratory tests were performed in general accordance with *ASTM International* standards. The types of laboratory tests performed are presented in Table 4-1. A brief description of the testing methods is listed below.

Table 4-1: Laboratory Testing Program

Test Description	Test Method
Amount of Material in Soils Finer than No. 200 Sieve	ASTM D 1140
Water (Moisture) Content of Soil	ASTM D 2216
Liquid Limit, Plastic Limit and Plasticity Index of Soils	ASTM D 4318

Amount of Materials in Soils Finer than No. 200 (75-µm) Sieve (ASTM D 1140)
This test method determines the amount of materials in soils finer than the No. 200 (75-µm) sieve by washing. The loss in weight resulting from the wash treatment is presented as a percentage of the original sample and is reported as the percentage of silt and clay particles in the sample.

Water (Moisture) Content of Soil by Mass (ASTM D 2216)
This test method determines water (moisture) content by mass of soil where the reduction in mass by drying is due to loss of water. The water (moisture) content of soil, expressed as a percentage, is defined as the ratio of the mass of water to the mass of soil solids. Moisture content may provide an indication of cohesive soil shear strength and compressibility when compared to Atterberg Limits.

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D 4318)
This test method determines the liquid limit, plastic limit and the plasticity index of soils. These tests, also known as Atterberg limits, are used from soil classification purposes. They also provide an indication of the volume change potential of a soil when considered in conjunction with the natural moisture content. The liquid limit and plastic limit establish boundaries of consistency for plastic soils. The plasticity index is the difference between the liquid limit and plastic limit.

Standard geotechnical laboratory test results and soil properties encountered in the project borings are presented on the boring logs in Appendix B of this report.

5 SITE AND SUBSURFACE CONDITIONS

General

Our interpretations of soil and groundwater conditions within the project site are based on information obtained at the soil boring locations only. This information has been used as the basis for our geotechnical information included in this report. Subsurface conditions may vary at areas not explored by the soil borings. Significant variations at areas not explored by the soil borings will require reassessment of our geotechnical information.

Site Description and Surface Conditions

The site is located at the TPWD Powderhorn Wildlife Management Area in Port O'Connor, Texas. At the time of our field investigation, the site was covered by native grasses and weeds with an existing metal building to east northeast. The site appears to be relatively flat with poor drainage.

Subsurface Soil Stratigraphy and Properties

The general soil profile encountered in the project borings consisted of variations of very loose to medium dense cohesionless and semi-cohesionless POORLY GRADED SAND with SILT (SP-SM), POORLY GRADED SAND (SP), SILT with SAND (ML), and SILTY SAND (SM).

Results of Atterberg Limit tests on selected samples from the project borings indicated the soils were non-plastic. In-situ moisture contents of the soils ranged from 17% to 28%. The amount of material passing the No. 200 ranged from 2% to 82% within the selected soil sample tested for grain size distribution.

Detailed descriptions of the soils encountered at the boring locations are presented on the boring logs in Appendix B.

Groundwater Observations

Groundwater was encountered at a depth of 8-ft while dry-augering the borings. The water levels were at a depth of about 7-ft when measured about 15 minutes later.

Groundwater levels may fluctuate with climatic and seasonal variations as well as tidal changes and should be verified before construction. Accurate determination of the static groundwater level is typically made with a standpipe piezometer. Installation of a piezometer to evaluate the long-term groundwater conditions was not included within the current scope of services.

Shrink / Swell Potential

The tendency for a soil to shrink and swell with change in moisture content is a function of clay content and type which are generally reflected in soil consistency as defined by Atterberg Limits. A generalized relationship between shrink/swell potential and soil plasticity index (PI) is shown in Table 5-1 below.

Table 5-1: General Relationship Between PI and Shrink/Swell Potential

P.I. Range	Shrink/Swell Potential
0 – 15	Low
15 – 25	Medium
25 – 35	High
> 35	Very High

The amount of expansion that will actually occur with increase in moisture content is inversely related to the overburden pressure. Therefore, the larger the overburden pressure, the smaller the amount of expansion. Near-surface soils are thus most susceptible to shrink/swell behavior because they experience low magnitude of overburden. Overall, the soils at this site possess negligible to low shrink/swell potential.

6 SHALLOW FOUNDATION RECOMMENDATIONS

Discussion

The project involves construction of a one story, 3,500-ft^2, pre-engineered metal building at the TPWD Powderhorn Wildlife Management Area in Port O'Connor, Texas. We understand that the new structure will impose no more than 50 kips column loads to the foundation and wall loads of no more than 3 kips per lineal foot.

The subsurface conditions at the site include cohesionless/semi-cohesionless sands throughout the boring depths. The soils in the zone of seasonal moisture change (the upper 10-ft to 12-ft) are non-expansive and are subject to negligible to low shrink-swell movements with changes in moisture content. Consequently, shallow supported continuous strip footings (beams) and individual spread footings (isolated) or stiffened, conventionally reinforced, monolithic, waffle-type beam and slab-on-grade foundations can be considered for the support of the new building so long as any fill materials added to the site consist of low plasticity structural select fill materials. Recommendations for design and construction of shallow strip and/or spread footings or waffle-type beam and slab-on-grade foundation are provided below.

Earthwork recommendations are provided in Section 7 of this report entitled *"Earthwork Considerations"*.

Shallow Foundations and Floor Slabs

Shallow foundations and interior floor slabs-on-grade can be considered for support of the metal building. Recommendations for design and construction of these foundation components are provided below.

Potential Vertical Rise

Based on the results of our field and laboratory programs, the potential vertical rise (PVR) for existing subsurface stratigraphy at the site as determined by Test Method TEX-124-E is estimated to be less than 1.0-in for "existing" conditions. To maintain a PVR of less than 1-inch, any fill material placed in the building area must be composed of non-expansive structural fill material meeting classification and plasticity requirements in Section 7.1 below. The use of expansive fill material in the building areas could result in an increase of PVR to possibly over 1.0-in and should not be allowed.

It is generally accepted that a primary source of foundation distress is soil movements associated with shrink/swell behavior of the underlying supporting stiff soils. As discussed above measures to reduce PVR at this site will not be necessary in the design of the shallow foundation system for the proposed new structure. Typically, it is desirable to reduce potential movements below shallow foundations to the order of one (1)-in. or less. The PVR presented above can be maintained with the placement of a structural select fill pad for site grading purposes.

Material and compaction requirements for non-expansive structural fill are provided below in Section 7.1 of this report. It is recommended that select fill be used for elevation of the foundation/building pad above existing grade at least 6 inches to provide positive drainage away from the foundation and structure.

We recommend that site preparation also include:

- Removal of at least the top 8-in of existing soils for site stripping purposes to at least 5-ft. beyond the outside perimeter of the foundation.
- After achieving specified subgrade elevation to receive fill, proof-roll exposed subgrade and compact as indicated below.
- After compaction, testing, and acceptance of subgrade, immediately place and compact non-expansive structural fill material to at least 5-ft beyond the outside perimeter of the foundation.
- Maintain moisture in structural fill pad until the concrete foundation is constructed.

It should be noted that these methods for maintaining/reducing shrink/swell movements are designed for normal seasonal changes in soil moisture content of the subgrade soils. Excessive shrink/swell movements can be expected if increases in soil moisture content occur as a result of broken water and sewer lines, improper drainage of surface water, shrubbery and trees planted near the foundation slab, excessive lawn or shrubbery irrigation, or decreases in soil moisture during prolonged drought(s).

Special care should be taken not to allow the exposed subgrade soils to become extremely wet or extremely dry of the existing moisture content. Therefore, it is vitally important that delays between excavation and fill placement be avoided. If construction occurs during rainy weather and the exposed subgrade soils are allowed to become wet or saturated, removal and replacement of excessively soft, wet soils or cement-treatment should be anticipated. The depth of undercutting should be determined in the field by TWE.

Spread and/or Continuous Footings

Foundation Depth and Allowable Bearing Pressure

The proposed structure placed on shallow footings established at 3.0-ft below finished grade on properly prepared and properly compacted native soils and/or imported non-expansive structural fill soils can be designed using a net allowable bearing pressure of 2,500-psf for strip (continuous) footings and for individual (spread) footings. These net allowable bearing pressures contain a factor of safety of at least 3.0 against a general bearing capacity failure. The footings should have minimum widths of 1.5-ft (strip) and 3-ft (spread) and maximum widths of 3-ft (strip) and 8-ft (spread), respectively.

Any foundation excavations to receive fill material should be properly prepared as described below in Section 7.

Overturning Resistance

The design of shallow footings subjected to vertical loads and overturning moment loads should incorporate a stability ratio as selected by the design structural engineer in accordance with the project specifications. The stability ratio is defined as the ratio of the stabilizing moment to overturning moment. The maximum foundation contact pressure should not exceed the recommended allowable soil bearing pressure given above.

Uplift Resistance

Resistance to vertical uplift force is provided by the weight of the concrete footings and backfill above the footings. We recommend that the ultimate uplift resistance be based on the unit weight of 150 pounds per cubic foot (pcf) for reinforced concrete above static groundwater level and 100 pcf for compacted backfill material above the static groundwater level. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.2 to calculate the allowable uplift resistance.

Lateral Resistance

Horizontal loads acting on shallow footings below grade will be resisted by passive resistance acting on one side of the footings and through coefficient of friction between the natural cohesionless/semi-cohesionless soils or structural select fill material and the concrete acting on the base of the footing. An allowable passive pressure of 150-psf can be used for non-expansive structural select fill used as backfill around the footings. An allowable coefficient of friction value of 0.4 can be used for shallow footings in ample contact with the supporting soils at the recommended bearing depth. These values should provide a factor of safety of about 1.5 with respect to the ultimate values.

Settlement

Estimated settlement for shallow footings placed at the recommended depth is based on proper preparation of the footing subgrade. This estimate assumes a uniformly loaded foundation with pressures that are no greater than the recommended allowable bearing pressures and assumes the foundations are designed and constructed in accordance with the recommendations provided in this report.

Settlement of properly designed and installed footings is estimated to be less than 1-in. Differential settlement across the foundation would be no more than three-quarters (3/4) of the total settlement.

Floor Slabs

Interior floor slabs cast on grade can be used for the new building. Support of the floor slab should be either properly compacted native soils or properly placed and compacted imported non-expansive structural select fill material. The non-expansive structural select fill material should meet the plasticity and classification as provided below in Section 7 Earthwork Considerations of the report. The floor slab should be designed and constructed for potential movements of up to one (1) in. at this site. We recommend the placement of a vapor barrier between the bottom of the floor slab and the supporting subgrade soils to eliminate moisture movement up through the floor slab.

Stiffened, Conventionally Reinforced, Monolithic, Waffle-Type Beam and Slab-on-Grade Foundations

A stiffened, conventionally reinforced, monolithic, waffle-type beam and slab-on-grade foundation should be designed and constructed to resist movements of the supporting soils without inducing distress to the superstructure or foundation. The rigidity of the foundation is developed by grade beams that begin at perimeter of the foundation and criss-cross the interior of the foundation forming a "waffle" pattern. The foundation is often designed using the BRAB or WRI/CRSI design methodologies. These methods are similar and design parameters for each are provided below.

The conventionally reinforced, monolithic beam and slab-on-grade foundation can be designed using the BRAB design parameters presented in Table 6-1 below.

Table 6-1: BRAB Design Parameters	
Effective Plasticity Index (PI_{eff})	15
Climatic Rating (C_w)	18
Support Index (C)	1.0
Undrained Shear Strength (ksf)	1.25

Alternatively, the foundations can be designed using the WRI/CRSI design parameters presented in Table 6-2 below.

Table 6-2: WRI/CRSI Design Parameters	
Effective Plasticity Index (PI_{eff})	15
Climatic Rating (C_w)	18
1- C	0.0

The design parameters presented above in Tables 6-1 and 6-2 are based on the subsurface stratigraphy that existed at the time of the soil borings.

Grade beams should bear at a minimum depth of 3-ft below finished grade in properly prepared native or fill soils. Grade beams can be designed as strip footings using an allowable bearing pressure of 2,500 pounds per square foot (psf). Concentrated loads can be supported by enlarged areas that are designed as spread footings. Spread footings founded at a depth of at least 3-ft below finished grade can be designed using an allowable bearing pressure of 2,500 psf. These allowable bearing pressures should provide a factor of safety of at least 3.0 against soil shear failure. Spread footings and grade beams should have minimum widths of 3-ft and 1.5-ft, respectively, even if the actual bearing pressure is less than the design value. The placement of a vapor barrier below the foundation is recommended to eliminate movement of moisture up through the foundation.

Uplift Resistance

Resistance to vertical uplift force is provided by the weight of the concrete foundation plus the resistance of the foundations to rotation. For this site, the bearing pressures presented above when applied at the foundation toe can be increased by 30% for transient loads such as wind loads. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.2 to calculate the allowable uplift resistance.

Lateral Resistance

Horizontal loads acting on shallow foundations bearing below grade will be resisted by some passive resistance acting on one side of the foundation and through friction acting between the base of the foundation and underlying soils. An allowable passive pressure of 150-psf can be used for the natural undisturbed soils and/or properly compacted structural select fill material used as backfill around the foundation. A coefficient of friction of 0.40 can be used for foundations in good contact with properly prepared native soils and/or properly compacted structural select fill at the recommended foundation depth. These values should provide a factor of safety of about 1.5 with respect to the ultimate values.

Shallow Foundation Construction

If weak, soft, wet, or otherwise unsuitable fill soils are encountered during construction at the recommended foundation depth, we recommend that either the unsuitable materials should be over-excavated, dried, and re-compacted in accordance with requirement for structural fill or the foundation depth be extended to the depth of competent soil lying beneath the unsuitable soil. The footing excavations should not be allowed to remain open for extended periods. If footings must remain open for extended periods, the use of a lean concrete mud mat to reduce moisture changes or other disturbance to the bearing soils should be considered.

Shallow foundations could be either open-cut and formed or machine excavated. Any weak or loose soil encountered during foundation excavation should be removed and replaced with properly compacted non-expansive structural fill prior to form installation and steel placement. Since groundwater was encountered well below the foundation bearing depth during the field investigation, we expect that the shallow foundation excavations can be performed in the dry barring inclement weather.

Excavation in shallow dry semi-cohesionless soils at the site could be subject to caving and sloughing and the bottom of the excavation may be sensitive to disturbance. A 2-in. thick seal slab of lean concrete could be placed over the exposed soils to protect the soils in the bottom of the excavation from disturbance prior to and during foundation construction. For formed foundations, any voids between the foundation and the excavation walls should be backfilled with cement-stabilized sand [one and one-half (1-1/2) sacks of cement per ton of sand] or flowable fill.

The performance of the shallow foundations associated with the project will be highly dependent upon the quality of construction. Thus, it is recommended that shallow foundation construction be monitored by a representative of TWE experienced in quality control testing and inspection procedures to help evaluate foundation construction. TWE would be pleased to develop a plan for shallow foundation monitoring to be incorporated in the overall quality control program.

7 EARTHWORK CONSIDERATIONS

Subgrade Preparation and Structural Select Fill

Areas designated for new construction should be stripped of all surface vegetation, loose topsoil, sidewalks, and major root systems. Tree stumps shall be completely removed and backfilled, if applicable. Any large, open subgrade areas to receive fill soils or new foundations should be proof rolled with at least a 20-ton pneumatic roller, loaded dump truck, or equivalent, to detect weak areas. Such weak areas should be removed and replaced with soils exhibiting similar classification, moisture content, and density as the adjacent in-place soils. Subsequent to proof rolling, and just prior to placement of structural fill or foundation construction, the exposed subgrade should be compacted to at least 95% of the maximum dry density at a moisture near (-2% to +3%) the optimum moisture in accordance with Standard Proctor (*ASTM D 698*) procedures.

Proper site drainage should be maintained during construction so that ponding of surface runoff does not occur and cause construction delays and/or inhibit site access. Due to the nature of the subgrade, the cohesionless soils can become dry and loose. If the subgrade becomes dry and loose, the subgrade should be watered and recompacted to establish a stiff bearing surface.

The maximum loose thickness for each lift will depend on the type of compaction equipment used. Recommended fill layer thickness are summarized in Table 7-1 below.

Table 7-1: Compaction Equipment and Maximum Lift Thickness

Compaction Equipment	Maximum Lift Thickness
Mechanical Hand Tamper	4.0-in
Pneumatic Tired Roller	6.0-in
Tamping Foot Roller	8.0-in
Sheepsfoot Roller	8.0-in

Non-expansive structural select fill (structural fill) for this project should consist of a clean low-plasticity sandy clay (CL) or clayey sand (SC) material with a liquid limit of less than 40 and a plasticity index between 7 and 20. The structural fill should be placed in thin lifts, not exceeding 8-in. loose measure, moisture conditioned to between -2% and +3% of optimum moisture content, and compacted to a minimum 95% of the maximum dry density as determined by *ASTM D 698* (Standard Proctor).

Clean on-site native soils can also be considered for use as non-expansive structural select fill material provided the material is evaluated by the project geotechnical engineer before use. This material should be placed and compacted in a manner similar to that for non-expansive structural select fill material.

Prior to any filling operations, samples of the proposed borrow materials should be obtained for soil classification and laboratory moisture-density testing. The tests will provide a basis for evaluation of fill compaction by in-place density testing. A qualified soil technician should perform sufficient in-place density tests during the earthwork operations to verify that proper levels of compaction are being attained.

Drainage

The performance of the foundation system for the proposed structure will not only be dependent upon the quality of construction but also upon the stability of the moisture content of the near surface soils. Therefore, we highly recommend that site drainage be developed so that ponding of surface runoff near the building does not occur. Accumulations of water near the foundation could cause significant moisture variations in the soils adjacent to the foundations thus increasing the potential for structural distress.

8 PAVEMENT DESIGN RECOMMENDATIONS

Pavement Design Criteria

For the proposed use of the parking lot and driveways and accounting for the shallow subsurface soil conditions encountered in the borings, either a flexible or a rigid pavement system can be considered. Since detailed traffic loads and frequencies were not available at the time of this report, we have assumed traffic frequencies and loading for similar projects that have been completed in the past. The assumed traffic frequencies and loads used to design pavement sections for this project are presented in Table 8-1 below.

Table 8-1: Vehicle Classification and Traffic Loading

Pavement Area	Traffic Design Index	Description
Light-Duty Pavements	DI-1	Light traffic: few vehicles heavier than passenger cars; no regular use by heavily-loaded two (2) axle trucks or larger
Heavy-Duty Pavements	DI-2	Heavy traffic: Same as Light Duty traffic plus no more than five (5) delivery vans, and two (2) trash disposal trucks per week. No use by heavily-loaded trucks with more than three (3) axles

Based on estimated traffic conditions and methods found in AASHTO's, Guide for Design of Pavement Structures, design recommendations for flexible and rigid pavement sections are provided in the following sections of this report. A pavement design life of 20 years was used to determine the project 18-kip ESALs used in analysis. The traffic conditions presented above should be verified by the civil design engineer. TWE should be contacted for possible further recommendations if actual traffic conditions will vary from those presented above.

It should be noted that the pavement systems provided here-in were derived based on general soil characterizations of the subgrade. No specific testing (such as CBR's, resilient modulus tests, etc.) was performed for this project to evaluate the support characteristics of the subgrade.

Flexible Pavement Design

The primary design requirements needed for flexible pavement design according to the Pavement Design Guide included the following:

- Material Layer Coefficient;
- Soil Resilient Modulus, psi;
- Serviceability Indices;
- Drainage Coefficient;
- Overall Standard Deviation;
- Reliability, %; and,
- Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)

In our analysis, we assumed U.S. climatic region I (wet and no freeze characteristics), the values used for our analysis are presented in Table 8-2 below.

Table 8-2: Flexible Pavement Design Values

Description		Value
Material Coefficients	Hot Mix Asphalt Concrete (HMAC), Type D	0.44
	Crushed Limestone (Type A, Grade 1-2) [CLS]	0.14
	Properly Compacted Cement Stabilized Subgrade (CSS)	0.08
Serviceability Indices	Initial	4.2
	Terminal	2.5
Soil Resilient Modulus		3,000-psi
Drainage Coefficient		1.0
Overall Standard Deviation		0.45
Reliability		80
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL) – Light-Duty Pavement (DI-1)		54,600
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL) – Heavy-Duty Pavement (DI-2)		443,700
Structural Number Required – DI-1		2.83
Structural Number Required – DI-2		3.99

Table 8-3 on the following page provides the recommended minimum typical pavement section derived from our analysis using the AASHTO Guide.

Table 8-3: Recommended Minimum Typical Flexible Pavement Thicknesses				
Traffic Design Index	**HMAC, Type D**	**CLB**	**CSS**	**SN**
DI-1	2.5-in	8.0-in	8.0-in	2.94
DI-2	4.0-in	12.0-in	8.0-in	4.16

HMAC = Hot Mix Asphalt Concrete
CLB = Crushed Limestone Base (Type A, Grade 1-2)
CSS = Properly Compacted Cement Stabilized Subgrade
SN = Structural Number Provided by Pavement

Rigid Pavement Design

The primary design requirements needed for rigid pavement design according to the *AASHTO Guide* include the following:

- 28-day Concrete Modulus of Rupture, psi;
- 28-day Concrete Elastic Modulus, psi;
- Effective Modulus of Subgrade Reaction, pci (k-value);
- Serviceability Indices;
- Load Transfer Coefficient;
- Drainage Coefficient;
- Overall Standard Deviation;
- Reliability, %; and,
- Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)

In our analysis, we assumed U.S. climatic region I (wet and no freeze characteristics). The values used for our analyses are presented in Table 8-4 on the following page.

Table 8-4: Rigid Pavement Design Values		
Description		**Value**
28-day Concrete Modulus of Rupture (Mr)		500-psi
28-day Concrete Elastic Modulus		4,066,800-psi
Effective Modulus of Subgrade Reaction		50-pci
Serviceability Indices	Initial	4.5
	Terminal	2.5
Load Transfer Coefficient		3.2
Drainage Coefficient		1.0
Overall Standard Deviation		0.39
Reliability		80
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL) – Light-Duty Pavement (DI-1)		54,600
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL) – Heavy-Duty Pavement (DI-2)		443,700

Table 8-5 below provides the recommended minimum typical pavement section derived from our analysis using the AASHTO Pavement Design Guide.

Table 8-5: Recommended Minimum Typical Rigid Pavement Thicknesses		
Traffic Design Index	**RC**	**CSS**
DI-1	5.0-in	8.0-in
DI-2	7.0-in	8.0-in

RC = Reinforced Portland Cement Concrete
CSS = Properly Compacted Cement Stabilized Subgrade

Reinforcing steel consisting of deformed steel rebar should be used in concrete pavement. Thickness is based on concrete flexural strength, soil modulus and traffic volume. Selection of steel is dependent on joint spacing, slab thickness and other factors as discussed in Portland Cement Association publications. The following suggested guidelines for the concrete pavement should be modified by the civil-structural engineer based upon the actual configuration of the pavement layout and published Portland Cement Association and ACI articles. Table 8-6 on the following page presents these guidelines.

Table 8-6: Rigid Pavement Components	
Component	**Description**
Minimum Reinforcing Steel	#3 bars should be spaced at 18-in on centers in both directions.
Minimum Dowel Size	3/4-in bars, 18-in in length, with one (1) end treated to slip with spacing of 12-in on centers at each joint.
Control Joint Spacing	Maximum control joint spacing should be 15-ft. If sawcut, control joints should be cut as soon as the concrete has hardened sufficiently to permit sawing without excessive raveling which is usually within four (4) to twenty-four (24) hours of concrete placement.
Isolation / Expansion Joints	Expansion joints should be used in areas adjacent to structures, such as manholes and walls.

Pavement Section Materials

Hot Mix Asphalt Concrete (HMAC), Type D

HMAC, Type D should conform to Item 340, "Dense-Graded Hot-Mix Asphalt" of the Texas Department of Transportation (TxDOT) 2004 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges. The HMAC should provide a minimum tensile strength (dry) of 85 to 200 psi when tested in accordance with TxDOT Test Method Tex-226-F, and should be compacted at 96% of the theoretical density as determined from the asphaltic mixture design prepared in accordance with TxDOT Test Method Tex-207-F "Determining Density of Compacted Bituminous Mixtures". In place density should meet TxDOT Item 340.4 requirements.

Crushed Limestone Base (CLB)

CLB should conform to TxDOT, Item No. 247 "Flexible Base", Type A, Grade 1-2 and should be compacted to 100% of the maximum dry density determined by TxDOT Test Method Tex-113-E within ±2% of the optimum moisture content.

Reinforced Concrete (RC)

RC should be provided in accordance with TxDOT Item 421 "Hydraulic Cement Concrete", 2004. Concrete should be designed to meet a minimum average flexural strength (modulus of rupture) of at least 500-psi at 28-days or a minimum average compressive strength of 4,500-psi at 28-days. Reinforcing steel consisting of deformed steel rebar should be used in accordance with TxDOT Item 440 "Reinforcing Steel."

The first few loads of concrete should be checked for slump, air and temperature on start-up production days to check for concrete conformance and consistency. Concrete should be sampled and strength test specimens [two (2) specimens per test] prepared on the initial day of production and for each 400-yd^2 or fraction thereof of concrete pavement thereafter. At least one (1) set of strength test specimens should be prepared for each production day. Slump, air and temperature tests should be performed each time strength test specimens are made. Concrete temperature should also be monitored to ensure that concrete is consistently within the temperature requirements.

Cement-Stabilized Subgrade (CSS)

Cement stabilization of the subgrade soils is recommended for both flexible pavement section and rigid pavement section included in Table 8-1 through Table 8-4 above. Proper preparation and cement stabilization of the roadway subgrade will improve long-term pavement performance by increasing its stiffness and load carrying capacity.

After completion of necessary stripping and clearing, the exposed soil subgrade should be carefully evaluated by probing and testing. Any unsuitable material (shell, gravel, organic material, wet, soft, or loose soil) still in place should be removed. The exposed soil subgrade should be further evaluated by proofrolling with a heavy pneumatic tired roller, loaded dump truck or similar equipment weighing at least 20-tons to ensure that soft or loose material does not exist beneath the exposed soils. Proofrolling procedures should be observed routinely by a qualified representative of TWE. Any undesirable material revealed should be removed and replaced in a controlled manner with soils similar in classification or select fill.

Once final subgrade elevation is achieved and prior to placement of crushed limestone base material, the exposed surface of the pavement subgrade soil should be scarified to a depth of 8-in for the flexible pavement section and the rigid pavement section and mixed with cement in conformance with TxDOT Item 275 "Cement Treatment (Road-Mixed)". The appropriate quantity of cement required should be determined after the roadway alignment is stripped and subgrade soils are exposed by use of a laboratory soil treatability study in accordance with Tex-120-E "Cement Treated Materials" but no less than 3% cement by dry weight should be used.

Cement used for stabilization should meet the requirement of DMS-4600, "Hydraulic Cement" and TxDOT's "*Hydraulic Cement Quality Monitoring Program (HCQMP)".* The cement stabilized subgrade should be compacted to a minimum 95% of the maximum dry density as determined by Standard Proctor (ASTM D 698) at a moisture content within the range of 0 to +3% above optimum.

Water ponded next to the edge of the pavement will weaken the subgrade and base material layers leading to premature pavement failure. Therefore, good perimeter surface drainage with a minimum 2% slope away from the roadway is recommended.

Pavement Drainage and Maintenance

Good perimeter surface drainage with a minimum 2% slope away from the pavement is recommended. Providing positive drainage away from the pavement and maintaining the pavement to prevent infiltration of water into the subgrade soils is essential. Water ponding adjacent to the pavement will infiltrate the base material and/or subgrade and result in high maintenance costs and premature pavement failure and, therefore, should be avoided. Periodic maintenance should be performed on the pavement sections to seal any surface cracks and prevent infiltration of water into the subgrade.

9 DESIGN REVIEW AND LIMITATIONS

Design Review and Construction Monitoring

Geotechnical Design Review

Geotechnical review of the design drawings and specifications should be performed prior to construction. This review is recommended to check that the geotechnical recommendations and construction guidelines presented herein have been properly interpreted and incorporated into the construction documents.

Construction Monitoring

The performance of the foundations and pavements for this project will be highly dependent on the quality of construction. Thus, it is recommended that construction activities be monitored by an experienced laboratory proficient in quality control testing/inspection procedures. TWE would be pleased to assist in the development of a plan for construction monitoring to be incorporated in the overall quality control program.

Construction surveillance is recommended and has been assumed in preparing our recommendations. These field services are required to check for changes in conditions that may result in modifications to our recommendations. Performance of the foundation system and pavements will be directly related to the Contractor's adherence to the recommendations in this report and the project plans and specifications. Testing should be provided for all site preparation, foundation concrete pours, and pavement construction activities. TWE would be pleased to provide these services to verify that construction has been performed in accordance with the intentions of this report upon request.

Limitations

Scope of Study

The scope of this study, as well as the conclusions and recommendations provided herein, were developed based on our understanding of the project. Assumptions were made when specific information was unknown. Revisions to our conclusions and recommendations could be necessary as a result of any significant project changes or if our assumptions are incorrect. Construction dewatering design, earth retention design, and construction site safety are the responsibility of the Contractor and have not been addressed herein. The scope of our study did not include evaluation of areal fill conditions or geologic faults. In addition, assessment of environmental conditions, including investigation for hazardous materials/pollutants/wastes, regulatory compliance, threatened or endangered species, cultural resources, floodplains, and jurisdictional wetlands were beyond the scope of our study.

Warranty

The professional services that form the basis for this report have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in the same locality. No warranty, expressed or implied, is made as to the professional advice set forth.

Subsurface Variations

Our interpretations of subsurface conditions are based on data obtained at the boring locations only and at the time of our field exploration. Subsurface variations could exist between the boring locations and at areas not explored. The validity of our recommendations is based, in part, on assumptions made about subsurface conditions in areas not explored. Such assumptions can only be confirmed during construction. Therefore, construction observations by qualified geotechnical representatives are recommended to check for variations in subsurface conditions. Significant changes from our assumptions could require modification to our findings and recommendations.

Report Reliance

This report was prepared as an instrument of service for the sole and exclusive use by Richter Architects and/or the Department of Texas Parks and Wildlife subject to the limitations stated herein and with specific application to the referenced project. This report should not be applied for any other purpose or project, expect as described herein.

This report shall remain the property of Tolunay-Wong Engineers, Inc. No third party may use or rely upon the information provided herein without our express written consent. If any party other than Richter Architects and/or the Department of Texas Parks and Wildlife chooses to rely on this instrument without our consent, said party expressly waives any rights it may otherwise have to claim its reliance on this instrument of professional service that resulted in injury, loss, or damage of any kind and will defend and indemnify Tolunay-Wong Engineers, Inc., from any such claim.

Report Distribution

If any changes in the nature, design or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions modified or verified in writing by TWE. TWE is not responsible for any claims, damages or liability associated with interpretation or reuse of the subsurface data or engineering analyses without the expressed written authorization of TWE.

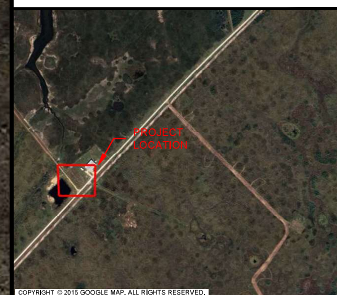
This geotechnical engineering report is intended to assist in the planning and design of the project and is neither appropriate nor intended for use as technical specifications. This report should not be incorporated into the construction documents either directly or by reference. We recommend the Client engage the services of a Design Engineer to perform final design of any site improvements and to prepare construction plans and specifications.

We appreciate the opportunity to be of service during this phase of the project and we look forward to continuing our services during the construction phase and on future projects.

APPENDIX A

SOIL BORING LOCATION PLAN
TWE DRAWING NO. 25.53.064-1

VICINITY MAP



SOIL BORING COORDINATES

BORING	DEPTH	LATITUDE	LONGITUDE
B–1	40'	28°26'55.12"N	96°30'58.89"W
B–2	40'	28°26'55.27"N	96°30'57.61"W

LEGEND

SYMBOL	DESCRIPTION
●	BORING LOCATIONS

TOLUNAY-WONG ENGINEERS

TWE Preliminary Boring Location Plan
TPWD – Powderhorn Wildlife Management Area
Port O'Connor, Texas

Drawn	*J.N.V.*	*5/29/2025*
Checked	*J.B*	
Approved	*D.R.R.*	
Scale	*N.T.S.*	

TWE DRAWING NO. *25.53.064–1*

APPENDIX B

LOGS OF PROJECT BORINGS AND A KEY TO
TERMS AND SYMBOLS USED ON BORING LOGS

LOG OF BORING B-1

PROJECT: TPWD PEMB
Port O'Connor, Tx

CLIENT: Richter Architects
Corpus Christi, Tx

COORDINATES: N 28° 26' 55.12"
W 96° 30' 57.61"

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 40'
Wash Bored: to

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
	Loose tan and brown POORLY GRADED SAND with SILT (SP-SM)		1/6" 4/6" 5/6"	18		NP	NP				5	
	-becomes medium dense and color changes to tan		4/6" 5/6" 6/6"									
5	Medium dense tan POORLY GRADED SAND (SP)		4/6" 8/6" 9/6"	20							4	
			5/6" 5/6" 12/6"									
10	Loose tan and brown POORLY GRADED SAND with SILT (SP- SM)		3/6" 4/6" 4/6"	26		NP	NP				8	
	-becomes very loose and color changes to tan and gray		1/6" 2/6" 2/6"									
15	Very loose gray POORLY GRADED SAND (SP)		1/6" 2/6" 2/6"	25							2	
20			1/6" 1/6" 2/6"									
25			1/6" 1/6" 2/6"	25							4	
30			1/6" 1/6" 1/6"									
35	Very loose gray and brown SILT with SAND (ML)		2/6" 1/6" 3/6"	28							82	

COMPLETION DEPTH: 40 ft
DATE BORING STARTED: 6/6/25
DATE BORING COMPLETED: 6/6/25
LOGGER: B.Harris
PROJECT NO.: 25.53.064

NOTES: Groundwater encountered at a depth of about 8.0-ft while advancing the boring. After 15 minutes, water level measured at a depth of 7.2-ft. Boring was terminated at 40.0-ft

TOLUNAY-WONG ENGINEERS, INC.

LOG OF BORING B-1

PROJECT: TPWD PEMB
Port O'Connor, Tx

CLIENT: Richter Architects
Corpus Christi, Tx

COORDINATES: N 28° 26' 55.12"
W 96° 30' 57.61"

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 40'
Wash Bored: to

ELEVATION (FT) / DEPTH (FT)	SAMPLE TYPE	SYMBOL	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
			-very loose gray and brown SILT with SAND (ML) (cont.)		1/6" 2/6" 2/6"									
40			Bottom @ 40'											
45														
50														
55														
60														
65														
70														

COMPLETION DEPTH: 40 ft
DATE BORING STARTED: 6/6/25
DATE BORING COMPLETED: 6/6/25
LOGGER: B.Harris
PROJECT NO.: 25.53.064

NOTES: Groundwater encountered at a depth of about 8.0-ft while advancing the boring. After 15 minutes, water level measured at a depth of 7.2-ft. Boring was terminated at 40.0-ft

LOG OF BORING B-2

PROJECT: TPWD PEMB
Port O'Connor, Tx

CLIENT: Richter Architects
Corpus Christi, Tx

COORDINATES: N 28° 26' 55.27"
W 96° 30' 57.61"

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 40'
Wash Bored: to

MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
Loose tan and brown POORLY GRADED SAND with SILT (SP-SM)		2/6" 4/6" 4/6"	17		NP	NP				5	
-becomes very loose and color changes to tan		2/6" 2/6" 2/6"									
Loose tan SILTY SAND (SM)		2/6" 3/6" 4/6"	20							17	
-becomes medium dense		4/6" 7/6" 7/6"									
Loose tan POORY GRADED SAND with SILT (SP-SM)		2/6" 4/6" 4/6"	24		NP	NP				10	
-becomes very loose and color changes to tan and gray		1/6" 1/6" 2/6"									
Very loose gray POORLY GRADED SAND (SP)		2/6" 2/6" 2/6"	28							3	
-becomes loose		1/6" 2/6" 5/6"									
		1/6" 2/6" 4/6"	23							4	
		3/6" 4/6" 4/6"									
-becomes very loose		3/6" 2/6" 2/6"	25							4	

ELEVATION (FT) / DEPTH (FT): 5, 10, 15, 20, 25, 30, 35

COMPLETION DEPTH: 40 ft
DATE BORING STARTED: 6/6/25
DATE BORING COMPLETED: 6/6/25
LOGGER: B.Harris
PROJECT NO.: 25.53.064

NOTES: Groundwater encountered at a depth of about 8.0-ft while advancing the boring. After 15 minutes, water level measured at a depth of 7.0-ft. Boring was terminated at 40.0-ft

LOG OF BORING B-2

PROJECT: TPWD PEMB
Port O'Connor, Tx

CLIENT: Richter Architects
Corpus Christi, Tx

COORDINATES: N 28° 26' 55.27"
W 96° 30' 57.61"
SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0' to 40'
Wash Bored: to

ELEVATION (FT) / DEPTH (FT)	SAMPLE TYPE	SYMBOL	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
			-very loose gray POORLY GRADED SAND (SP) (cont.)											
			-color changes to gray and brown		1/6" 1/6" 2/6"									
40			Bottom @ 40'											
45														
50														
55														
60														
65														
70														

COMPLETION DEPTH: 40 ft
DATE BORING STARTED: 6/6/25
DATE BORING COMPLETED: 6/6/25
LOGGER: B.Harris
PROJECT NO.: 25.53.064

NOTES: Groundwater encountered at a depth of about 8.0-ft while advancing the boring. After 15 minutes, water level measured at a depth of 7.0-ft. Boring was terminated at 40.0-ft

KEY TO SYMBOLS AND TERMS USED ON BORING LOGS FOR SOIL

Most Common Unified Soil Classifications System Symbols

- Lean Clay (CL)
- Lean Clay w/ Sand (CL)
- Sandy Lean Clay (CL)
- Fat Clay (CH)
- Fat Clay w/ Sand (CH)
- Sandy Fat Clay (CH)
- Silty Clay (CL-ML)
- Sandy Silty Clay (CL-ML)
- Silty Clayey Sand (SC-SM)
- Clayey Sand (SC)
- Sandy Silt (ML)
- Silty Sand (SM)
- Silt w/ Sand (ML)
- Well Graded Sand (SW)
- Well Graded Sand w/ Gravel (SW-GM)
- Poorly Graded Sand (SP)
- Poorly Graded Sand w/ Silt (SP-SM)
- Silt (ML)
- Elastic Silt (MH)
- Elastic Silt w/ Sand (MH-SP)
- Silty Gravel (GM)
- Clayey Gravel (GC)
- Well Graded Gravel (GW)
- Well Graded Gravel w/ Sand (SP-GM)
- Poorly Graded Gravel (GP)
- Peat

Miscellaneous Materials

- Fill
- Concrete
- Asphalt and/or Base

Sampler Symbols	Meaning
	Pavement core
	Thin - walled tube sample
	Standard Penetration Test (SPT)
	Auger sample
	Sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

2.50	Pocket penetrometer reading in tons per square foot
(T)1.13	Torvane Measurement in tons per square foot
8/6"	Blow count per 6 - in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

Wc (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Qu (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid Limit in percent
PI	Plasticity Index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail @ 15% strain

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIONLESS SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty sands, and sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, and clayey sands.

Relative Density	Typical N_{60} Value Range*
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	Over 50

* N_{60} is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996).

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

Pocket Penetrometer (tsf)	Typical Compressive Strength (tsf)	Consistency	Typical SPT "N_{60}" Value Range**
pp< 0.50	$q_u < 0.25$	Very soft	≤ 2
0.50 ≤ pp< 0.75	$0.25 \leq q_u < 0.50$	Soft	3-4
0.75 ≤ pp< 1.50	$0.50 \leq q_u < 1.00$	Firm	5-8
1.50 ≤ pp< 3.00	$1.00 \leq q_u < 2.00$	Stiff	9-15
3.00 ≤ pp< 4.50	$2.00 \leq q_u < 4.00$	Very Stiff	16-30
pp ≥ 4.50	$q_u \geq 4.00$	Hard	≥ 31

** An "N_{60}" value of 31 or greater corresponds to a hard consistency. The correlation of consistency with a typical SPT "N_{60}" value range is approximate.