GEOTECHNICAL ENGINEERING STUDY

FOR

PROPOSED FISHING HARBOR WASTEWATER TREATMENT PLANT
BROWNSVILLE, CAMERON COUNTY, TEXAS

RABA KISTNER

Raba Kistner, Inc.
800 East Hackberry
McAllen, TX 78501
www.rkci.com

P 956.682.5332
F 956.682.5487
TBPE Firm F-3257
TBAE Firm BR 3427

Project No. ABA24-005-00
April 26, 2024

Mr. Ariel Chavez, P.E., RPLS
Director of Engineering Services
Port of Brownsville – Brownsville Navigation District
1000 Foust Road
Brownsville, Texas 78521

RE: Geotechnical Engineering Study
Proposed Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Road
Approximately 375 ft South of its Intersection with TX-48
Brownsville, Cameron County, Texas

Dear Mr. Chavez:

RABA KISTNER, Inc. (RKI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **RKI** Proposal No. PBA24-010-00, dated March 13, 2024. Written authorization for this study was received by our firm via electronic-mail attachment on Monday, April 8, 2024 by means of the Purchase Order (P.O.) Number PO84387, dated March 14, 2024. The purpose of this study was to drill borings within the subject site, to perform laboratory testing on selected samples to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation and pavement recommendations and construction guidelines for the proposed wastewater treatment plant.

The following report contains our foundation and pavement recommendations and considerations based on our current understanding of the finished floor elevation, design tolerances, and structural and pavement loads. If any of these parameters changes, then different considerations may apply to our recommendations for the foundation and pavement systems, and **RKI** recommends that a meeting be held with the Port of Brownsville – Brownsville Navigation District (CLIENT) and design team to evaluate these alternatives.

We appreciate the opportunity to be of professional service to you on this project. Should you have any questions about the information presented in this report, please call. We look forward to assisting the Port of Brownsville – Brownsville Navigation District during the construction of the project by conducting the construction materials engineering and testing services (quality assurance program).

Very truly yours,

RABA KISTNER, INC.

Saul Cruz, EIT Graduate Engineer	Katrin M. Leonard, P.E. Vice President STATE OF TEXAS KATRIN M. LEONARD 95810 LICENSED PROFESSIONAL ENGINEER Apr. 26, 2024

Attachments

SC/KML

Copies Submitted: Above (1)

GEOTECHNICAL ENGINEERING STUDY

For

PROPOSED FISHING HARBOR WASTEWATER TREATMENT PLANT
ALONG THE EAST SIDE OF FISHERMANS PLACE ROAD
APPROXIMATELY 375 FT SOUTH OF ITS INTERSECTION WITH TX-48
BROWNSVILLE, CAMERON COUNTY, TEXAS

Prepared for

PORT OF BROWNSVILLE – BROWNSVILLE NAVIGATION DISTRICT
Brownsville, Texas

Prepared by

RABA KISTNER, INC.
McAllen, Texas

PROJECT NO. ABA24-005-00

April 26, 2024

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INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed Fishing Harbor Wastewater Treatment Plant, to be located along the east side of Fishermans Place Road, approximately 375 ft south of its intersection with TX-48 in Brownsville, Cameron County, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation and pavement design and construction considerations.

PROJECT DESCRIPTION

We understand that the proposed project will consists of the design and construction of a new wastewater treatment plant. The proposed facility is planned to include the following structures: 1) an operational building; 2) a generator structure; 3) an about 10 ft to 15 ft deep, lift station; 4) an oil/water separator structure; 5) a headworks structure; 6) two, SBR basin structures; 7) blower structures; 8) an aerobic digestor structure; 9) a sludge belt press structure; 10) a chlorine contact chamber structure; and 11) their associated parking and driveway areas. The proposed wastewater treatment plant is planned to be located along the east side of Fishermans Place Road, approximately 375 ft south of its intersection with TX-48 in Brownsville, Cameron County, Texas.

At the time of our field activities, the project site can be described as an undeveloped tract of land. In general, the topography at the subject site is relatively flat, with an estimated vertical relief of less than 3 ft across the site. Surface drainage is visually estimated to be poor. The project site is bounded to the north by an existing wastewater treatment plant; to the east and south by undeveloped tracts of lands; and to the west by Fishermans Place Road.

The proposed structures are expected to create relatively light to moderate loads to be carried by the foundation systems, which are anticipated to consist of shallow foundation systems, while the proposed lift station structure is anticipated to consist of a concrete mat foundation system. The pavement systems are anticipated to consist of flexible (asphalt) and/or rigid (concrete) pavement systems. Traffic data is not available, and section thicknesses provided herein are based on assumed traffic frequency.

For purposes of this geotechnical engineering report, the finished grade elevation (FGE) of the proposed structures was assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study, since no site grading information was provided to us at the time of the preparation of this report.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of South Texas for the use of the Port of Brownsville – Brownsville Navigation District (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses and is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from four, widely-spaced borings, drilled at this site, our understanding of the project information provided to us by the CLIENT, and the assumption that site grading will result in only minor changes in the topography existing at the time of our study. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site, particularly:

- our groundwater observations that were made during the drilling process and when the region was experiencing severe to moderate drought conditions; and
- areas that were previously filled, disturbed and/or backfilled.

The nature and extent of variations along the subject site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the subject site. No environmental opinions are presented in this report. **RKI**'s scope of work does not include the investigation, detection, or design related to the prevention of any biological pollutants. The term "biological pollutants" includes, but is not limited to, mold, fungi, spores, bacteria, and viruses, and the byproduct of any such biological organisms

If final grade elevations are significantly different from the grades assumed in this report, our office should be informed about these changes. If needed and/or desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the subject site were evaluated by four borings drilled within the site, as shown in the following table.

Proposed Structures	Number of Borings	Depth, ft. *	Boring Identification
Fishing Harbor Wastewater Treatment Plant	4	30	B-1 through B-4

*below the ground surface elevation existing at the time of our study.

The borings (designated as "B-") were drilled on April 10, 2024, at the locations shown on the Boring Location Map, Figure 1. The boring locations are approximate and were located in the field by an **RKI** representative based on the boring location map provided to our office from the CLIENT via electronic-mail attachment on March 7, 2024. The borings were drilled to the depths indicated in the previous table using a truck-mounted, rotary-drilling rig. The borings were conducted utilizing straight flight augers and were backfilled with the auger cuttings following completion of the drilling operations. During the drilling operations, Split-Spoon (with Standard Penetration Test, SPT) and Shelby-tube (ST) samples were collected.

The SPT and ST samples were obtained in accordance with accepted standard practices and the penetration test results are presented as “blows per foot” on the boring logs. Representative portions of the samples were sealed in containers to reduce moisture loss, labeled, packaged, and transported to our laboratory for subsequent testing and classification.

In the laboratory, each sample was evaluated and visually classified by a member of our Geotechnical Engineering staff in general accordance with the Unified Soil Classification System (USCS). The geotechnical engineering properties of the strata were evaluated by the following laboratory tests: natural moisture content, Atterberg limits, unconfined compressive strength tests, dry unit weight determinations, and percent passing a No. 200 sieve determinations.

The laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 5. A key to the classification of terms and symbols used on the logs is presented on Figure 6. The results of the laboratory and field testing are also tabulated on Figure 7 for ease of reference.

SPT results are noted as “blows per ft” on the boring logs and on Figure 7 where “blows per ft” refers to the number of blows by a falling 140-lb (pound) hammer required for 1 ft of penetration into the subsurface materials. Where hard materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the written request of the CLIENT.

GENERAL SITE CONDITIONS

GEOLOGY

A cursory review of the Geologic Atlas of Texas, (McAllen-Brownsville Sheet, dated 1976), published by the Bureau of Economic Geology at the University of Texas at Austin, indicates that the subject site appears to be located within Alluvium (floodplain) deposits consisting of clays, silts, sands, and gravel deposits of the Quaternary epoch (Holocene period).

According to the Soil Survey of Cameron County, Texas, published by the United States Department of Agriculture - Soil Conservation Service, in cooperation with the Texas Agricultural Experiment Station, the project site appears to be located within the Sejita-Lomalta-Barrada soil association consisting of level, poorly drained and very poorly drained, clays and silty clay loams. The corresponding soil symbol appears to be LM, Lomaltal clay, 0 to 1 percent slopes.

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads* of the 2015 International Building Code (IBC), the following information has been summarized for seismic considerations associated with this site.

- Site Class Definition (Chapter 20 of the American Society of Civil Engineers [ASCE] 7): **Class D**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 feet of soil may be may be characterized as a stiff soil profile.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United Stated of a 0.2-Second, Spectral Response Acceleration (5% of Critical Damping): **S_s = 0.035g**.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of a 1-Second, Spectral Response Acceleration (5% of Critical Damping): **S_1 = 0.013g**.
- Value of Site Coefficient: **F_a = 1.6**.
- Value of Site Coefficient: **F_v = 2.4**.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec., adjusted **S_{ms} = 0.056g**.
- 1 sec., adjusted **S_{m1} = 0.032g**.

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec.: **S_{DS} = 0.038g**.
- 1 sec.: **S_{D1} = 0.021g**.

STRATIGRAPHY

On the basis of the borings drilled for this site, the subsurface stratigraphy can be described as of moderately plastic to highly plastic, fine-grained soils with various amounts of sand. Each stratum has been designated by grouping materials that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by **RKI** in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

GROUNDWATER

Groundwater was encountered in the borings at depths ranging from about 6-1/2 ft to 7 ft below the ground surface elevations existing at the time of our study. In addition, saturated subsurface conditions were observed in Boring B-4 at a depth of about 2 ft below the ground surface elevation existing at the time of our study. The groundwater level in the borings may not have stabilized, particularly in less permeable cohesive soil, prior to backfilling. Hence, there is a potential for groundwater to exist beneath this site at shallower depths on a transient basis following periods of precipitation. Fluctuations in groundwater levels occur due to variations in rainfall, surface water run-off, recharge, or other factors not evident at the time of exploration. In addition, groundwater may potentially occur as a perched condition at the planned fill and soil interface, or within permeable soils or backfill. The construction process itself may also cause variations in the groundwater level.

FOUNDATION RECOMMENDATIONS AND CONSIDERATIONS

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared the foundation recommendations based on the assumption that the FGE of the proposed structures will be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study and the stratigraphic conditions encountered in the borings at the time of our study. If site grading plans differ from the assumed finished grades, we must be retained to review the site grading plans prior to bidding the project for construction. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

Site features that will influence the geotechnical approach to the proposed project include:

- Potential to encounter relatively shallow groundwater seepage during excavation and site grading operations;
- Soft, wet, fine-grained soils that are sensitive to disturbance,
- Presence of expansive soil/fill and potential for soil-related movements; and
- Potential for light to moderate foundation loads.

The following recommendations are based on the data obtained from our field and laboratory studies, our past experience with geotechnical conditions similar to those at this site, the project information provided to us by the CLIENT, and our engineering design analyses.

The following foundation recommendations are available to support the proposed structures:

- For the proposed operational building, generator structure, oil/water separator structure, headworks structure, SBR basin structures, blower structures, aerobic digestor structure, sludge belt press structure, and chlorine contact chamber structures: Shallow foundation systems, consisting of conventional spread and/or continuous footings with fill-supported concrete floor slabs; and

- For the proposed lift station: A shallow foundation system, consisting of a concrete mat foundation system.

Please note that the foundation capacities presented herein are based on the Allowable Stress Design methodology. In general, the allowable values given herein for shallow foundations can be increased by 33 percent for seismic, wind or other transitory loads (2018 IBC, Section 1806.1).

GROUNDWATER SEEPAGE AND DEWATERING

As discussed herein, the observed groundwater levels may not represent the conditions at the time of construction and may be encountered shallower than that observed in our boings. Typically, the Contractor is responsible for designing, installing and maintaining a dewatering system for groundwater control and taking precautions to avoid distress to nearby existing structures, as a result of dewatering. We recommend the Contractor consider retaining a dewatering expert to assist in identifying, implementing and monitoring the most suitable and cost-effective method to control groundwater.

Summary of Observed Groundwater Depths at Time of Exploration

Structures	Boring Identification	Approximate Depth to Groundwater Encountered During the Drilling Operations (ft) *
Fishing Harbor Wastewater Treatment Plant	B-1 through B-4**	6.5 to 7

* below the ground surface elevation existing at the time of our study.
**saturated subsurface conditions were observed in Boring B-4 at a depth of about 2 ft below the ground surface elevation existing at the time of our study

Excavations below the groundwater table generally require lowering the piezometric level to permit construction in a relatively dry state. This should be performed to control seepage into the excavations and to reduce artesian water pressures below the bottom of the excavations.

In cohesive soils where seepage is usually low, groundwater is generally managed by collection in trench bottom sumps for pumped disposal. Care should be taken to have a redundant pumping system that allows for overnight pumping. Water must not be allowed to pond in the trench bottoms. The softening of soils can lead to instability and sloughing of trench sidewalls. In addition, if cohesive soils contain lenses/layers of water-bearing granular or cohesionless soils, they may have to be dewatered using techniques for cohesionless soils. Deep wells and/or well point systems, as well as sumps and pumps after completion of the excavations are commonly used. The implementation of one and/or more of these methods should be anticipated for the construction of the proposed water and wastewater utility improvements.

Generally, the groundwater depth should be lowered to a depth of at least 3 ft below the planned excavation bottom to provide a firm working surface. Extended and/or extensive dewatering can result in settlement of existing structures/utilities in the vicinity; the Contractor is to take necessary precautions to minimize the effects on these structures. We recommend that a pre-construction survey of nearby structures/utilities be performed prior to construction to document existing conditions / distress prior to construction commencement.

Seasonal variations and/or unforeseen environmental conditions may cause elevated groundwater to be encountered during excavation and construction activities. We suggest the Contractor provide a line item for dewatering in the bid package in the event that dewatering is required.

The design of dewatering systems is beyond the scope of this study. The General Contractor should be prepared to control excess water encountered in the excavations due to perched water pockets, seepage, and/or rainfall. Proper construction procedures and equipment will be critical for proper performance of the dewatered excavations. Additionally, protection of personnel entering the excavations and providing a dry, stable subgrade upon which to construct foundations will be crucial.

Alternative methods such as jet grouting, grout injection, deep soil mixing, or other ground improvement techniques may be considered to reduce the groundwater seepage and improve/maintain exposed subgrades for the proposed improvements.

SENSITIVE SOIL SUBGRADES

Based on our experience in the area and results of our borings, excavations for the proposed structures will expose soft, wet, sensitive fine-grained soils that require a special grading approach to establish and maintain a stable subgrade. When these sensitive soils are encountered, the geotechnical engineer should be contacted to observe the exposed subgrade. Proof-rolling and/or moisture conditioning of exposed subgrades may be waived if, in the opinion of the geotechnical engineer, it could result in disturbance to an otherwise stable subgrade. When these sensitive soils are encountered, we recommend excavating the subgrade areas using a trackhoe equipped with a toothless bucket working above the proposed subgrade. **Construction equipment or foot traffic should be prohibited from trafficking on the potentially sensitive subgrade.**

If soft soils are encountered at the bottom of the structures' trenches, consideration should be given to protecting the excavations' bottoms by placing a thin mud mat (layer of flowable fill or lean concrete) immediately following excavations. This will reduce disturbance from foot traffic and other construction activities. Other options, such as ground improvement as discussed in the previous section, are available and can be provided pending site conditions and configurations. All necessary precautions should be implemented to protect open excavations from the accumulation of groundwater seepage, surface water runoff and rain.

EXPANSIVE, SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at this site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values on the order of 1-1/2 inches were estimated for the stratigraphic conditions encountered in the borings. The PVR values were estimated using a surcharge load of 1 pound per square inch (psi) for the concrete slab and dry moisture conditions within the regional zone of seasonal moisture variation (estimated active zone of 6 ft to 8 ft).

For reworked on-site materials or for imported materials used for site grading, if any, the PVR values could vary and will depend on the quality and characteristics of the materials used. Recommendations for building pad select fill are provided herein.

The TxDOT method of estimating expansive, soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive, soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive, soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the estimated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering....) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

SETTLEMENT POTENTIAL

The project site is underlain by layers of very soft to soft, clay soils, extending down to a depth of about 20 ft below the ground surface elevation existing at the time of our study. Based on the low shear strengths of these soils, the soils have the potential for settlements under the applied structural loads. The potential settlements can only be estimated once the site grading plans, foundation dimensions, and structural loads have been established for this project.

PVR REDUCTION RECOMMENDATIONS

As previously mentioned, for purposes of this geotechnical engineering report, the FGE of the proposed structures were assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study, since no site grading information was provided to us at the time of the preparation of this report.

To reduce expansive, soil-related movements in at-grade construction beneath the structures' footprint areas to about 1 inch, we recommend to remove the upper 2 ft (24 inches) of the existing subgrade soils, and to replace them with properly-compacted, suitable, select fill materials within the proposed structures' footprint areas up to their FGE, which was assumed to be about 12 inches (1 ft) above the ground surface elevation existing at the time of our study. It should be noted that saturated subsurface conditions were observed in Boring B-4 at a depth of about 2 ft below the ground surface elevation existing at the time of our study. As previously mentioned, excavations below the groundwater table generally require lowering the piezometric level to permit construction in a relatively dry state. This should be performed to control seepage into the excavations and to reduce artesian water pressures below the bottom of the excavations.

Keep in mind that the estimated PVR values are computed based on the recommendations for the selection and placement of suitable, select fill materials which are addressed in the Foundation Construction Considerations section of the report.

Drainage Considerations

When overexcavation and select fill replacement is selected as a method to reduce the potential for expansive, soil-related movements at any site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Filling excavations in relatively impervious clay soils with relatively pervious select fill material creates a "bathtub" beneath the structures, which can result in ponding or trapped water within the fill unless good surface and subsurface drainage is provided.

Water entering the fill surface during construction or entering the fill exposed beyond the building lines after construction may create problems with fill moisture control during compaction and increased access for moisture to the underlying expansive clays both during and after construction.

Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include, but are not limited to, the following:

- Installing berms or swales on the uphill side of the construction areas to divert surface runoff away from the excavation/fill areas during construction;
- Sloping of the top of the subgrade with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the structures' perimeters;
- Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
- Sloping of a final, well-maintained, impervious clay or pavement surface (downward away from the proposed structures) over the select fill material and any perimeter drain extending beyond the building lines, with a minimum gradient of 6 in. in 5 ft;
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the structures' perimeters;
- Locating the water-bearing utilities, roof drainage outlets, and irrigation spray heads outside of the select fill and perimeter drain boundaries; and
- Raising the elevation of the ground level floor slabs.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

FOUNDATION SUBGRADE CONSIDERATIONS

Due to the very soft to soft nature of the soils encountered in our borings at this site, relatively low bearing capacities are estimated, and thus, there is a potential for settlement-related movements if heavier column and strip loads then those described herein are applied. In addition, it is likely that difficulties will be experienced during foundation construction due to the high moisture content that we encountered within the upper 5 ft and saturated soils conditions observed in Boring B-4 at a depth of about 2 ft below the ground surface elevation existing at the time of our study. Therefore, supporting the structures on monolithic mat foundations may be considered to avoid constructing beams in potentially sloughing material.

SHALLOW FOUNDATION

The proposed operational building; generator structure, oil/water separator structure, headworks structure, SBR basin structures, blower structures, aerobic digestor structure, sludge belt press structure, and chlorine contact chamber structures may be founded on conventional spread and/or continuous footing foundations in conjunction with fill-supported concrete slabs, provided that the shallow foundation systems can be designed to withstand the anticipated soil-related movements (see the *Foundation Analyses* section of this report) without impairing either the structural or the operational performance of the proposed structures.

Allowable Soil-Bearing Capacity

Shallow foundations bearing on native, undisturbed soils and/or on new, properly-compacted, suitable, select fill materials may be proportioned using the design parameters shown in the following table:

Minimum depth below FGE:	24 in.
Minimum beam width:	12 in.
Maximum allowable soil-bearing pressure:	1,000 psf

The maximum allowable soil-bearing pressure presented previously will provide a factor of safety of about 3 with respect to the measured soil shear strengths, and provided that the subgrades are prepared in accordance with the recommendations outlined in the *Site Preparation* subsection of the *Foundation Construction Considerations* section of this report and that the site improvement procedure included in the *PVR Reduction Recommendations* subsection of this report is implemented. Provided that the site improvement procedure recommended in this report is properly implemented, then it is anticipated that total settlements will be in the order of about 1 inch. Differential settlements typically are estimated to be about one-half the total estimated settlement for most subsurface conditions.

Furthermore, the design parameters presented on the previous table are contingent upon the fill materials being selected and placed in accordance with the recommendations presented in the *Select Fill* subsection of the *Foundation Construction Considerations* section of this report. Should select fill selection and placement differ from the recommendations presented herein, **RKI** should be informed of the deviations in order to reevaluate our recommendations and design criteria.

Site Improvement with Tensar® Mechanically Stabilization Application

Alternatively, specialty ground improvement systems such as Tensar® Mechanically Stabilization Application, Geopiers®, Vibro Piers, Helical anchors, or stone columns may be considered. The specialty contractors that install these systems can design these systems based on specific details of column loads and layouts for the proposed structure. Since these methods are proprietary, you should contact companies that specialize in these techniques for specific design recommendations. These foundation systems can typically provide equivalent bearing pressures up to 4,000 pounds per square foot (psf). We have contacted Tensar® and their preliminary mechanically stabilization application provides an allowable bearing capacity of 2,000 psf, including a factor of safety of 3.0. Please refer to Figure 8 for Tensar® Mechanically Stabilization Application.

Uplift Resistance

Resistance to vertical force (uplift) is provided by the weight of the concrete footing plus the weight of the soil directly above the footing. For this site, it is recommended that the ultimate uplift resistance be based on total unit weights for soil and concrete of 100 pcf and 150 pcf, respectively. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.5 to calculate the allowable uplift resistance.

Lateral Resistance

Horizontal loads acting on spread footings will be resisted by passive earth pressure acting on one side of the footing and by base friction for footings in cohesive soils. An allowable soil adhesion value of 500 psf may be used for footings in contact with properly prepared cohesive select fill at the recommended footing depths. This value should provide a factor of safety of 2.0 with respect to the ultimate value.

Modulus of Subgrade Reaction

A modulus of subgrade reaction value of 75 pci may be utilized, provided that the proposed structures' floor slabs will be bearing on a minimum 6-inch thick, select fill layer, selected and placed in accordance with the recommendations presented in the *Select Fill* subsection of the *Foundation Construction Considerations* section of this report.

Wire Reinforcement Institute (WRI) Criteria

Beam and slab-on-fill foundations are sometimes designed using criteria developed by the WRI. On the basis of the subsurface stratigraphy encountered, a general effective plasticity index for the proposed structures' foundations of 34 percent and a climatic rating (C_w) of 15 should be utilized for the design of the proposed structures' foundations.

PROPOSED LIFT STATION STRUCTURE

We understand that the proposed lift station will be founded at depths ranging from about 10 to 15 ft below the existing ground surface elevation existing at the time of our study. As such, on the basis of the subsurface conditions encountered at the time of our field drilling activities, groundwater seepage and subgrade condition (as previously discussed) our design recommendations for the proposed lift station are provided in the following.

Shallow Foundation for the Proposed Lift Station

The weight of the lift station is expected to be less than the weight of soil to be replaced, and hence only nominal settlement is anticipated. The structure should be designed to resist buoyant forces by using either soil anchors or a thickened base slab with a diameter larger than the structure. The weight of the backfill material above the base slab can be utilized for resisting uplift. Buoyant densities should be used when calculating uplift capacities. The recommended maximum allowable soil-bearing pressure for the lift station is as shown on the following table:

Proposed Structure	Associated Boring	Approximate Bearing Depths (ft)*	Approximate Depth to Groundwater Encountered During the Drilling Operations (ft) *	Maximum Allowable Soil-Bearing Pressure, Pounds per Square Foot (psf)
Lift Station	B-1	10 to 15	7	1,000

* below the ground surface elevation existing at the time of our study.

Again, please note that dewatering methods will be required for the control of groundwater seepage during construction of the proposed structure. As such, the implementation of one and/or more of these dewatering methods should be anticipated for construction activities.

The maximum allowable soil-bearing pressure presented previously will provide a factor of safety of 3 with respect to the measured soil shear strength, provided that the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* subsection of the *Foundation Construction Considerations* of this report.

Furthermore, the design parameters presented on the previous table are contingent upon the fill materials being selected and placed in accordance with the recommendations presented in the *Select Fill* subsection of the *Foundation Construction Considerations* section of this report. Should select fill selection and placement differ from the recommendations presented herein, **RKI** should be informed of the deviations in order to reevaluate our recommendations and design criteria.

Lateral Earth Pressures

Equivalent fluid density values for computation of lateral soil pressures acting on below-grade structures were evaluated for various types of backfill materials that may be placed behind the below-grade structures. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented in the following table in preferential order for use as backfill materials.

Soil Type	Estimated Total Unit Weight (pcf)	Lateral Earth Pressure Coefficients			Equivalent Fluid Unit Weight (pcf)			Internal Friction Angle, Φ (°)
		At-Rest (K_0)	Active (K_a)	Passive (K_p)	At-Rest	Active	Passive	
Washed Gravel	135	0.45	0.29	3.40	60	40	460	33
Crushed Limestone	145	0.38	0.24	4.20	55	35	610	38
Clean Sand	120	0.50	0.33	3.00	60	40	360	30
Pit Run Clayey Gravel or Sands	135	0.48	0.32	3.12	65	45	425	31
Clays	120	0.74	0.59	1.70	90	70	205	15
Select Fill	115	0.66	0.49	2.04	75	55	235	20

The values tabulated above under "Active Conditions" pertain to flexible retaining walls free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding walls the values under "At-Rest Conditions" should be used. The at-rest condition is present when the wall is not allowed to move. Once the wall moves outward a short distance, it relieves part of the horizontal stress. The horizontal movement required to reach the active condition may be estimated by using 0.01*H (where H is the retained height). For example, for a 10 ft. retained height, horizontal movements up to 1.2 inches may be required to develop the active condition. Once the soil attains the active condition, the horizontal stress in the soil (and thus the pressure acting on the wall) will be reduced. However, features/structures directly behind the wall may experience settlements similar to the horizontal movements. Where these types of movements are objectionable, the retaining wall should be designed using At-Rest Conditions.

For the provided values to be valid for sand or gravel backfill, the backfill should be placed in a wedge extending upward and away from the edge of the wall at a 45-degree angle or flatter. If sand and gravel are to be placed within a steeper wedge, the values for Pit Run Clayey Gravels/Sands, or Clays provided above should be used. Further, any soft soil on the excavation slope should be removed prior to placement of backfill.

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as loading from construction equipment, vehicular loads, or future storage near the structures. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral earth pressures. Please note that the depth to groundwater observed at the time of drilling may not reflect future levels. Hence, groundwater levels may occur shallower than those observed in our borings.

The on-site natural clays exhibit significant shrink/swell characteristics. The use of highly clays soils as backfill against the proposed retaining structures is not recommended. Clays generally provide higher design active earthen pressures, as indicated above, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

Backfill Compaction

Placement and compaction of backfill behind the below-grade structures will be critical, particularly at locations where deep backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* subsection of the *Construction Considerations* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the structures, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum dry density in a series of 3 to 5 passes. Thinner lifts may be required to achieve the required compaction level.

Waterproofing

Consideration should be given to applying waterproofing coatings to any below-grade walls. Waterproofing of the below-grade walls for capillary moisture is often accomplished by painting the wall exteriors with a bituminous material. For greater seepage protection, membrane waterproofing would be required.

Considerations for Shallow Foundation Excavations

Shallow foundation excavations should be observed by the Geotechnical Engineer or their representative prior to placement of reinforcing steel and concrete. This is necessary to document that the bearing soils at the bottom of the excavations are similar to those encountered in the borings and that excessive soft materials and water are not present in the excavations. If soft or yielding soils are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevation.

Disturbance from foot traffic and from the accumulation of excess water can result in losses in bearing capacity and increased settlement. If inclement weather is anticipated at the time construction, consideration should be given to protecting the bottoms of beam trenches by placing a thin mud mat (layer of flowable fill or lean concrete) at the bottom of trenches immediately following excavation. This will reduce disturbance from foot traffic and will impede the infiltration of surface water. All necessary precautions should be implemented to protect open excavations from the accumulation of surface water runoff and rain.

AREA FLATWORK

It should be noted that ground-supported flatwork such as walkways, driveways, sidewalks, etc., will be subject to the same magnitude of potential soil-related movements as discussed previously. Thus, where these types of elements abut rigid building foundations or isolated/suspended structures, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut the main structure to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements to match the adjacent building performance.

BURIED PIPE RECOMMENDATIONS

We anticipate that the water and wastewater alignments will be installed using open-cut methods. Trenchless methods have not been indicated to us at this time. The following sections provide our recommendations with respect to buried pipe design including the loads imposed on buried pipe, guidelines for thrust restraint, and our recommendations for bedding and backfill. In addition, installation considerations and guidelines are also provided with respect to trench safety, excavation dewatering and equipment.

The discussion presented in this report should be carefully coordinated with the utility manufacturer/supplier to determine if there are any conflicts with the manufacturer's design or construction standards or guidelines. Any such conflicts or concerns should be resolved between to owners, the manufacturer, and the design team prior to the beginning of construction.

General

Loads on buried pipes result from a combination of material properties of the pipe and surrounding soils, the methods and techniques used during the installation process (i.e. material used for the haunch, the amount of compactive effort in the backfill materials, etc.), live loads such as roadway traffic, and internal forces due to the transmission of fluids within the pipe. As such, care should be taken to assure design assumptions are validated by review of project specifications prior to construction and appropriate quality control/ quality assurance monitoring during construction.

Earth Loads

Soil loads are similar to in-place loads in that the weight of the soils directly above and around the pipe should be considered. The soil loads affecting the pipe are dependent on the method of installation. As discussed herein, the methods of installation for the purposes of this report are trenching. The American Concrete Pipe Association[1] recommends computing the earth load from trenching with the following equations:

Trenching

$$W_d = C_d \gamma_s B_d^2 + \frac{D_o^2 (4 - \pi)}{8} \gamma_s$$

Where:

W_d	=	backfill load, lb/ LF
C_d	=	trench load coefficient
γ_s	=	soil unit weight, pcf
B_d	=	width of trench, ft
D_o	=	outside diameter, ft

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K\mu'}$$

Where:

K	=	ratio of active lateral unit pressure to vertical unit pressure
μ'	=	tan $\emptyset'$, coefficient of friction between backfill material and the trench walls
H	=	height of fill, ft

Typical values for Kμ' are as follows:
0.192 (Max. for granular materials without cohesion),
0.165 (Max. for sand and gravel),
0.150 (Max. for saturated top soil),
0.130 (Max. for ordinary clay),
0.110 (Max. for saturated clay).

The trench width must be carefully observed. As the trench width increases, the trench load transitions to an embankment load. That is, the walls of the trench are far enough away from the pipe that the trench walls no longer help support the soil adjacent to the pipe. This is the transition width. If the

[1] Concrete Pipe Handbook, American Concrete Pipe Association, (2011), Irving, Texas

trench width is equal to or larger than the transition width, embankment conditions should be used to calculate the earth loads. If required, we can provide these recommendations upon request.

Vehicular Traffic Loads

The Project Civil Engineer should review anticipated traffic loading and frequencies to appropriately account for traffic loading and frequency for buried pipes crossing underneath roadways. We recommend using the simplified load distribution method suggested in the AASHTO Standard Specifications for Highway Bridges[2]. That is, AASHTO assumes the stress induced by traffic at the ground surface is uniformly distributed to an area with sides equal to 1-3/4 times the depth of fill above the buried pipe. As an example, the single dual wheel HS-20 case (or 16,000 lb load) can be modeled with the following:

$$LiveLoad = P \big/ (0.83 + 1.75H)(1.67 + 1.75H)$$

Where:

P	=	applied load (lbs)
H	=	depth of fill above the buried pipe (ft)

For example, assuming depths of fill above the buried pipe of 8 ft, the live load for a single dual wheel HS-20 loading condition is about 69 psf.

Trench Subgrade

The bottoms of trench excavations should expose firm competent soils and should be relatively dry and free of loose, soft, or disturbed soil. If fill soils are encountered at the base of trench excavations, the materials competency should be evaluated through probing and density testing. Soft/loose, wet, weak, or deleterious materials should be over-excavated to expose firm soils. At locations where soft or loose soils extend for some depth, overexcavation to stronger soils may prove infeasible and/or uneconomical. In the event of encountering these areas of deep soft or weak soils, we recommend that the bottom of the trench excavation be over-excavated by 1- to 2-ft, and replaced with an open-graded aggregate, encapsulated in a filter fabric such as Mirafi 140N, that will allow for drainage of water, as well as provide a stable working platform.

Bedding and Backfill

Bedding is the material used along the bottom of the trench to provide uniform support for the buried pipe. When other unyielding foundation material is encountered, a more compressible material should be imported for bedding for the pipe. Materials used as bedding or for the initial backfill around the pipe preferably should be crushed stone or gravel aggregate. A minimum of 6 in. of bedding material should be provided beneath the bottom of the pipe. The bedding should be graded to provide uniform support prior to placement of the pipe.

[2] "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)," (2000) ASCE 15-98, American Society of Civil Engineers, Reston, VA, Section 9.1.2, Page 9.

Initial backfill should extend from the bedding material to a height of no less than 12 in. above the top of the pipe. These materials should be placed in 10 in. maximum loose lifts and vibrated. It is not recommended that water jetting be allowed to consolidate the backfill for this project. Bedding and initial backfill shall be compacted to a minimum of 95% of the maximum dry density defined by ASTM D698 for material with appreciable amount of fines. For cohesionless material, the material shall be compacted to a minimum of 70% of relative density as determined by ASTM D4253 and D4254.

Due to the relatively shallow groundwater, we recommend that the bedding and initial backfill be encapsulated with filter fabric (such as Mirafi 140N) to reduce the migration of fines into the void spaces of the crushed stone or gravel aggregate. Long-term migration of fines into the open void spaces of the stone may induce subsidence of grade support features such as pavement and flatwork.

Above the initial backfill, natural on-site soils may be used as secondary backfill, provided it is free of roots, organics, or other degradable/deleterious material and provided the maximum particle size does not exceed 4 in. The secondary backfill should be placed in loose lifts not exceeding 8-inches in thickness* and compacted to at least:

- For undeveloped areas where ground settlement is not a concern, secondary backfill material shall be placed in lifts with maximum compacted thickness of 6 inches (8 inch loose lifts). Thinner lifts may be required to achieve the specified compaction. This material may be placed mechanically or by other means to provide at least 90% of the maximum dry density as defined by ASTM D698 at 0 to 4 percent above optimum moisture content.

- Backfill under Road, Concrete Slabs, and areas where ground settlement is a concern. Backfill material shall be placed in lifts with a maximum compacted thickness of 6 inches (8 inch loose lifts). Thinner lifts may be required to achieve the specified compaction. For trench excavations less then 10 ft the backfill for trenches under roads, concrete slabs, and related items shall be compacted to 98% of the maximum dry density as defined by ASTM D698 at 0 to 4 percent above optimum moisture content.

Flowable fill may be considered for use as backfill in restricted work areas that prevent proper compaction of granular backfill materials. The flowable fill should be relatively low strength (less than 100 psi) for ease of excavation for repairs.

* depending on the compaction equipment utilized to compact the secondary backfill, thinner loose lifts may be required to achieve the required level of compaction. If thicker lifts are requested by the utility contractor, we recommend that the materials testing firm be retained to monitor the backfill process on a full-time basis.

Trench Lateral Flow Barriers

Due to the relatively shallow groundwater, we recommend that lateral flow barriers be installed to reduce the potential for migration of water through the bedding and primary backfill. For the lateral flow barrier, a vertical barrier would be constructed around the utility pipe within the bedding and primary backfill. Flowable fill or lean concrete may be utilized to construct the lateral flow barrier. As a minimum, the lateral flow barriers would be constructed where the alignment enters and exists the 100-year floodplain, if any. Alternatively, consideration could be given to using flowable fill instead of granular backfill around the pipe within the area(s). This option would further reduce the ability of the groundwater from migrating along the trench granular backfill.

Trench Excavations on Existing Slopes

A drainage and/or flood study was beyond our scope of services. However, trenches excavated along slopes intersecting potential areas of high water flow may require additional backfill erosion protection.

Erosion control measures may include riprap underlain by filter fabric (such as Mirafi 600X or an approved substitute) or riprap concreted in place. Articulated concrete mats may be used as an alternative to riprap systems. Articulated concrete mats consist of concrete blocks connected by galvanized or polyester cables. Mats are supplied assembled and can be handled with a lifting beam, which picks up mats from both ends. The mats, if used, should be installed per manufacture's requirements.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the structures' foundations and to facilitate rapid drainage away from the structures' foundations. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations and floor slabs (which can in turn result in cracking in the sheetrock partition walls, and shifting of ceiling tiles, as well as improper operation of windows and doors).

Current ordinances, in compliance with the Americans with Disabilities Act (ADA), may dictate maximum slopes for walks and drives around and into new buildings. These slope requirements can result in drainage problems for buildings supported on expansive soils. We recommend that, on all sides of the structures, the maximum permissible slope be provided away from the structures.

Also to help control drainage in the vicinity of the structures, we recommend that roof/gutter downspouts and landscaping irrigation systems not be located adjacent to the structures' foundations. Where a select fill overbuild is provided outside of the floor slab/foundation footprint, the surface should be sealed with an impermeable layer (pavement or clay cap) to reduce infiltration of both irrigation and surface waters. Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired. Other drainage and subsurface drainage issues are discussed in the *Expansive Soil-Related Movements* section of this report.

SITE PREPARATION

The structures' areas and all areas to support select fill should be stripped of all vegetation and/or organic topsoil down to a minimum depth of 8 inches, and extending a minimum of 5 ft beyond the structures' footprint areas.

Exposed subgrades should be thoroughly proofrolled in order to locate and densify any weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his/her representative to document subgrade conditions and preparation. Weak or soft areas identified during proofrolling should be treated with hydrated lime or Portland cement, or removed and replaced with a suitable, compacted select fill in accordance with the recommendations presented under the *Select Fill* subsection of this section of the report. If the treatment option is selected, the weak or soft areas may be mixed with hydrated lime or Portland cement down to a minimum depth of 8 inches in order to aid in drying the soils and develop a firm working surface. Proofrolling operations and any excavation/backfill activities should be observed by **RKI** representatives to document subgrade preparation.

Upon completion of the proofrolling operations and just prior to fill placement, the exposed subgrades should be moisture-conditioned by scarifying to a minimum depth of 8 in. and recompacting to a minimum of 98 percent of the maximum dry density as determined from the American Society for Testing and Materials (ASTM) D698, Compaction Test. The moisture content of the subgrade should be maintained within the range of the optimum moisture content to three percentage points above the optimum moisture content until permanently covered.

SELECT FILL

Materials used as select fill for final site grading preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specification for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A through Type E, Grades 1, 2, 3, and 5.

Alternatively, the following soils, as classified according to the USCS, may be considered satisfactory for use as select fill materials at this site: SC, GC, CL, and combinations of these soils. In addition to the USCS classification, alternative select fill materials shall have a maximum liquid limit of 40 percent, a plasticity index between 7 and 18 percent, and a maximum particle size not exceeding 4 inches or one-half the loose lift thickness, whichever is smaller. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a minimum rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

If the above listed alternative materials are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for pre-approval a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the General Contractor. The General Contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. It should also be noted that when using alternative fill materials, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.

Soils classified as CH, MH, ML, SM, GM, OH, OL, and Pt under the USCS and not meeting the alternative select fill material requirements, are **not** considered suitable for use as select fill materials at this site. The native soils at this site are **not** considered suitable for use as select fill materials.

Select fill should be placed in loose lifts **not** exceeding 8 in. in thickness and compacted to at least 98 percent of the maximum dry density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until the final lift of fill is permanently covered.

The select fill should be properly compacted in accordance with these recommendations and tested by **RKI** personnel for compaction as specified.

GENERAL FILL

Areas requiring fill that do not have requirements for reducing the expansive, soil-related movements, such as green spaces and general areas, can utilize on-site soils. These materials should have maximum particle sizes of 4 inches and placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 98 percent of maximum density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

TEMPORARY EXCAVATION SLOPING AND SHORING

Depending on the planned invert elevation, temporary slopes or retention systems may be required to install the water and wastewater utility lines. In areas where back slopes are feasible and have heights less than 10 ft, excavation slopes should be consistent with safety regulations. Worker safety and classification of soil type is the responsibility of the contractor. Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless shoring or underpinned support systems are provided.

Based on the limited and widely spaced borings, the surficial materials encountered during excavations for the proposed project are anticipated to consist of localized existing fill and water bearing fine-grained soil. Localized fills, disturbed areas, or soils that can contain perched groundwater should be classified as OSHA Type C soil.

OSHA guidelines for temporary slopes performed in Type C materials should be constructed at 1 Vertical (V): 1.5 Horizontal (H), or flatter. A professional engineer must evaluate excavations extending deeper than 20 ft.

The contractor should be aware that excavation depths and inclinations (including adjacent existing slopes) should not exceed those specified in local, state or federal safety regulations, e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the contractor, or earthwork or utility subcontractors could be subjected to substantial penalties. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods and sequencing of construction operations.

Temporary slopes left open may undergo sloughing and result in an unstable situation. The contractor should evaluate stability and failure consequences before open cut slopes are made. Minor sloughing of open face slopes may occur. If the slope is expected to remain open for an extended time, an impermeable membrane covering the slopes could be considered as a means to reduce the potential for slope degradation and instability.

It is important to note that soils encountered in the construction excavations may vary across the site and that even if the OSHA criteria are used, there is a potential for slope failure. If different subsurface conditions are encountered at the time of construction, **RKI** should be contacted to evaluate the conditions encountered.

An excavated temporary slope may not be feasible at all locations, and a temporary retention system may be required. While many different types and configurations of retention systems can be used, the more common include trench boxes. The design of the system should be performed by the contractor that performs the work. The design should account for the possibility of overexcavating unsuitable or disturbed subgrades. The contractor should also be responsible for monitoring the performance of the retention system. OSHA regulations should be followed with respect to bracing requirements. Worker safety and classification of soil type is the responsibility of the contractor. Presented in the following sections are considerations for the retention system.

EXCAVATION EQUIPMENT

The boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that General Contractors and their subcontractors interested in bidding on the work perform their own tests in the form of test pits and/or test piers to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

WET WEATHER CONDITIONS

Earthwork contractors should be made aware of the moisture sensitivity of the near surface soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft, and unworkable. Drainage trenches within the soils to be excavated, reworked and/or recompacted may be required. Additionally, subgrade stabilization techniques, such as chemical (cement, flyash or hydrated lime) treatment, may be required to provide a more weather-resistant working surface during pad construction. Therefore, we recommend that consideration be given to construction during the dryer months. Alternatively, the contractor should protect all exposed areas once topsoil has been stripped, as well as provide positive drainage during earthwork operations.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur. These types of slabs will generally be constructed as monolithic, grid type beam and slab foundations.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches (such as fractures within a rock mass or at contacts between rock and clay formations). It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials; and
- Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements for a 20-year design period are presented in this report. The CLIENT may select either pavement type depending on the performance criteria established for the proposed project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.
For either pavement type, drainage conditions will have a significant impact on long-term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SWELL/HEAVE POTENTIAL

It should be understood that pavement sections in expansive soil environments can develop longitudinal cracking along unprotected pavement edges. In the semi-arid climate of the project site, this condition typically occurs along the unprotected edges of pavements where the adjoining grounds are not developed.

The longitudinal cracking generally occurs between 2 to 4 feet inside of and parallel to the unprotected edges of the pavement. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and embankments. Differential drying and shrinkage of the highly expansive soil subgrade between the exposed pavement edge and that beneath the pavement section commonly causes the cracking. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

A horizontal barrier is commonly in the form of a paved shoulder extending 8 feet or greater beyond the edge of the pavement. Other methods of shoulder treatment, such as using geofabrics beyond the edge of the roadway, are sometimes used in an effort to help reduce longitudinal cracking. Although this alternative does not eliminate the longitudinal cracking phenomenon, the location of the cracking is transferred to the shoulder rather than within the traffic lane.

Vertical barriers installed along the unprotected edges of pavements are also effective in preventing non-uniform drying and shrinkage of the subgrade soils. These barriers are typically in the form of a vertical moisture barrier/membrane extending a minimum of 6 feet below the top of the subgrade at the pavement edge. Both types of barriers must be sealed at the edge of the pavement to prevent a crack that would facilitate the drying of the subgrade soils.

A more economical alternative, which will not limit the shrinkage of the underlying subgrade soils but may help reduce the occurrence of longitudinal cracking, is the use of a geogrid base reinforcement in the pavement section. Geogrid gives the pavement section a tensile strength component that is not otherwise inherent in a typical flexible base pavement section. Another consideration is to treat the subgrade soil with lime.

SUBGRADE CONDITIONS

A single generalized subgrade condition has been assumed for this site. The predominant subgrade soils used in developing the pavement sections for this project are the plastic, clay soils. On the basis of our past experience with similar subsurface conditions in this area, a design California Bearing Ratio (CBR) value of 3 was assigned to evaluate the pavement components. This design CBR value assumes that the subgrade soils will be prepared in accordance with the recommendations stated in the *Subgrade Preparation* subsection of the *Pavement Construction Guidelines* section of this report.

LIME TREATMENT OF SUBGRADE

In general, the subgrade soils at this site are plastic in nature and can be difficult to work with, particularly during periods of inclement weather. The performance of the subgrade soils may be improved by treating the upper 8 inches with hydrated lime. A sufficient quantity of hydrated lime should be mixed with the subgrade soils to decrease the plasticity index of the soil-lime mixture to 18 percent or less and to increase the pH of the soil-lime mixture to at least 12.4. For estimating purposes, we recommend that a minimum of 3 percent lime by weight be considered for lime treatment. For construction purposes, we recommend that the percent of hydrated lime treatment be determined by appropriate laboratory testing at the time of construction.

Based on a recently reported adverse reaction to lime addition in certain sulfate-containing soils, it is strongly recommended to perform additional laboratory testing to determine the concentration of soluble sulfates in the subgrade soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

DESIGN INFORMATION

The following recommendations for the pavement sections are based on our past experience with similar subgrade soils; an assumed traffic loading; an assumed CBR test value for the subgrade soils; and design procedures by the American Association of State Highway and Transportation Officials (AASHTO). The pavement design and analyses performed are based directly on the 1993 and 1997 editions of the "Guide for the Design of Pavement Structure" by AASHTO.

For a 20-year design period, Equivalent Single Axle Loads (ESAL's) were estimated for an assumed traffic loading of 1 tractor-trailer truck per day and two garbage trucks per week for the proposed parking lot and driveway areas. This corresponds to about 27,000 ESAL's. It is recommended that the project Civil Engineer review the above-mentioned levels of traffic and design period to ensure that they are appropriate for the intended use of the proposed wastewater treatment plant.

FLEXIBLE PAVEMENTS

The following flexible pavement section is available for this site, and other sections may be considered upon request:

LTS (in)	FBM (in.)	HMAC (in.)
8	8	2

Where: LTS = Lime-Treated Subgrade
FBM = Flexible Base Material
HMAC = Hot-Mix Asphaltic Concrete Surface Course

Garbage Dumpsters

Where flexible pavements are constructed at any site, it is recommended that reinforced concrete pads be provided in front of and beneath trash receptacles. The dumpster trucks should be parked on the concrete pads when the receptacles are lifted. It is suggested that such pads also be provided in drives where the dumpster trucks make turns with small radii to access the receptacles. The concrete pads at this site should be a minimum of 5-1/2 inches thick and reinforced with conventional steel reinforcing bars, and underlain by 8 inches of prepared subgrade.

RIGID PAVEMENTS

The following rigid pavement section is available for this site:

Pavement Area	Lime-Treated Subgrade (in.)	Reinforced Concrete (in.)
Automobile Drive and Parking Lot Areas	8	5-1/2

It is recommended that the concrete pavements be reinforced with reinforcing steel bars. As a minimum, the reinforcing bars should be No. 3 reinforcing bars spaced at about 15 in. on center in both directions (depending upon slab dimensions). The concrete reinforcing should be placed approximately 1/3 the slab thickness below the surface, but not less than 2 in. The reinforcing steel should not extend across construction or expansion joints.

Joints in concrete pavements aid in the construction and control the location and magnitude of cracks. Where practical, lay out the construction, expansion, control, and sawed joints to form square panels, but not to exceed American Concrete Institute (ACI) 302.69 Code recommendations. The ratio of slab length-to-width should not exceed 1.25. Recommended joint spacings are 15 ft longitudinal and 15 ft transverse.

All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab. Sawing of control joints should begin as soon as the concrete will not ravel, generally the day after placement. Control joints may be hand formed or formed by using a premolded fill. We recommend that all longitudinal and transverse construction joints be dowelled to promote load transfer.

If possible, the pavement should develop a minimum slope of 0.015 ft/ft to provide surface drainage. Reinforced concrete pavement should cure a minimum of 7 days before allowing any traffic.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Areas to support pavements should be stripped of all vegetation and/or organic topsoil down to a minimum depth of 8 inches and extend a minimum of 2 ft beyond the pavement perimeters. Upon completion of site stripping activities, the exposed subgrade should be thoroughly proofrolled in accordance with the *Site Preparation* subsection recommendations provided in the *Foundation Construction Considerations* section of this report. Likewise, upon completion of the proofrolling activities and just prior to select fill placement, the exposed subgrade should be scarified and recompacted as recommended in such subsection.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. Curbs should completely penetrate flexible base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs.
3) Pavement surfaces should be maintained to help reduce surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ON-SITE CLAY SOILS

The pavement recommendations presented in this report were prepared assuming that on-site clay soils will be used for site grading in the proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and compacted to a minimum of 98 percent of the maximum dry density as determined from ASTM D698. The moisture content of the subgrade should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until permanently covered. We recommend that on-site sand fill materials be free of roots, vegetation, and/or other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

LIME TREATMENT OF SUBGRADE

The strength properties of the subgrade soils that will underlie the pavements may be increased by treating them with hydrated lime. For estimating purposes, we recommend that 3 percent hydrated lime by weight be used in order to improve its shear strength characteristics, reduce its plasticity below 18 percent, and increase its pH above 12.4. Lime-treated subgrade soils should be compacted to a minimum of 95 percent of the maximum dry density at a moisture content within the range of optimum moisture content to three percentage points above the optimum moisture content as determined by ASTM D1557.

SELECT FILL

If implemented, select fill materials utilized for achieving finished subgrade elevations in pavement areas should be in accordance with the *Select Fill* subsection recommendations provided in the *Foundation Construction Considerations* section of this report.

FLEXIBLE BASE COURSE

The flexible base course should consist of material conforming to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A through Type E, Grades 1, 2, 3, and 5.

The flexible base course should be placed in lifts with a maximum compacted thickness of 8 in. and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the base course materials should be maintained within the range of three percentage points below the optimum moisture content to three percentage points above the optimum moisture content until permanently covered.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 341, Dense-Graded Hot-Mix Asphalt, Type D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

PORTLAND CEMENT CONCRETE

The Portland cement concrete pavement should be air entrained to result in a 4 percent plus/minus 1 percent air, should have a maximum slump of 5 inches, and should have a minimum 28-day compressive strength of 3,500 psi. A liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, **RKI** is retained to perform construction observation and testing services during the construction of the project. This is because:

- **RKI** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **RKI** understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- **RKI** knows what subsurface conditions are anticipated at the site.
- **RKI** is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables **RKI** to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- **RKI** has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- **RKI** cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that **RKI** and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. **RKI** looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS

RABA KISTNER

Engineering • Testing • Environmental
Facilities • Infrastructure

800 E. Hackberry
McAllen, Texas 78501

(956) 682-5332 TEL
(956) 682-5487 FAX
www.rkci.com
TBPE Firm F-3257

BORING LOCATION MAP
PROP. FISHING HARBOR WASTEWATER TREAT PLANT

ALONG THE EAST SIDE OF FISHERMANS PLACE RD
BROWNSVILLE, CAMERON COUNTY, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.:	ABA24-005-00
ISSUE DATE:	04-25-2024
DRAWN BY:	DV
CHECKED BY:	AD
REVIEWED BY:	SC
FIGURE:	1

LOG OF BORING NO. B-1

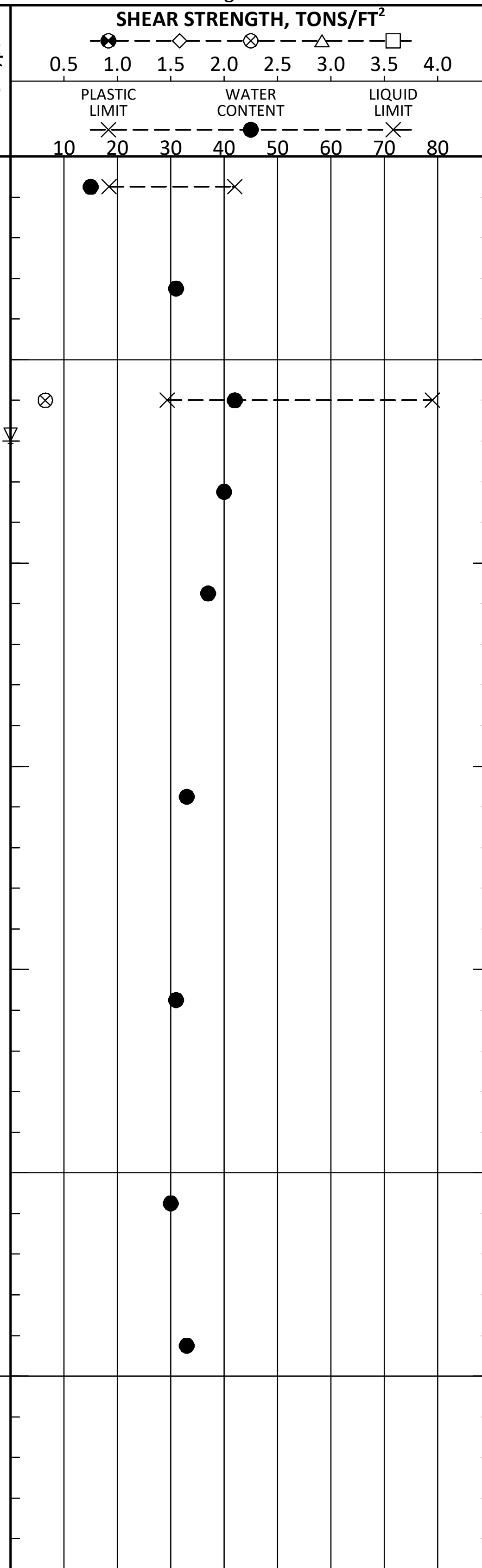
Prop. Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Rd
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger **LOCATION:** See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SANDY LEAN CLAY (CL) stiff to firm, dark brown to brown	12		24	
		5			67
5	**FAT CLAY (CH)** soft to firm, brown		80	50	
	During the drilling operations, groundwater was encountered at a depth of about 8 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.	3			94
10		4			
15		6			
20		5			
25	**LEAN CLAY with SAND (CL)** stiff to very stiff, brown to light brown	13			
		18			
30	Boring terminated at a depth of about 30 ft.				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	30.0 ft	**DEPTH TO WATER:**	7 ft	**PROJ. No.:**	ABA24-005-00
DATE DRILLED:	4/10/2024	**DATE MEASURED:**	4/10/2024	**FIGURE:**	2

LOG OF BORING NO. B-2

Prop. Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Rd
Brownsville, Cameron County, Texas

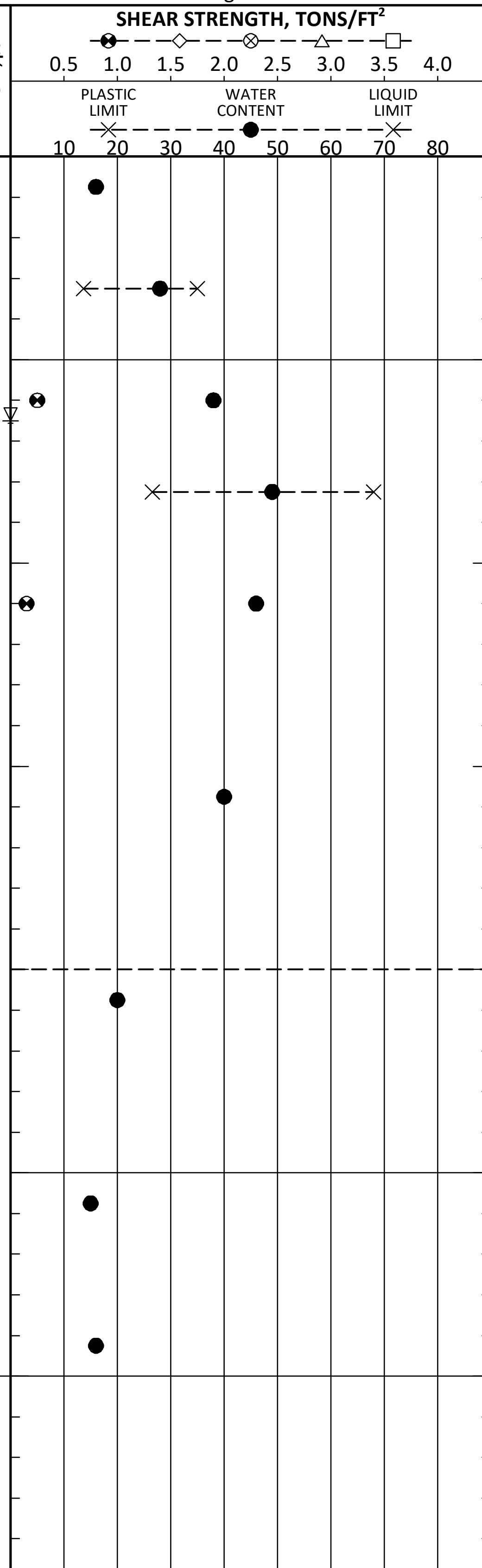
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0 – 5	**SANDY LEAN CLAY (CL)** stiff to soft, dark brown to brown	11			66
		4		21	
5 – 20	**FAT CLAY (CH)** very soft to soft, brown				98
	During the drilling operations, groundwater was encountered at a depth of about 6.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.	2		41	
		4			
20 – 25	**FAT CLAY (CH)** firm, brown	7			
25 – 30	**LEAN CLAY with SAND (CL)** very stiff, brown to light brown	22			
		26			
30	Boring terminated at a depth of about 30 ft.				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 30.0 ft	**DEPTH TO WATER:** 6.5 ft	**PROJ. No.:** ABA24-005-00	
DATE DRILLED: 4/10/2024	**DATE MEASURED:** 4/10/2024	**FIGURE:** 3	

LOG OF BORING NO. B-3

Prop. Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Rd
Brownsville, Cameron County, Texas

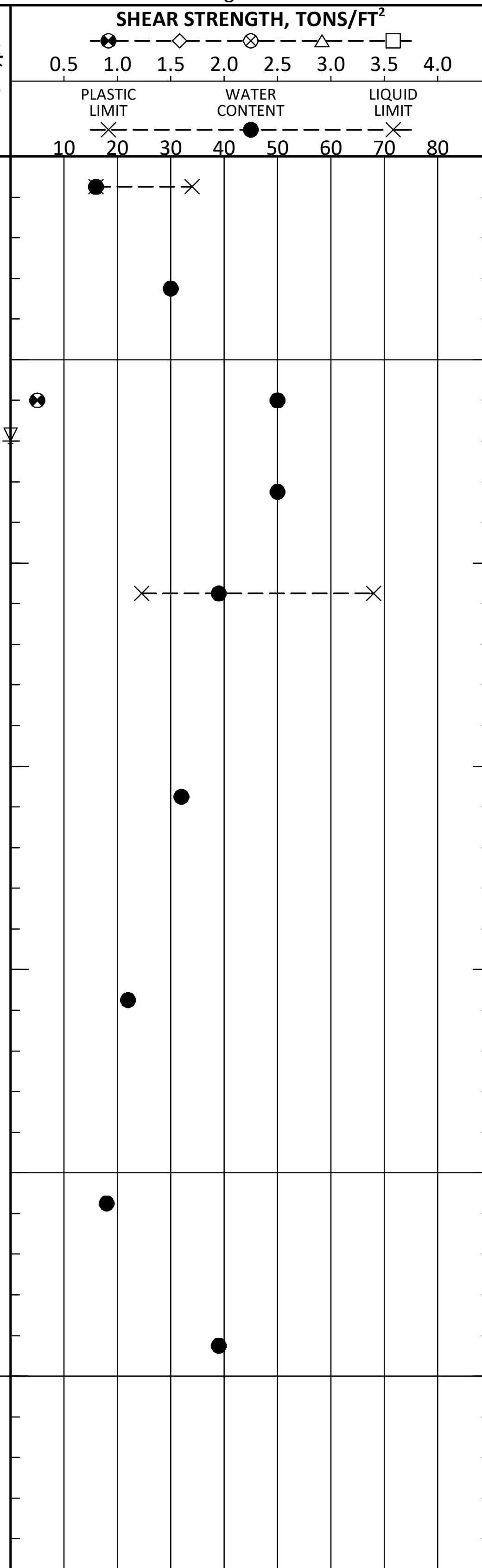
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–5	**LEAN CLAY with SAND (CL)** stiff to soft, dark brown to brown	12		18	
		4			85
5–25	**FAT CLAY (CH)** very soft to soft, brown				
	During the drilling operations, groundwater was encountered at a depth of about 7.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.	2			93
		3		43	
		4			
	- with sand seams below a depth of about 20 ft	3			
25–30	**LEAN CLAY with SAND (CL)** stiff, brown to light brown	14			
		15			
30	Boring terminated at a depth of about 30 ft.				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 30.0 ft	**DEPTH TO WATER:** 7 ft	**PROJ. No.:** ABA24-005-00	
DATE DRILLED: 4/10/2024	**DATE MEASURED:** 4/10/2024	**FIGURE:** 4	

LOG OF BORING NO. B-4

Prop. Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Rd
Brownsville, Cameron County, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger **LOCATION:** See Figure 1

SURFACE ELEVATION: Existing Grade, ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**SANDY LEAN CLAY (CL)** firm to hard, dark brown to brown	8			63
	- saturated at a depth of about 2 ft	REF/1"		25	
5	**FAT CLAY (CH)** soft, brown	4			100
	During the drilling operations, groundwater was encountered at a depth of about 9 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 7 ft.		78	32	
10		4			
15		3			
20	- with sand seams below a depth of about 20 ft	4			
25	**LEAN CLAY with SAND (CL)** very stiff, brown to light brown	16			
		19			
30	Boring terminated at a depth of about 30 ft.				

SHEAR STRENGTH, TONS/FT^2: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 30.0 ft	**DEPTH TO WATER:** 7 ft	**PROJ. No.:** ABA24-005-00	
DATE DRILLED: 4/10/2024	**DATE MEASURED:** 4/10/2024	**FIGURE:** 5	

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

CALCAREOUS
PEAT
CALICHE
SAND
CLAY
SANDY
CLAYEY
SILT
GRAVEL
SILTY
GRAVELLY
FILL

ROCK TERMS

CHALK
LIMESTONE
CLAYSTONE
MARL
CLAY-SHALE
METAMORPHIC
CONGLOMERATE
SANDSTONE
DOLOMITE
SHALE
IGNEOUS
SILTSTONE

OTHER

ASPHALT
BASE
CONCRETE/CEMENT
BRICKS / PAVERS
WASTE
NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

BLANK PIPE
BENTONITE
BENTONITE & CUTTINGS
CUTTINGS
SAND
SCREEN
CEMENT GROUT
CONCRETE/CEMENT
GRAVEL
VOLCLAY

SAMPLE TYPES

AIR ROTARY
MUD ROTARY
SHELBY TUBE
GRAB SAMPLE
NO RECOVERY
SPLIT BARREL
CORE
NX CORE
SPLIT SPOON
GEOPROBE SAMPLER
PITCHER
TEXAS CONE PENETROMETER
ROTOSONIC -DAMAGED
ROTOSONIC -INTACT
DISTURBED

STRENGTH TEST TYPES

POCKET PENETROMETER
TORVANE
UNCONFINED COMPRESSION
TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ABA24-005-00

RABAKISTNER

REVISED 04/2012

FIGURE 6a

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
EI = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Prop. Fishing Harbor Wastewater Treatment Plant
Along the East Side of Fishermans Place Rd
Brownsville, Cameron County, Texas

FILE NAME: ABA24-005-00.GPJ 4/23/2024

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	0.0 to 1.5	12	15	42	18	24	CL				
	2.5 to 4.0	5	31						67		
	5.0 to 7.0		42	79	29	50	CH	80		0.33	UC
	7.5 to 9.0	3	40						94		
	10.0 to 11.5	4	37								
	15.0 to 16.5	6	33								
	20.0 to 21.5	5	31								
	25.0 to 26.5	13	30								
	28.5 to 30.0	18	33								
B-2	0.0 to 1.5	11	16						66		
	2.5 to 4.0	4	28	35	14	21	CL				
	5.0 to 7.0		38						98	0.25	PP
	7.5 to 9.0	2	49	68	27	41	CH				
	10.0 to 12.0		46							0.15	PP
	15.0 to 16.5	4	40								
	20.0 to 21.5	7	20								
	25.0 to 26.5	22	15								
	28.5 to 30.0	26	16								
B-3	0.0 to 1.5	12	16	34	16	18	CL				
	2.5 to 4.0	4	30						85		
	5.0 to 7.0		50							0.25	PP
	7.5 to 9.0	2	50						93		
	10.0 to 11.5	3	39	68	25	43	CH				
	15.0 to 16.5	4	32								
	20.0 to 21.5	3	22								
	25.0 to 26.5	14	18								
	28.5 to 30.0	15	39								
B-4	0.0 to 1.5	8	17						63		
	2.5 to 2.6	REF/1"	39	44	19	25	CL				
	5.0 to 6.5	4	35						100		
	7.0 to 9.0		50	56	24	32	CH	78		0.28	UC
	10.0 to 11.5	4	33								
	15.0 to 16.5	3	44								
	20.0 to 21.5	4	24								
	25.0 to 26.5	16	18								
	28.5 to 30.0	19	18								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial CNBD = Cound Not Be Determined NP = Non-Plastic

PROJECT NO. ABA24-005-00

RABAKISTNER

Tensar Stabilization Application – Fishing Harbor WWTP – Brownsville, Texas

Project Overview

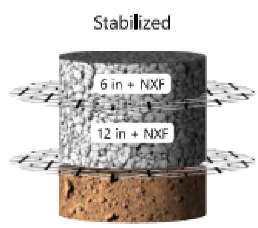
The project consists of constructing a mechanically stabilized bearing pad to support foundations for the Fishing Harbor Wastewater Treatment Plant at the Port of Brownsville in Brownsville, Texas. The purpose of this document is to present a preliminary assessment for a mechanically stabilized pad that could be considered to support foundations on the project.

Site Conditions

Tensar was provided geotechnical data for the project prepared by Raba-Kistner Consultants, Inc. (RKCI Project No. ABA24-005-00). Boring logs indicate subsurface soils generally consist of clays (CL/CH). An undrained shear strength of 500-psf is assumed for the subsurface soils. Groundwater was encountered in the borings at approximate depths varying from 6 to 9-feet below existing grade.

Mechanically Stabilized Layer

The purpose of the mechanically stabilized layer (MSL) is to support foundations, which could consist of spread footings and / or strip footings. The MSL presented in Figure 1 is calculated to provide an allowable bearing capacity of 2,000-psf for square spread footings up to 5-ft by 5-ft or strip footings. The allowable bearing capacity includes a factor of safety of 3.0 against bearing capacity failure. When foundation sizes and loadings are finalized, Tensar can provide further iterations.



Calculations

The calculations were performed with Tensar+ software, which uses the T-Value (Lees 2017) method to calculate bearing capacity of a mechanically stabilized layer (MSL) consisting of crushed angular aggregate and Tensar InterAx geogrid. The T-value method was developed with Finite Element Analysis and is quantified as a ratio of the net bearing capacity on the granular layer to the bearing capacity on the underlying natural clay soil. The load transfer efficiency of the granular layer with the geogrid is expressed as a dimensionless T-value in the equation:

$$\frac{q_u}{q_s} = (1 + T\frac{H}{B})^2 \leq \frac{q_g}{q_s}$$

Tensar+ software estimates settlements using the stress distribution method for variable soil layers. Research has shown an increase in the angle of distribution, within and below the MSL, with the inclusion of Tensar multiaxial geogrids.

Installation

Tensar should be contacted prior to installation to provide guidance regarding: 1) field verification of subgrade conditions, 2) overlap of InterAx geogrid, 3) guidance on required lateral distances for the MSL beyond the footprint of the foundations being supported, and 4) other general guidance for geogrid installation.

Prepared April 24, 2024 by: Eden W. Thomas, P.E. EThomas@tensarcorp.com 601-940-1600

Shallow Foundation

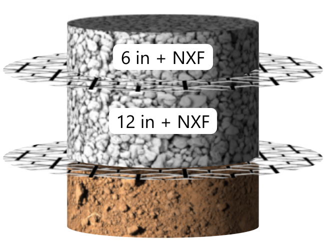
Application Suggestion Summary

Design	Preliminary Iteration 1 - Foundation Pad - 04-24-24	**Reference**	
Project	Fishing Harbor WWTP	**Location**	Brownsville, TX, USA
Customer		**Designer**	Eden Thomas
Company	Tensar - CMC	**Date**	April 24, 2024

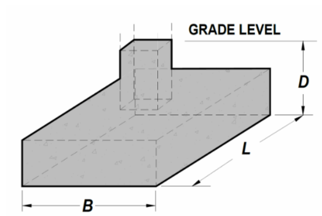
esign Meth dol gy
A design approach using the T-va ue meth d for bearing capacit calculations and various set ement approaches for the design of cost effective shallow foundation support with mechanically stabilized aggregate layers.

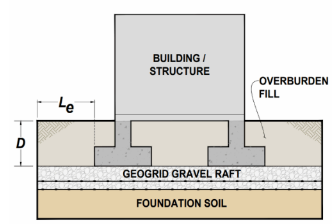
Results

Stabilized



Unstabilized





	Stabilized	Unstabilized
Required thickness	18 in	29 in
Settlement	0.2 in	0.7 in
InterAx Foundation geogrid type	NXF	
Bearing capacity factor of safety	3.73	3.02
Ultimate bearing capacity, q_u	7,460 psf	6,032 psf
Lateral extents beyond footing, L_e	≥ 6 ft	

Notes

a. Surface and/or subsurface groundwater are unique to each site and can vary seasonally. The presence of groundwater below the stabilized aggregate layer can affect foundation bearing resistance or global stability. The surface water may cause erosion and lead to poor performance or even failure of the stabilized aggregate layer. Impact of surface and subsurface water should be assessed by the client or client's engineer.

b. No alternate geogrids will be accepted without demonstrating the load spread efficiency of the mechanically stabilized layer through large scale triaxial testing, numerical modeling and full-scale validation. No products, including all TriAx and Biaxial geogrids should be considered equivalent to the specified geogrid based on index properties or in-air testing. Substitution of the specified product will invalidate the design and may result in failure.

Shallow Foundation

Application Suggestion Summary

Tensar
A Division of CMC

Design	Preliminary Iteration 1 - Foundation Pad - 04-24-24	**Reference**	
Project	Fishing Harbor WWTP	**oca ion**	Brownsville, TX, USA
Customer		**Desi ner**	Eden Thomas
Company	Tensar - CMC	**Date**	April 24, 2024

arameters

Foo ng

Length	5 ft
Width	5 ft
Depth	1 ft
Max. bearing press re	2,000 psf
Bearing capacity FS	3
Overburden unit weigh	115 pcf

Gro ndwa er

Depth	0 ft

Foundation So l

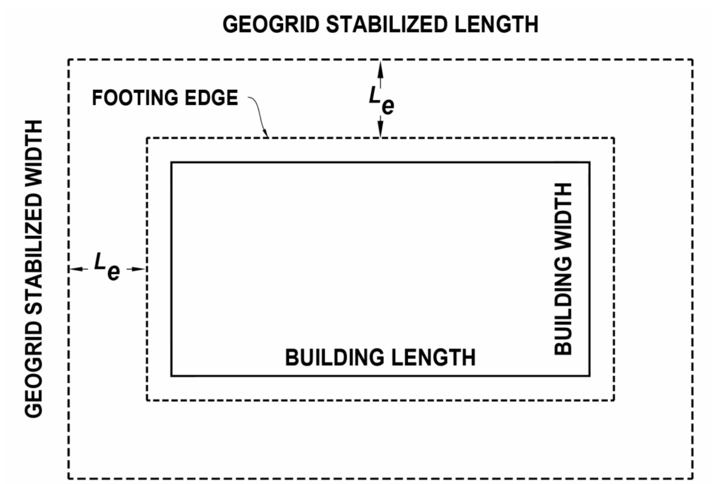
Cohesion	Cohesive
Undrained shear strength	500 psf

Aggregate

Bulk unit weight	130 pcf
Friction angle	40 deg

Soil strata

Material	Thickness	Bulk unit weight	SPT	OCR	Cc/(1+e0)	Cr/Cc
Clay	100 ft	120 pcf		2	0.1	0



Specification
To protect the performance and value achieved, a specification is generated/attached to accompany the completed design. The content of the specification provides the most effective protection to the achieved performance in a format that can be used within any project documentation.

Supporting Documents
To provide further support and advice for your completed Tensar design, documents relating to this application can be found in the "Resources" section of Tensar+. These include installation advice, background to the available value and project case studies.

This report was prepared using Tensar+ (2.11.8) ensar International Corporati n reserves the right t change its product specifications at any time. It is the responsibili y of the pers n specifying the use of th s product and of the purchaser to ensure that product specifications relied upon for design or procurement purposes are current and that the product is suitable for its intended use in each instance.

Project Performance Specification

InterAx™ Foundation NXF geogrid for shallow foundation support

General

1. The conforming shallow foundation support shall consist of a Mechanically Stabilized Layer (MSL) of defined thickness 18 in to carry imposed loading from a rectangular pressure distribution with maximum imposed pressure of 2000 psf over Cohesive subgrade soils with an undrained shear strength of 500 psf. The resulting MSL will achieve or exceed a target factor of safety for bearing capacity of 3 and the settlement under the footing will be limited to 0.2 in.
2. The Mechanically Stabilized Layer shall consist of two components:
   A. The stabilization geogrid component of the MSL shall be Tensar InterAx™ Foundation geogrid NXF which is manufactured from a coextruded, composite polymer sheet, which is then punched and oriented. The resulting structure consists of continuous and non-continuous ribs forming three aperture geometries (hexagon, trapezoid, and triangle) and an unimpeded suspended hexagon.
   B. A natural aggregate comprising well-graded, hard, angular particles. The well graded aggregate shall have a maximum particle size of 3 in, a maximum fines content of 15%, and a peak angle of friction of at least 40 degrees.
3. The mechanically stabilized layer thickness has been determined to meet or exceed the stated project specific performance targets and is based on the derivation of a project specific "load transfer efficiency" or "T-Value" of the mechanically stabilized layer incorporating stabilization geogrid(s).
4. The load spread efficiency of the MSL has been determined using the following
   A. A suitable constitutive model to simulate mechanically stabilized aggregate material behaviour. Design parameters are determined using a large 0.5m diameter triaxial testing on the aggregate and geogrid combinations to determine stabilized strength of the resulting composite layer
   B. Simulation of the application in FEA validated by the full-scale testing
   C. An extensive parametric study to derive relationships between MSL-specific T-values and subgrade strength for the design method adopted
   D. Full scale testing of mechanically stabilized aggregate layers with the same geogrid products to bearing capacity failure to demonstrate and quantify the stabilization geogrid benefit
5. Any alternative proposals or products shall be accompanied by a technical submission demonstrating the equivalence of the alternative composite construction along with the supporting test data, at least three weeks prior to the required product approval date.
6. The properties for the geogrid component of the mechanically stabilized layer are included below for the purposes of product identification only and should not be associated with expected performance of the geogrid component of the mechanically stabilized layer. It should be noted that index properties or performance characteristics for any alternative geogrid in isolation, such as tensile properties measured in air, geometric dimensions, aperture stability modulus, or quality control data will not be accepted as an adequate demonstration of equivalent platform performance.

InterAx™ Foundation NXF Properties	Test/Measurement Method	General
Aperture shapes		Hexagonal, Trapezoidal, & Triangular
Structure		Coextruded & Integrally Formed
Rib shape		Rectangular
Continuous parallel rib pitch	Nominal	80 mm (3.2 in)
Rib aspect ratio[2]		> 1.2
Color identification		White / Black / White

FDN_PSPS

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02
2. Ratio of the mid-rib depth to the mid-rib width

Dimensions and Delivery

The geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 3.8 meters (12.5 feet) in width by 60 meters (197 feet) in length.

Tensar International Corporation
2500 Northwinds Pkwy., Ste. 500
Atlanta, Georgia 30009
Phone: 800-TENSAR-1
www.tensarcorp.com

NOT FOR CONSTRUCTION. NOT A DESIGN DOCUMENT. NOT A CONTRACT. This Application Suggestion is provided free-of-charge without contract for purposes of introducing the initial concept and potential feasibility of using Tensar® Geogrids in the Application described herein. This Application Suggestion is necessarily based on limited and incomplete information and assumptions so that an early assessment can be made by others of the possible suitability of the Application illustrated. This document does not constitute a soils report, a geotechnical report, or engineering, design, construction or other professional advice or instruction concerning the Application or the project, and is not to be used or relied upon as such. Actual site and ground conditions may materially vary from those used for illustration in this document, including with respect to (i) the project site and its surroundings, (ii) ground conditions including subgrade, foundation and surface soil conditions, (iii) topographical, environmental and seasonal factors, (iv) water conditions above and below ground, (v) factors affecting the backfill requirements, and (vi) factors affecting loading, performance and other project requirements and expectations. These and other considerations should be evaluated and confirmed by a qualified and experienced geotechnical engineer engaged by the site owner or other designated party, and Tensar has not been engaged in such capacity. The construction methods, materials and solutions at the project site, and their suitability, manner of use and performance, are the responsibility of the site owner and/or those contracted to provide same. Tensar can provide its products and systems, and related formal engineering calculations and a design, if engaged to do so pursuant to the terms and conditions of a written and duly signed contract covering such items. However, this Application Suggestion does not constitute a contract, or create any warranty, responsibility or duty on Tensar's part, with respect to or in favor of the recipient or any other party.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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