Subsurface Exploration and Foundation Analysis
Proposed New Lift Station #3
Mayfair Subdivision
New Braunfels, Texas

InTEC Project No. S251705-R1
September 02, 2025

Southstar Communities
2055 Central Plaza, Suite 110, Box 195
New Braunfels, Texas 78130

Integrated Testing and Engineering Company of San Antonio, L.P.
12028 Radium, San Antonio, Texas 78216—Phone: (210) 525 9033—Fax: (210) 525 9032

September 02, 2025

Southstar Communities
2055 Central Plaza, Suite 110, Box 195
New Braunfels, Texas 78130

Attention: **Mr. Jim Vater**
Email: jim@southstartx.com

Re: Subsurface Exploration and Foundation Analysis
Proposed New Lift Station #3
Mayfair Subdivision
New Braunfels, Texas

InTEC Project No. S251705-R1

Ladies & Gentlemen:

Integrated Testing and Engineering Company of San Antonio (InTEC) has completed a **soil and foundation engineering report** at the above referenced project site. The results of the exploration are presented in this report.

We appreciate and wish to thank you for the opportunity to be of service to you on this project. If we can be of additional assistance during the materials testing-quality control phase of construction, please call us.

Sincerely,
InTEC of San Antonio

STATE OF TEXAS
MURALI SUBRAMANIAM
102417
LICENSED
PROFESSIONAL ENGINEER
InTEC of SAN ANTONIO
F-7623

09/02/2025

Murali Subramaniam, Ph. D., P.E.
Vice President

EXECUTIVE SUMMARY

The soil conditions at the location of the **proposed new Lift Station #3 at Mayfair Subdivision in New Braunfels, Texas** were explored by **drilling four borings to depths of 7 to 29 feet.** Laboratory tests were performed on selected specimens to evaluate the engineering characteristics of various soil strata encountered in the borings. Our findings and recommendations based on the field investigations and the laboratory tests are summarized below:

- The subsurface soils at the boring locations consist of dark brown clays, tan clays, tan and gray clays, and tan silty clays with caliche and marl seams.
- The results of our laboratory testing and engineering evaluation indicate that the underlying shallow clays are **moderately plastic to plastic in character.** These clays possess moderate to high shear strength.
- Potential vertical movement on the order of **3 to 4 inches** is estimated.
- Recommendations for lift station design and construction by various options such as a) open cut installation, b) retained excavation, and c) caisson installation are presented in this report. Recommendations for foundations to support the antenna tower and generator pads are also presented in this report.
- Ground water was not encountered in the borings at the time of drilling.

Detailed descriptions of subsurface conditions, engineering analysis, and design recommendations are included in this report.

TABLE OF CONTENTS

INTRODUCTION

General

This report presents the results of our **subsurface exploration and foundation analysis at the location of the proposed new Lift Station #3 at Mayfair Subdivision in New Braunfels, Texas.** This project was authorized by **Mr. Jim Vater.**

Purpose and Scope of Services

The purpose of our geotechnical investigation was to evaluate the site's subsurface and ground water conditions and provide **geotechnical engineering recommendations** for the foundation design and construction phase of the lift station project. Our scope of services includes the following:

1) drilling and sampling of four borings – to depths of 7 to 29 feet;
2) evaluation of the in-place conditions of the subsurface soils through field penetration tests;
3) observation of the ground water conditions during drilling operations;
4) performing laboratory tests such as Atterberg limits and Moisture content tests;
5) review and evaluation of the field and laboratory test programs during their execution with modifications of these programs, when necessary, to adjust to subsurface conditions revealed by them;
6) compilation, generalization and analysis of the field and laboratory data in relation to the project requirements;
7) preparation of recommendations for the design and construction phase of the project;

The Scope of Services **did not include slope stability analysis, pavement study, or any environmental assessment** for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the Boring Logs regarding odors, colors or unusual or suspicious items or conditions are strictly for the information of the client.

Project Description

The proposed project involves the development of the new Lift Station #3 at Mayfair Subdivision in New Braunfels, Texas. The proposed lift station project consists of a new Wet Well to be founded at an approximate depth of 27-ft below existing grade elevation, equipment & generator pads, a 20-ft tall light pole, and the associated parking and drive areas. Structural details are not available for our review at this time.

Review of the geologic map indicates the the site is located within Kpg, Pecan Gap Chalk, formation. Karst features are formed in limestone, dolomite, or gypsum by dissolution. A geophysical study of the site may indicate the presence and potential impact of Karst features, caves, or significant cavities on the building performance and construction delays. Geophysical study is not within the scope of this investigation.

SUBSURFACE EXPLORATION

Scope

The field exploration to determine the engineering characteristics of the subsurface materials included a reconnaissance of the project site, drilling the borings, performing Standard Penetration Tests, and obtaining Split Barrel samples.

Four soil borings were drilled at the project site. The borings were **drilled to depths of 7 to 29 feet below the presently existing ground surface**. The boring location was selected by the project civil engineer and established in the field by the drilling crew using normal taping procedures.

Drilling and Sampling

The soil boring was performed with a truck mounted drilling rig equipped with a rotary head. Conventional solid stem augers were used to advance the hole and samples of the subsurface materials were obtained **using a Split Barrel sampler.** The samples were identified according to boring number and depth, encased in polyethylene plastic wrapping to protect against moisture loss, and transported to our laboratory in special containers. The samples were collected as a part of our field exploration procedure.

Field Tests and Water Level Measurements

Penetration Tests – During the sampling procedures, **Standard Penetration Tests were performed** in the boring in conjunction with the split-barrel sampling. The standard penetration value (N) is defined as the number of blows of a 140-pound hammer, falling thirty inches, required to advance the split-spoon sampler one foot into the soil. The sampler is lowered to the bottom of the drill hole and the number of blows recorded for each of the three successive increments of six inches penetration. The "N" value is obtained by adding the second and third incremental numbers. The results of the standard penetration test indicate the relative density and comparative consistency of the soils, and thereby provide a basis for estimating the relative strength and compressibility of the soil profile components.

Water Level Measurements – Ground water was not encountered in the borings at the time of drilling. In relatively pervious soils, such as sandy soils, the indicated elevations are considered reliable ground water levels. In relatively impervious soils, the accurate determination of the ground water elevation may not be possible even after several days of observation. Seasonal variations, temperature and recent rainfall conditions may influence the levels of the ground water table and volumes of water will depend on the permeability of the soils.

Field Log

A field log was prepared for the boring. The log contained information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as silt, clay, gravel or sand and observations of ground water. It also contained an interpretation of subsurface conditions between samples. **Therefore, the log included both factual and interpretive information.**

Presentation of the Data

The final logs represent our interpretation of **the contents of the field logs for the purpose delineated by our client.** The final logs are included on **Plates 2 thru 5** included in the Illustration section. A key to classification terms and symbols used on the logs is presented on **Plate 6**.

LABORATORY TESTING PROGRAM

Purpose

In addition to the field exploration, a supplemental laboratory-testing program was conducted to determine additional **pertinent engineering characteristics** of the subsurface materials necessary in evaluating the soil parameters.

Laboratory Tests

All phases of the laboratory-testing program were performed **in general accordance with the indicated applicable ASTM Specifications** as indicated in Table No. 1.

Table No. 1 – Laboratory Test Procedures

Laboratory Test	Applicable Test Standard
Liquid Limit, Plastic Limit and Plasticity Index of the Soils	ASTM D 4318
Moisture Content	ASTM D 2216

In the laboratory, each sample **was observed and classified by a geotechnical engineer.** As a part of this classification procedure, the natural water contents of selected specimens were determined. Liquid and plastic limit tests were performed on representative specimens to determine the plasticity characteristics of the different soil strata encountered.

Presentation of the Data

The laboratory tests were conducted in the laboratory to evaluate the engineering characteristics of the subsurface materials. The results of all these tests are presented on the Boring Logs. These laboratory test results were used to classify the soils encountered according to the Unified Soil Classification System (ASTM D 2487).

GENERAL SUBSURFACE CONDITIONS

Soil Stratigraphy

The soils underlying the site may be grouped into **two to three generalized strata** with similar physical and engineering properties. The lines designating the interface between soil strata on the log represent approximate boundaries. Transition between materials may be gradual. The soil stratigraphy information at the boring location is presented in the **Boring Logs, Plates 2 thru 5.**

The engineering characteristics of the underlying soils, based the results of the laboratory tests performed on selected samples, are summarized and presented in the following paragraph.

The underlying dark brown clays, tan clays, tan and gray silty clays, and tan silty clays are moderately plastic to plastic in character with tested liquid limit values varying from 41 to 58 and plasticity index values ranging from 23 to 37. The results of Standard Penetration Tests performed within these soils varied from 12 to greater than 50 blows per foot.

Soil stratigraphy may vary across the site. If deviations from the noted subsurface conditions are encountered during construction, they should be brought to the attention of InTEC. We may revise the recommendations after evaluating the significance of the changed conditions. If the construction crew encounters, at the time of grade beam excavations or during utility trench excavations, conditions such as abundant gravel, fill material, or sand seams, please contact InTEC.

Ground Water Observations

Ground water was not encountered in the borings at the time of drilling. Short term field observations generally do not provide accurate ground water levels. The contractor should verify the subsurface water conditions prior to any excavation activities. The low permeability of the soils would require several days or longer for ground water to enter and stabilize in the bore hole. Ground water levels will fluctuate with seasonal climatic variations and changes in the land use.

It is not unusual to encounter shallow groundwater during or after periods of rainfall. The surface water tends to percolate down through the surface until it encounters a relatively impervious layer.

FOUNDATIONS ON EXPANSIVE SOIL

General

There are many plastic clays **that swell considerably** when **water is added to them and then shrink with the loss of water.** Foundations constructed on these clays are **subjected to large uplifting forces caused by the swelling.**

In the characterization of a building site, two major factors that contribute to potential shrink-swell problems must be considered. Problems can arise if a) **the soil has expansive or shrinkage properties** and b) the environmental conditions that cause **moisture changes to occur in the soil.**

Evaluation of the Shrink-Swell Potential of the Soils

Subsurface sampling, laboratory testing and data analysis is used in the evaluation of the shrink-swell potential of the soils under the foundations.

The Mechanism of Swelling

The mechanism of swelling in expansive clays is complex and is influenced by a **number of factors.** Basically, expansion is a result of changes in the soil-water system that disturbs the internal stress equilibrium. Clay particles in **general have negative electrical charges** on their surfaces and positively charged ends. The negative charges are balanced by actions in the soil water and give rise to an electrical interparticle force field. In addition, adsorptive forces exist between the clay crystals and water molecules, and Van Der Waals surface forces exist between particles. Thus, there exists an internal electro-chemical force system **that must be in equilibrium with the externally applied stresses and capillary tension in the soil water.** If the soil water chemistry is changed either by changing the amount of water or the chemical composition, the interparticle force field will change. If the change in internal forces is not balanced by a corresponding change in the state of stress, **the particle spacing will change** so as to adjust the interparticle forces until equilibrium is reached. **This change in particle spacing manifests itself as a shrinkage or swelling.**

Antecedent Rainfall Ratio

This is a measure of the local climate and is defined as **the total monthly rainfall for the month of and the month prior to laying the slab divided by twice the average monthly rate measured for the period.** The intent of this ratio is to give a relative measure of ground moisture conditions at the time the slab is placed. Thus, if a slab is placed at the end of a wet period, the slab should be expected to experience some loss of support around the perimeter as the wet soil begins to dry out and shrink. The opposite effect could be anticipated if the slab is placed at the end of an extended dry period; as the wet season occurs, uplift around the perimeter may occur as the soil at the edge of the slab gains in moisture content.

Age of Slab

The length of time since the slab was cast provides an indication of the type of swelling of the soil profile that can be expected to be found beneath the slab.

Initial Moisture Condition and Moisture Variation

Volume change in an expansive soil mass is the result of increases or decreases in water content. The initial moisture content influences the swell and shrink potential relative to possible limits, or ranges, in moisture content. Moisture content alone is useless as an indicator or predictor of shrink-swell potential. **The relationship of moisture content to limiting moisture contents such as the plastic limit and liquid limit must be known.**

If the moisture content is below or near plastic limit, the soils have high potential to swell. It has been reported that **expansive soils with liquidity index*** in the range of **0.20 to 0.40 will tend to experience little additional swell.**

The availability of water to an expansive soil profile is influenced by many environmental and man-made factors. Generally, the upper few feet of the profile are subjected to the widest ranges of moisture variation, and is least restrained against movement by overburden. **This upper stratum of the profile is referred to as the active zone.** Moisture variation in the active zone of a natural soil profile is affected by climatic cycles at the surface, and fluctuating groundwater levels at the lower moisture boundary. The surficial boundary moisture conditions are changed significantly simply by placing a barrier such as a building floor slab or pavement between the soil and atmospheric environment. **Other obvious and direct**

causes of moisture variation result from altered drainage conditions or man-made sources of water, such as irrigation or leaky plumbing. The latter factors are difficult to quantify and incorporate into the analysis, but should be controlled to the extent possible for each situation. **For example, proper drainage and attention to landscaping are simple means of minimizing moisture fluctuations near structures, and should always be taken into consideration.**

Man Made Conditions That Can Be Altered

There are a number of factors that can influence whether a soil might shrink or swell and the magnitude of this movement. **For the most part, either the owner or the designer has some control over whether the factor will be avoided altogether or if not avoided, the degree to which the factor will be allowed to influence the shrink-swell process.**

Lot Drainage This provides a measure of the slope of the ground surface with respect to available free surface water that may accumulate around the slab. **Most builders are aware of the importance of sloping the final grade of the soil away from the structure so that rain water is not allowed to collect and pond against or adjacent to the foundations.** If water were allowed to accumulate next to the foundation, it would provide an available source of free water to the expansive soil underlying the foundation. Similarly, surface water drainage patterns or swales must not be altered so that runoff is allowed to collect next to the foundation.

Topography **This provides a measure of the downhill movement that is associated with light foundations built on slopes in expansive soil areas.** The designer should be aware that as the soil swells, it heaves perpendicularly to the ground surface or slope, but when it shrinks, it recedes in the direction of gravity and gradually moves downslope in a sawtooth fashion over a number of shrink-swell cycles. In addition to the shrink-swell influence, the soil will exhibit viscoelastic properties and creep downhill under the steady influence of the weight of the soil. Therefore, if the building constructed on this slope is not to move downhill with the soil, it must be designed to compensate for this lateral soil influence.

Pre-Construction Vegetation **Large amount of vegetation existing on a site before construction may have desiccated the site to some degree, especially where large trees grew before clearing.**

* LIQUIDITY INDEX = {NATURAL WATER CONTENT - PLASTIC LIMIT} / {LIQUID LIMIT - PLASTIC LIMIT}

Constructing over a desiccated soil can produce some dramatic instances of heave and associated structural distress and damage as it wets up.

Post-Construction Vegetation **The type, amount, and location of vegetation that has been allowed to grow since construction can cause localized desiccation. Planting trees or large shrubs near a building can result in loss of foundation support as the tree or shrub removes water from the soil and dries it out.** Conversely, the opposite effect can occur if flowerbeds or shrubs are planted next to the foundation and these beds are kept well-watered or flooded. This practice can result in swelling of the soil around the perimeter where the soil is kept wet**.**

Summation

It is beyond the scope of this investigation to do more than point out that the above factors have a definite influence on the amount and type of swell to which a slab-on-ground is subjected during its useful life**.**

DESIGN ENGINEERING ANALYSIS

Foundation Design Considerations

Review of the boring and test data indicates that the following factors will affect the foundation design and construction at this site:

1) Moderately plastic to plastic clays underlie the project site. Structures supported at shallow depths will be subjected to potential vertical movements on the order of **3 to 4 inches.**

2) Underlying soils have adequate strength to support the proposed lift station.

3) The select fill should be placed and compacted as recommended under *Select Fill* in the "Construction Guidelines" section of this report.

4) Ground water was not encountered in the borings at the time of drilling.

Vertical Movements

The potential vertical rise (PVR) for slab-on grade construction at the location of the structures had been estimated using Texas Department of Transportation Procedure TXDOT-124-E. This method utilizes the liquid limits, plasticity indices, and in-situ moisture contents for soils in the seasonally active zone, estimated to be about twelve feet at the project site.

The estimated PVR value provided is based on the proposed floor system applying a sustained surcharge load of approximately 1.0 lb. per square inch on the subgrade materials**. Potential vertical movement on the order of 3 to 4 inches was estimated at the existing grade elevation.** These high PVR values will be realized if the subsoils are subjected to **moisture changes from average soil moisture conditions to wet soil moisture conditions.**

The PVR values are based on the current site grades. If cut and fill operations in excess of 6 inches are performed, the PVR values could change significantly. Higher PVR values than the above-mentioned values will occur in areas where water is allowed to pond for extended periods.

If the existing grade is lower than the finish grade elevation, compacted crushed limestone select fill should be used to raise the grade. The select fill should be placed and compacted as recommended under *Select Fill* in the "Construction Guidelines" section in this report. Each lift should be tested for compaction compliance and approved by InTEC before placement and compaction of the subsequent lifts.

If the underlying clays are removed to a depth of 5-ft and replaced by compacted select fill, potential vertical movement on the order of one inch may be estimated. The select fill should be placed in 6-inch lifts and compacted as recommended under *Select Fill* in the "Construction Guidelines" section in this report.

Notes:

- The select fill should be placed and compacted as recommended under *Select Fill* in the "Construction Guidelines" section in this report. The compacted select fill should extend a minimum of 3-ft outside the perimeter grade beams. Each lift should be tested and approved by the geotechnical engineer before placement of the subsequent lifts.

- The removal and replacement depth may vary across the site to achieve a desired potential vertical movement value based on a) existing grade elevation b) finish grade elevation (at the time of construction).

- If over excavation and select fill replacement is used to lower potential vertical movements, the bottom of excavation should be drained properly. It should not act as a bathtub and hold water in the event any accidental source of water enters the excavation. The subgrade at the excavation level may be sloped down (at least 1% slope) to one side of the excavation and drained using drain gravel and drain pipe. The drain pipe should be daylighted in a detention pond or natural drainage feature.

- The select fill extending 3-ft outside the building should be covered by 2-ft thick compacted impervious clay. The impervious clay (with plasticity index value 35 or greater) should be placed in 8-inch loose lifts and compacted to a minimum of 95 percent of the maximum ASTM D 698 dry density at a water content between Optimum and Optimum Plus two percentage points. The top surface of clay seal should be sloped away from the building perimeter.

- Coping with problems of shrink/swell due to expansive clays is a “fact of life” in the Texas region of south western U.S.A. Support of the buildings on deep underreamed footings with a structurally suspended floor slab (12-inch void) will provide a foundation system with the least risk for distress due to shrink/swell of the clays. It should be noted that expansive clay does not shrink/swell without changes in moisture content, and thus good site design is very important to minimize foundation movements.

- It is our experience that support of the walls and columns on stiffened grid type beam and slab foundation or post tensioned beam and slab foundation will provide reasonable performance of the foundation if the clay subsoils are wet at the time of earthwork construction. However, some shrink/swell will probably occur causing some cracks in the floor slab and interior walls due to the foundation system because the subsoil conditions between borings are unknown, the moisture content of the clays and groundwater conditions at the time of construction are unknown, and construction practices can adversely affect the supporting properties of the subsoils.

Flatwork

Ground supported flatwork adjacent to the buildings will be subjected to the movements due to shrink / swell of the underlying soils. Differential movement between the flatwork and the building may result in a trip hazard. Reducing the potential vertical movements as described in the *Vertical Movements* section will reduce the different movement described above.

FOUNDATION RECOMMENDATIONS

General

The following recommendations are based on the data obtained from our field and laboratory tests, our past experience with geotechnical conditions similar to those at this site, and our engineering design analysis.

Surface drainage is very important around the structures and in the pavement areas. The surface water should be drained as fast as possible around the structures and roadways. If enough slopes are not available for a good surface drainage, gratings and pipes may be used to carry the water and drain the area fast. In some areas, a) where surface water gets into the subsurface and b) water travels laterally within the underlying gravel layers should be collected by French Drains and disposed of in the drain areas.

At the time of construction, utility and grade beam excavations may reveal that the soils encountered are different from those encountered in the borings. If this happens, we should be informed. We may revise the foundation recommendations accordingly. If the construction crew encounters, at the time of grade beam excavations or during utility trench excavations, conditions such as abundant gravel, fill material, or sand seams, please contact InTEC.

Foundation Selection

The type and depth of foundation suitable for a given structure primarily depends on several factors: the subsurface conditions, topography, site drainage, the function of the structure, the loads it may carry, and the cost of the foundation. Additional considerations may include acceptable performance criteria set by the owner, architect, or structural designer with respect to vertical and differential movements, which the structure can withstand without damage.

Based on the above mentioned conditions, engineering design analysis considered slab on grade foundation for equipment pads, and various options such as open cut installation, retained excavation, and caisson installation for the lift station.

LIFT STATION FOUNDATIONS

It is our understanding that the bottom of the lift station is approximately 27-ft below existing grade. Based on the soils encountered in the boring, tan silty clays with caliche and marl seams are expected at this depth.

1. Ground water was not encountered in the borings at the time of drilling. Short term field observations generally do not provide accurate ground water levels. The contractor should verify the subsurface water conditions prior to any excavation activities. The low permeability of the soils would require several days or longer for ground water to enter and stabilize in the bore hole. Ground water levels will fluctuate with seasonal climatic variations and changes in land use. It is not unusual to encounter shallow groundwater during or after periods of rainfall. The surface water tends to percolate down through the surface until it encounters a relatively impervious layer.

2. Potential vertical movement on the order of **2 inches** is anticipated at the bottom of the lift station elevation (approximately 27-ft below existing grade elevation). **If the bottom of the lift station slab is underlain by 2 feet thick compacted crushed limestone select fill, potential vertical movement on the order of 1 inch is anticipated**. <u>If proper drainage is not maintained or water is allowed to pond for extended periods, then the resulting potential vertical movements will be much greater than 2 or 3 times the anticipated vertical movements.</u>

3. It is our understanding that a mat foundation slab is planned to be used. A soil modulus value of **200 pci** is recommended.

4. An Allowable Bearing Capacity value of **2,500 lbs per sq ft** is recommended for the mat foundation slab supported at a minimum depth of 27-ft below existing grade. A Factor of Safety value of 2.5 was used in the evaluation of the Allowable Bearing Capacity. The above recommendation assumes that the final bearing surface consists of undisturbed sandy clays. Any water seepage observed during excavation should be pumped. Contractor should have dewatering measures readily available should excessive water seepage be observed during construction. If subgrade is too wet for a work, the underlying subgrade soils may be cement treated to a depth of 12-inches or twelve inches of gravel may be used to obtain a working platform.

5. The bearing surfaces should not be disturbed during construction.

6. Excavation options for the lift station include: (a) open-cut, (b) retained excavation, and (c) caisson. These excavation procedures and their effect on design and construction of the lift station are discussed in the following paragraphs.

(a) Open Cut Installation for Lift Station

The open-cut option is one of the installation techniques for below-grade facilities. The approach provides the best access for construction and has the least effect on the design of the structure. However, the demands on space and earthwork are upper-bound, and dewatering of transmissive strata, must be sufficient to avoid slope instabilities and maintain a stable bearing surface. The following paragraphs address the design and construction concerns of side-slope stability, dewatering, bottom stability, lateral earth pressures, and uplift design within the contexts of the open-cut option.

Side-slope Stability For open-cut excavation, temporary side-slopes of 1 (V): 1.5 (H) is common in underlying silty clays. It is important that unstable areas, if encountered, be sandbagged, corrected with earthwork, or cut off with a retention system to prevent propagation of the unstable area into a major slide. Sufficient berm area should be provided to accommodate sloughing of the construction slopes with time.

Dewatering Ground water, if encountered at the time of lift station installation, should be handled by sump and pump method of dewatering systems.

Bottom Stability The anticipated potential for bottom instability does not exist at this site since (1) silty clays were encountered at this site, and (2) permanent ground water is not likely at this site. Water seepage was encountered at an approximate depth of 27-ft during drilling. Bottom stability may be improved using methods such as installation of drain system and / or cement treatment of the soils strength.

Lateral Earth Pressures For open-cut techniques, the walls of the lift station will be subjected to lateral earth pressures developed from placed backfill. The magnitude of the lateral earth pressures is primarily dependent on the following factors:

a. Type of backfill materials used

b. Amount of compactive effort in placing the backfill

c. Method of construction

d. Rigidity of the walls

Cohesionless soils, such as sands with little or no fines, are preferred as backfill materials. Granular backfill should have less than 25% of materials finer than No. 200 sieve. Clay soils are typically less desirable materials because they are difficult to keep drained, produce larger earth pressures than granular soils, and can develop high swell pressures from expansion. Cohesive soils with plasticity indices greater than 20 are not recommended.

Granular backfill should be placed in maximum 12-inch lifts and compacted by vibratory or pneumatic equipment to 88% to 92% of the standard Proctor maximum dry density as determined by ASTM 698. We suggest that a 5-ft thick cohesive cap be compacted on top of the granular backfill to reduce surface runoff infiltration.

Over-compaction of the backfill materials should be avoided to reduce lateral earth pressures. A higher compactive effort to achieve 93% to 98% of ASTM D 698 would be required in cases where backfill will support a surcharge.

The lateral earth pressures may be calculated by multiplying the equivalent fluid density for the backfill type by the depth below the ground surface. We recommend installation of a piezometer to evaluate the groundwater level. The equivalent fluid densities for various backfill materials and compactive efforts are outlined in the following table. The equivalent fluid densities outlined for the high compactive effort should be used for level backfill with surcharge.

Table No. 2 – Equivalent Fluid Density Values

Soil Type	Compactive Effort (%)	Equivalent Fluid Density (pcf)	
		Above Water Table	Below Water Table*
Select cohesive backfill (7 <= PI <= 20)	88 to 92 93 to 98	65 85	95 105
Non-select cohesive backfill (PI > 20)	88 to 92 93 to 98	75 100	105 115
Bank Sand (Fines content < 25%)	88 to 92 93 to 98	55 75	89 98
*** These magnitudes do not include a water component, if applicable**			

Surcharge loads adjacent to the subsurface walls should also be considered. Local surcharge loads adjacent to the walls, if present, should be incorporated into the pressure diagrams. A surcharge load, ***q***, will typically result in a lateral load equal to ***0.4q to 0.5 q***. As a factor of safety for lateral design, the surcharge loads should be multiplied by a factor of 1.5.

Uplift Since permanent water table is not encountered at this site, uplift force need not be considered at this site.

(b) Retained Excavation for Lift Station

Retained excavations generally require less ground surface area than the open-cut approach and less groundwater control support for transmissive zones penetrated by the excavation and the retention system. A retained excavation detail can begin from ground surface or from an initial open-cut. The retention system can consist of driven sheet pile, soldier pile/lagging, or drilled shafts. The bracing system for any of the three methods can be cross-lot bracing or drilled-and-grouted soil tie-backs.

Bottom Stability Stability of the excavation bottom is not greatly affected by the retention system and should be addressed in a manner similar to that for the open-cut option.

Lateral Earth Pressure Lateral earth pressure design for structure walls constructed within a retained excavation is governed by the method of unit construction. Lift station walls are commonly cast against the retention system, so backfill is not placed behind the structure walls. The retention system may or may not be removed. If the structure is cast within the retention system such that the width of the fill body between the wall and the retention system is about 8-ft or more, lateral earth pressure behavior is generally dominated by the backfill. Procedures outlined in the "Open-Cut Installation" section should be used to estimate lateral earth pressures.

If the fill body is less than about 15-ft wide or if the structure walls incorporate the retention system, lateral earth pressure design is generally based on the natural soil stratifications. This design approach assumes that any backfill placed to fill gaps between the wall and the retention system is not over compacted and is similar in character to the natural soil conditions. This section of the report addresses lateral earth pressure design as affected by the in situ soils.

Factors affecting lateral earth pressure design include the following:

a. Type of retention system, such as cantilevered or braced
b. Design condition, such as long-term or during construction
c. Dewatering control
d. Soil type
e. Width of fill, if any, placed between the unit walls and the retention system
f. Quality of compaction

Common geometries of retention systems for lift station excavations are rectangular braced walls or circular shafts. Experience indicates that the lateral earth pressures acting on circular shafts are approximately equal to the pressures on rectangular walls.

Two design cases using these geometries for lateral earth pressure design are temporary braced excavation and permanent wall.

Local surcharge loads adjacent to the lift station, if present, should be incorporated into the pressure diagrams. A surcharge load, q. will typically result in a lateral load equal to 0.4 to 0.5 q. As a safety factor for lateral design, the surcharge loads should be multiplied by a factor of 1.5.

(c) Caisson Installation for Lift Station

The caisson procedure eliminates the need for a retention system and does not have the large area and earthwork requirements of the open cut approach. Caisson units, however, must be structurally stiffer than units installed by conventional techniques, and can experience problems with alignment and termination at the proper design depth. An 18-ft diameter hole may be drilled with a drilling rig. A heavily reinforced cylindrical wall and base slab may be constructed. The cylindrical wall may be designed as a retaining wall. For retaining wall design, design parameters presented in Table No. 2 may be used.

The depth required for anchor piers or piles are estimated from axial tensile capacity curves developed for the soil conditions encountered in the boring. These curves would be developed using methods discussed in “Drilled Shafts: Construction Procedures and Design Methods”, U.S. Department of Transportation, 1988, by L.C. Reese and M. W. O’Neill and using our area experience. Design axial uplift capacity curves will be provided, if this design option for uplift control is selected.

Adhesion or frictional resistance is not commonly included as a factor in uplift resistance for caisson installations because of (1) the potential for developing voids along the soil-wall interface during caisson excavation, (2) misalignment of the structure as it moves downward, and (3) the loss of ground sloughing that may occur during excavation. We anticipate a long term frictional resistance along the sides of the caisson of about 300 psf, provided the caisson is grouted in place.

Light Pole

Straight shaft piers may be used to support the 20-ft light pole. It is our understanding that the tower is to be constructed, approximately, at the existing grade elevation.

- **Piers founded at a depth of 20-ft below existing grade may be considered to support the light pole tower**. Please contact us if deeper pier depths are required.

- Based on the soils encountered in the borings, tan silty clays with caliche and marl seams are expected at this depth.

- The piers founded at a depth of 20-ft below existing grade may be sized for an Allowable End Bearing Capacity value of **7,500 lbs per sq ft**.

- Allowable Skin Friction value of **750 lbs per sq ft** is recommended. Skin friction value should be discarded for the **top 10-ft** of the pier shaft.

- Uplifts Forces Moisture variation in the expansive soils at this site can cause vertical movements of the subsurface soils. This potential vertical movement can mobilize uplift force along the shaft of a drilled pier. The uplift force acting on the shaft may be estimated by using the Equation No. 1.

  Fu = 34d - - - - - - (1)

  Fu = uplift force in kips

  d = Diameter of the shaft in feet

  Tension steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compression load carried by that footing. We recommend that each pier be reinforced with tension steel to withstand this net force or one percent of the cross-sectional area that shaft, whichever is maximum.

- Lateral Resistance of the Piers The lateral resistance and the respective lateral deflection at the top of the pier may be evaluated by using the L Pile program. Please contact InTEC to run the L-Pile analysis with the lateral load information. The geotechnical parameters for L-Pile program are presented in Table No. 3.

Table No. 3 – L-Pile Parameters

Depth Range, Feet	Shear Strength, TSF	Soil Modulus, PCI	Ɛ 50	γ PCF
10 – 20	1.0	750	0.005	105
Friction angle, phi, for clays = 0				

GENERATOR AND EQUIPMENT FOUNDATIONS

Stiffened grid type beam and slab foundation or Mat foundation may be used to support the Generator and other equipment provided all of the following items are followed:

(a) the anticipated potential vertical movements presented in the *Vertical Movements* section are acceptable to the owner,

(b) the owner (after discussing the resulting corresponding potential movements and their effect on the performance of the structures with the project structural engineer and the project architect) determines that the anticipated movements will not adversely affect the performance of the proposed structures,

(c) the subgrade is proof rolled, and

(d) all the recommendations presented in the "Construction Guidelines" section are followed.

Stiffened Grid Type Beam and Slab Foundations

It is desirable to design the foundations system utilizing the simplifying assumption that the loads are carried by the beams. The grade beams or mat foundation should be supported at a depth of 24 inches below final grade elevation. Allowable Bearing Capacity value is presented in Table No. 4 in the following page. The design plasticity index values will change when cut and fill operations are performed underneath the building pad. Stiffened grid type beam and slab foundation parameters are also presented in Table No. 4. For a mat foundation, a subgrade modulus value of **75 pci** is recommended.

Table No. 4 – Stiffened Grid Type Beam and Slab Foundation Parameters

Soil Condition	Net Allowable Bearing Capacity (psf)	Unconfined Compressive Strength (tsf)	Design Plasticity Index Value	Soil Support Index	Climatic Rating
Compacted Subgrade **(PVR of 3 to 4 inches)**	1,600	0.7	40	0.68	17
PVR Lowered to 1 inch	2,000	1.0	25	0.72	17

Method to lower PVR is presented on Page No. 14.

Notes:

- The grade beams should be founded on or within compacted existing soils or compacted select fill.
- Allowable Bearing Capacity values as presented above are recommended for grade beams founded at a minimum depth of 24 inches below final grade elevation.
- The bearing stratum should be verified by InTEC prior to installation of steel and concrete.
- Minimum grade beam width of 12 inches is recommended.

Seismic Design Criteria

The following seismic design criteria are presented based on the International Building code 2018 (IBC 2018) Section 1613 - Earthquake Loads. These criteria include the seismic site class and the spectral acceleration values and presented in Table No. 5 in the following page.

Table No. 5 – Seismic Design Criteria

Site Class Site class definition was determined based on Standard Penetration Test values – ASCE 7-16	D
Maximum considered earthquake 0.2 sec spectral response acceleration - Figure 1613.2.1(1) IBC 2018	S_s = 0.072g
Maximum considered earthquake 1.0 sec spectral response acceleration – Figure 1613.2.1(2) IBC 2018	S_1 = 0.031g

Note: the 2018 International Building Code (IBC) requires a site soil profile determination based on 100-ft depth for seismic site classification. The seismic site class definition considers that the same soils continue below the maximum depth of our borings. Deeper borings will be required to confirm the conditions below the maximum depth of our boring.

PAVEMENT GUIDELINES

General

Pavement areas for the proposed project are expected to include parking areas and driveways. The following recommendations are presented as a guideline for pavement design and construction. These recommendations are based on a) our previous experience with subgrade soils like those encountered at this site, b) pavement sections which have proved to be successful under similar design conditions, c) final pavement grades will provide adequate drainage for the pavement areas and that water will not be allowed to enter the pavement system by either edge penetration adjacent to landscape areas or penetration from the surface due to surface ponding, or inadequate maintenance of pavement joints, or surface cracks that may develop, and d) design criteria presented in the previous paragraph.

It is beyond the scope of this investigation to do more than point out the factors that have a definite influence on the amount and type of swell to which a pavement is subjected during its useful life. The design engineer must be aware of these factors as he develops his design and make adjustments as necessary according to the results of special measurements or from his engineering experience and judgment.

Pavement designs provide an adequate thickness of structural sections over a particular subgrade (in order to reduce the wheel load to a distributed level so that the subgrade can support load). The support characteristics of the subgrade are based on strength characteristics of the subgrade soils **and not on the shrinkage and swelling characteristics of the clays.** Therefore, the pavement sections may be adequate from a structural stand point, may still experience cracking and deformation due to shrinkage and swelling characteristics of the soils. In addition, if the proposed new pavement areas are used to carry temporary construction traffic, then heavier street sections may be needed. Please contact InTEC to discuss options.

It is very important to minimize moisture changes in the subgrade to lower the shrinkage and swell movements of the subgrade clays. The pavement and adjacent areas should be well drained. Proper maintenance should be performed by sealing the cracks as soon as they develop to prevent further water penetrations and damage. In our experience,

(a) majority of the pavement distress observed over the years were caused by changes in moisture content of the underlying subgrade and / or excessive moisture in the base section,

(b) pavements with a grade of one percent or more have performed better than the pavements with allowable minimum grade,

(c) pavements with no underground utilities have performed better than pavements with underground utilities and the associated laterals,

(d) pavements that are at a higher-grade elevation than the surrounding lots have performed better, and

(e) any design effort that minimizes moisture penetration into the pavement layers have performed better.

Periodic maintenance such as crack sealing should be anticipated for pavements constructed on clay subgrades.

<u>"Alligator" type Cracks</u>

A layer of aggregate base is typically used underneath the concrete curbs around the pavement areas. This layer of aggregate base underneath the concrete curb is conducive to the infiltration of surface water into the pavement areas. Water infiltration into the base layer can result in "alligator type" cracks especially when accompanied by construction traffic. Increasing the moisture content of the pavement sections will significantly impact the support characteristics. Penetrating the concrete curbs at least six inches into the native clays soils will act as a barrier to this type of water infiltration. In addition, French Drains installed on the outside of the curbs will reduce this type of water infiltration. Alligator type cracks are also caused by weak / soft pockets within the pavement layers.

<u>Longitudinal Cracks</u>

Asphalt pavements in expansive soil conditions, such as the soils encountered at this site, can develop longitudinal cracks along the pavement edges. The longitudinal cracking typically occurs about 1 to 4 feet inside of the pavement edges and they run parallel to the pavement edge. The longitudinal cracks are generally caused by differential drying and shrinkage of the underlying expansive clays. The moisture content change of the underlying subgrade clays can be reduced by installing moisture barriers. Vertical moisture barriers along the edge of the pavement or horizontal moisture barriers such as paved sidewalks or geogrid will help reduce the development of the longitudinal or reflective cracks.

Periodic Maintenance

The pavements constructed on clay subgrades such as the one encountered at this site will be subjected to swell and shrinkage related movements. Hence, **periodic maintenance such as crack sealing and surface finishing should be anticipated and performed.**

Pavement Sections

Parking areas and drives may be designed with either a flexible or rigid pavement. Pavement sections for both rigid and flexible types are recommended as follows for drive areas and parking areas. Brown clay subgrades are anticipated. Pavement recommendations presented here are based on existing soil conditions. If the subgrade condition is different from the anticipated brown clays, please contact us.

Table No. 6 – Concrete Pavement

	Parking Areas	Drive Areas	Heavy Duty
Reinforced Concrete (inches)	5 ½	6 ½	8
Cement treated subgrade (inches)	6	6	6

- Heavy Duty concrete section is recommended in areas which receive repetitive traffic such as drive-through lanes, entry / exit ramps, and trash dump areas, even if Asphaltic concrete pavement is used in other areas.

Table No. 7 – Asphalt Pavement

	Parking Areas	Drive Areas
Hot Mix Asphaltic Concrete-HMAC (inches)	2	3
Aggregate Base (inches)	9	9
Cement treated subgrade	6 inches	

- Heavy duty concrete section is recommended in areas which receive repetitive traffic such as drive-through lanes, entry / exit ramps, and trash dump areas, even if Asphaltic concrete pavement is used in other areas.

- It is our understanding that emergency vehicles, weighing up to 85,000 lbs, will also use the pavement at this site. The recommended pavement sections for Drive Areas and Heavy Duty Areas may be used for the use of emergency vehicles.

Notes:

- The recommendations are based on anticipated subgrade material. If cut and fill are performed, please let InTEC know.

- The flexible pavement recommendations are based on input parameters presented in Table No. 8. If recommendations for different inputs are needed, please contact us.

- Recommendations presented in the tables above do not apply to traffic areas with heavy truck or repetitive truck traffic areas.

- Final pavement subgrade Plasticity Index values are anticipated to be greater than 20:

    - Cement treated subgrade: The subgrade may be treated with cement. Application rate of 27 lbs per sq yard for 6 inch depth of treatment is recommended.

- Pavement section recommendations are based on subgrades prepared as described in this report. If water is allowed to get underneath the asphalt / concrete or if moisture content of the base or subgrade changes significantly, then pavement distress will occur.

- Deeper curbs, curbs extending a minimum 6 inches into subgrade will help reduce the moisture getting underneath the pavement.

- The pavement can experience cracking and deformation due to shrinkage and swelling characteristics of the soils as described in the Vertical Movements section of this report. Periodic maintenance will be required.

Table No. 8 – Input Parameters – Flexible Pavement Section Calculation

Parameter	Flexible Pavement Parking Areas	Flexible Pavement Drive Areas
ESAL	20,000	40,000
Reliability Level	R-70	R-70
Serviceability Loss	2.0	2.0
Standard Deviation	0.45	0.45
Service Life	20 years	20 years
Subgrade CBR	2.5	2.5

Subgrade Preparation

Existing asphalt pavement should be removed and the exposed subgrade should be properly prepared prior to pavement installation. The subgrade should be proof rolled and prepared as described in the previous section. Base course material should be placed immediately upon completion of the subgrade compaction operation to prevent drying of the soils due to exposure.

Base Course

Based on the surveys of available materials in the area, a base course of crushed limestone aggregate or gravel appears to be the most practical material for asphalt pavement project. The base course should conform to Texas State Department of Highways and Public Transportation Standard Specifications, Item 247, Type A, Grade 1-2. The base course should be compacted in two lifts to at least 95 percent of maximum dry density as determined by test method TXDOT-113-E at a moisture content in between optimum minus 1 and optimum plus 2 percent.

The existing asphalt and base material may be recycled and should meet the same specifications as noted above, TxDOT Item 247, Type A, Grade 1-2.

Asphaltic Concrete

The Asphaltic concrete surface course material and installation should conform to all applicable City Guidelines.

Reinforced Concrete

Concrete material and installation should follow all applicable City Guidelines. At a minimum, 28-day compressive strength requirement of 4000 psi may be utilized.

Concrete Pavement

Concrete pavement slabs should be provided with adequate steel reinforcement. Proper finishing of concrete pavements requires the use of sawed and sealed joints which should be designed in accordance with current Portland Cement Association guidelines. Dowel bars should be used to transfer loads at transverse joints. Related civil design factors such as drainage, cross-sectional configurations, surface elevations and environmental factors which will significantly affect the service life must be included in the preparation of the construction drawings and specifications. Normal periodic maintenance will be required, especially for open jointed areas which may allow surface water infiltration into the subgrade.

Table No. 9 – Concrete Pavement Reinforcement

Reinforcement:	#3 reinforcing steel bars (grade 60) at 18 inches on center each way (at 12 inches on center each way for 8 inch thick concrete)
Contraction joint spacing:	10 feet each way for 5 ½ inch thick concrete 12 feet each way for 6 ½ inch thick concrete 15 feet each way for 8 inch thick concrete The saw cuts should be planned based on features such as inlets, manholes, valves, etc. The saw cuts are recommended to be made the same day.
Contraction joint depth:	At least one-fourth (¼) of pavement thickness
Contraction joint width:	One-fourth (¼) inch or as required by joint sealant manufacturer
Expansion joint:	Expansion joints are not recommended
Isolation joint:	Features such as concrete inlet structures, man-holes, and valve covers should be isolated using isolation joints.

Perimeter Drainage

It is important that proper perimeter drainage be provided so that infiltration of surface water from compacted areas surrounding the pavement is minimized, or if this is not possible, curbs should extend through the base and into the subgrade. A crack sealant compatible to both asphalt and concrete should be installed at the concrete-asphalt interfaces.

The surface water may infiltrate from the compacted areas surrounding the pavement. These areas should be sloped away from the pavement areas and should be grass or concrete covered or rip-rapped. If a berm is provided before the sloped area, the berm should be sloped at least 1V to 8H. The sloped areas may be as steep as 1V to 3H. The sloped area will provide for drainage of the surface water away from the pavement areas.

CONSTRUCTION GUIDELINES

Construction Monitoring

As Geotechnical Engineer of Record for this project, InTEC should be involved in monitoring the foundation installation and earth work activities. Performance of any foundation system is not only dependent on the foundation design, but is strongly influenced by the quality of construction. Please contact our office prior of construction so that a plan for foundation and earthwork monitoring can be incorporated in the overall project quality control program.

Site Preparation

Site preparation will consist of **preparation of the subgrade, and placement of select structural fill.** The project geotechnical **engineer InTEC should approve the subgrade preparation, the fill materials, and the method of fill placement and compaction.**

In any areas where soil-supported floor slabs or pavement are to be used, vegetation and all loose or excessively organic material should be stripped to a minimum depth of six inches and removed from the site. Subsequent to stripping operations, the subgrade should be proof rolled prior to fill placement and recompacted to **95 percent of the maximum dry density as determined by ASTM D 698 test method within one percent below or three percent above optimum moisture content.** The exposed subgrade should not be allowed to dry out prior to placing structural fill.

Voids caused by site preparation should be replaced with select structural fill and compacted in accordance with the select fill compaction recommendations.

Proof Rolling

Proof rolling should be accomplished in order to locate and density any weak compressible zones under the building and pavement areas and prior to placement of the select fill or base. If any area, where soil-supported floor slabs are to be used, is disturbed, then it should be proof rolled as recommended here. A minimum of 10 passes of a 25-ton pneumatic roller should be used for planning purposes. The operating load and tire pressure should conform to the manufactures specification to produce a minimum ground contact pressure of 90 pound per square inch. Proof rolling should be performed under the inspection of InTEC Geotechnical Engineer: The soils that yield or settle under proof rolling operations should be removed, dried and compacted or replaced with compacted select fill to grade. Density test should be conducted as specified under Control Testing and Filed Observation after satisfactory proof rolling operation.

Select Fill

Any select structural fill used under the building should have a liquid limit less than 40 and a plasticity index in between 5 and 20. The fill should contain no particles greater than 3 inches in diameter. **The percent passing Sieve No. 200 should be less than 30 percent**.

Crushed limestone with sufficient fines to bind the aggregate together is a suitable select structural fill material. The fill materials should be placed in loose lifts not to exceed 8 inches thick (6-inches compacted) and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 698 procedure at a moisture content between optimum minus 1 percent and optimum plus 3 percent of the optimum water content.

Ground Water

In any areas where significant cuts (2-ft or more) are made to establish final grades for building pads, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. Subsurface drains may be required to intercept seasonal groundwater seepage. The need for these or other dewatering devices on building pads should be carefully addressed during construction. Our office could be contacted to visually inspect final pads to evaluate the need for such drains.

The ground water seepage may happen several years after construction if the rainfall rate or drainage changes within the project site or outside the project site. If seepage run off occurs towards the building an engineer should be called on to evaluate its effect and provision of French Drains at this location.

Drainage

Ground water seepage was not encountered in the borings at the time of drilling. However, minor ground water seepage may be encountered within the proposed foundation areas and grading excavations at the time of construction, especially after periods of heavy precipitation. **Small quantities of seepage may be handled by conventional sump and pump methods of dewatering.**

Temporary Drainage Measures

Temporary drainage provisions should be established, as necessary, to minimize water runoff into the construction areas. If standing water does accumulate, it should be removed by pumping as soon as possible.

Adequate protection against sloughing of soils should be provided for workers and inspectors entering the excavations. This protection should meet O.S.H.A. and other applicable building codes.

Time of Construction

If the foundation slab is installed during or after an extended dry period, the slab may experience greater movement around the edges when the soil moisture content increases, such as due to rain or irrigation. Similarly, a slab installed during or after a wet period may experience greater movement around the edges during the subsequent drying of the soils.

Control Testing and Field Observation

Subgrade preparation and select structural fill placement should be monitored by the project geotechnical engineer or his representative of InTEC. As a guideline, at least one in-place density test should be **performed for each 3,000 square feet of compacted surface lift.** However, a minimum of three density tests should be performed by InTEC on the subgrade or per lift of compaction. Any areas not meeting the required compaction should be re-compacted and retested until compliance is met.

Foundation Construction and Field Observation

It is recommended that all grade beam excavations be extended to the final grade and grade beams constructed as soon as possible to minimize potential damage to the bearing soils. Exposure to environment may weaken the soils at the bearing level if the foundation excavation remains open for long periods of time. The foundation bearing level should be free of loose soil, ponded water or debris. **The bearing level should be inspected by the project geotechnical engineer or his representative of InTEC and approved before placement of concrete.**

<u>Drilled Piers</u>

<u>Field Observations</u> Each drilled pier excavation must be monitored by a qualified individual who is familiar with the geotechnical aspects of the soil stratigraphy, structural configuration, foundation design details and assumptions prior to placing concrete. This is to observe that:

(1) The footing has been drilled to the specific dimensions at the correct depth established by the previously mentioned criteria;

(2) The bottom of the footing is concentric to the pier shaft;

(3) The pier shaft has been drilled plumb within specified tolerances along its total length;

(4) Excessive cuttings, build-up and soft, compressible material have been moved from the bottom of the excavation.

Placement of concrete should be accomplished as soon as possible for each footing to reduce changes in the moisture content or the state of stress of the foundation soils. We recommend that footings be concreted with the approval of the project geotechnical engineer. No completed footing excavation should be left overnight without concreting.

Surface run off or ground water seepage accumulating in the excavation should be pumped out and the condition of the bearing surface should be evaluated immediately prior to placing concrete.

<u>Casing</u> **Ground water was not encountered in the borings at the time of drilling**. However, ground water seepage may be encountered at shallower depths at the time of pier construction, especially after periods of heavy rainfall at some pier locations. Zones of sloughing soils may also happen during pier construction (gravelly soils were encountered in one of the borings). It is our understanding that TxDOT specifications are planned to be used and these specifications include the cost of casing if needed. If specifications other than TxDOT are used, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing which may be required.

If groundwater seepage occurs, the use of casing should help to minimize groundwater inflow into the pier excavation, although it may not alleviate seepage from the soils. If seepage persists even after casing installation, the water should be pumped out of the excavation immediately prior to placing concrete. **<u>If groundwater inflow is too severe to be controlled by pumping, the concrete should be tremied to the full</u>**

depth of the excavation to effectively displace the water. In this case, a "clean-out" bucket should be utilized to remove the soil from the pier bottom before placing steel and concrete.

If casing is utilized, removal of the casing should be performed with extreme care under proper supervision to minimize mixing of the surrounding soil and water with the concrete.

Variation in drilled shaft depths should be anticipated at this site. Contract documents should include pay items for constructing drilled shafts on a unit price basis.

Drilling Equipment **High power and high torque drilling equipment will be required to drill through the hard silty clays with marl seams encountered at this site**. Based on past experience, sandstone seams should be anticipated at this site. The information presented in the boring logs should not be used as a basis for selecting drilling equipment or for budgeting purposes by the contractor

DRAINAGE AND MAINTENANCE

Final drainage is very important for the performance of the proposed structure and the pavement. Plumbing leaks should be repaired as soon as possible in order to minimize the magnitude of moisture change under the slab. Large trees and shrubs should not be planted in the immediate vicinity of the structures, since root systems can cause a substantial reduction in soil volume in the vicinity of the trees during dry periods.

Adequate drainage should be provided to reduce seasonal variations in moisture content of the soils. All pavement and sidewalks within 10-ft of the structure should be sloped away from the structure to prevent ponding of water around the structure. Final grades within 10-ft of the structure should be adjusted to slope away from structures preferably at a minimum slope of 3 percent. Maintaining positive surface drainage throughout the life of the structure is essential.

In areas with pavement or sidewalks adjacent to the new structure, a positive seal must be provided and maintained between the structure and the pavement or sidewalk to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and flat-work is not uncommon. Maximum grades practical should be used for paving and flatwork to prevent areas where water can pond. In addition, allowances in final grades should take into consideration post construction movement of flatwork particularly if such movement would be critical. Normal maintenance should include inspection of all joints in paving and sidewalks, etc. as well as re-sealing where necessary.

Trench backfill for utilities should be properly placed and compacted as outlined in this report and in accordance with requirements of local City Standards. Since granular bedding backfill is used for most utility lines, the backfilled trench should be prevented from becoming a conduit and allowing an access for surface or subsurface water to travel toward the new structures. Concrete cut-off collars or clay plugs should be provided where utility lines cross the structure lines to prevent water traveling in the trench backfill and entering beneath the structure.

The PVR values estimated and stated under *Vertical Movements* are based on provision and maintenance of positive drainage to divert water away from the structures and the pavement areas. If the drainage is not maintained, the wetted front may move below the assumed twelve feet depth, and resulting PVR will be much greater than 2 or 3 times the stated values under *Vertical Movements*. Utility line leaks may contribute water and cause similar movements to occur.

LIMITATIONS

The analyses and recommendations submitted in this report are based upon the data obtained from **four borings** drilled at the site.

This report may not reflect the exact variations of the soil conditions across the site. The nature and extent of variations across the site may not become evident until construction commences.

The information contained in this report and on the Boring Logs are not intended to provide the contractor with all the information needed for proper selection of equipment, means and methods, or for cost and schedule estimation purposes. The use of information contained in the report for bidding purposes should be done at the contractor's option and risk.

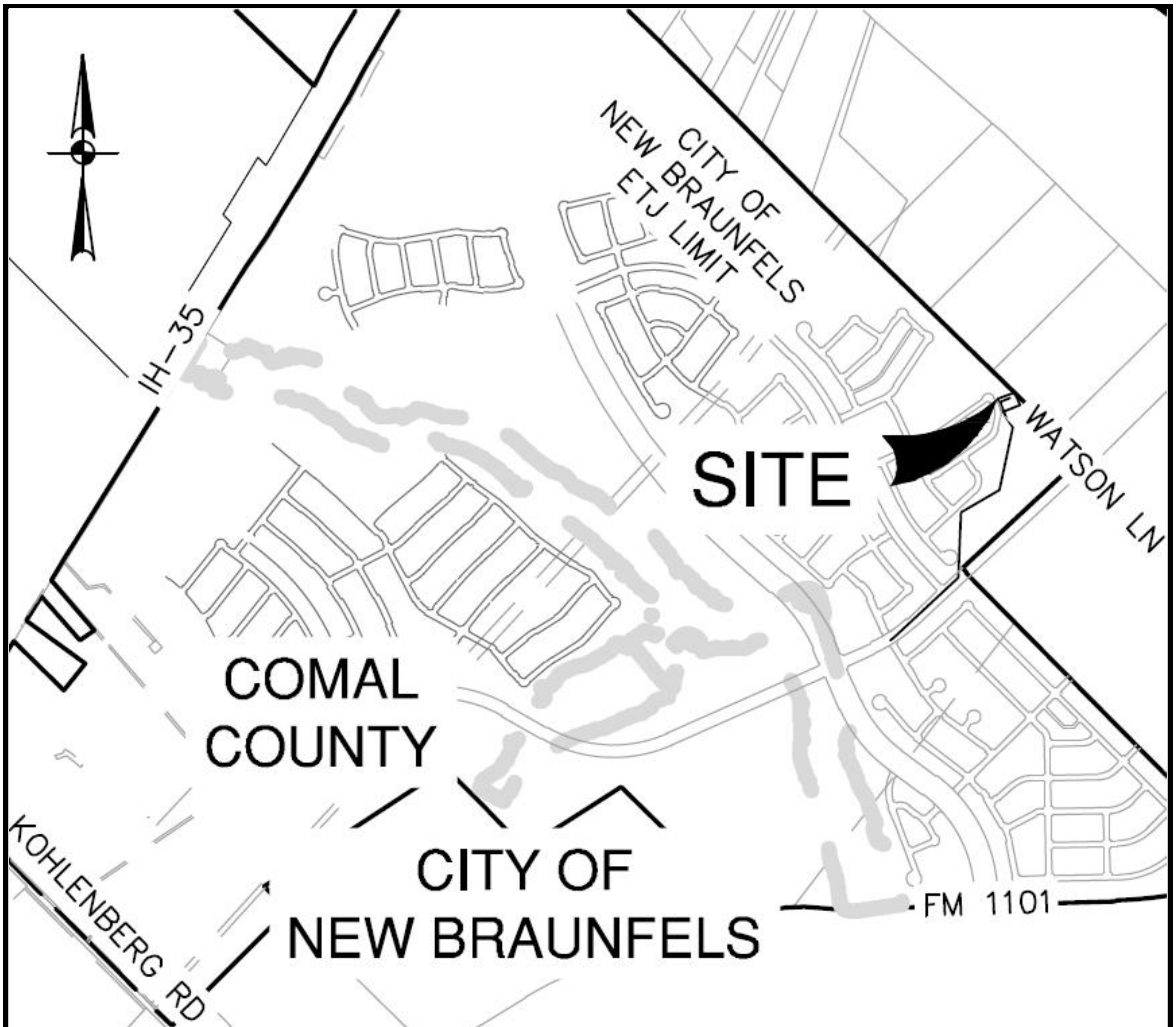
If variations then appear evident, it will be necessary to re-evaluate our recommendations after performing on-site observations and tests to establish the engineering significance of any variations.

The project geotechnical engineer should review final plan for the proposed structures so that he may determine if change in the foundation recommendations are required.

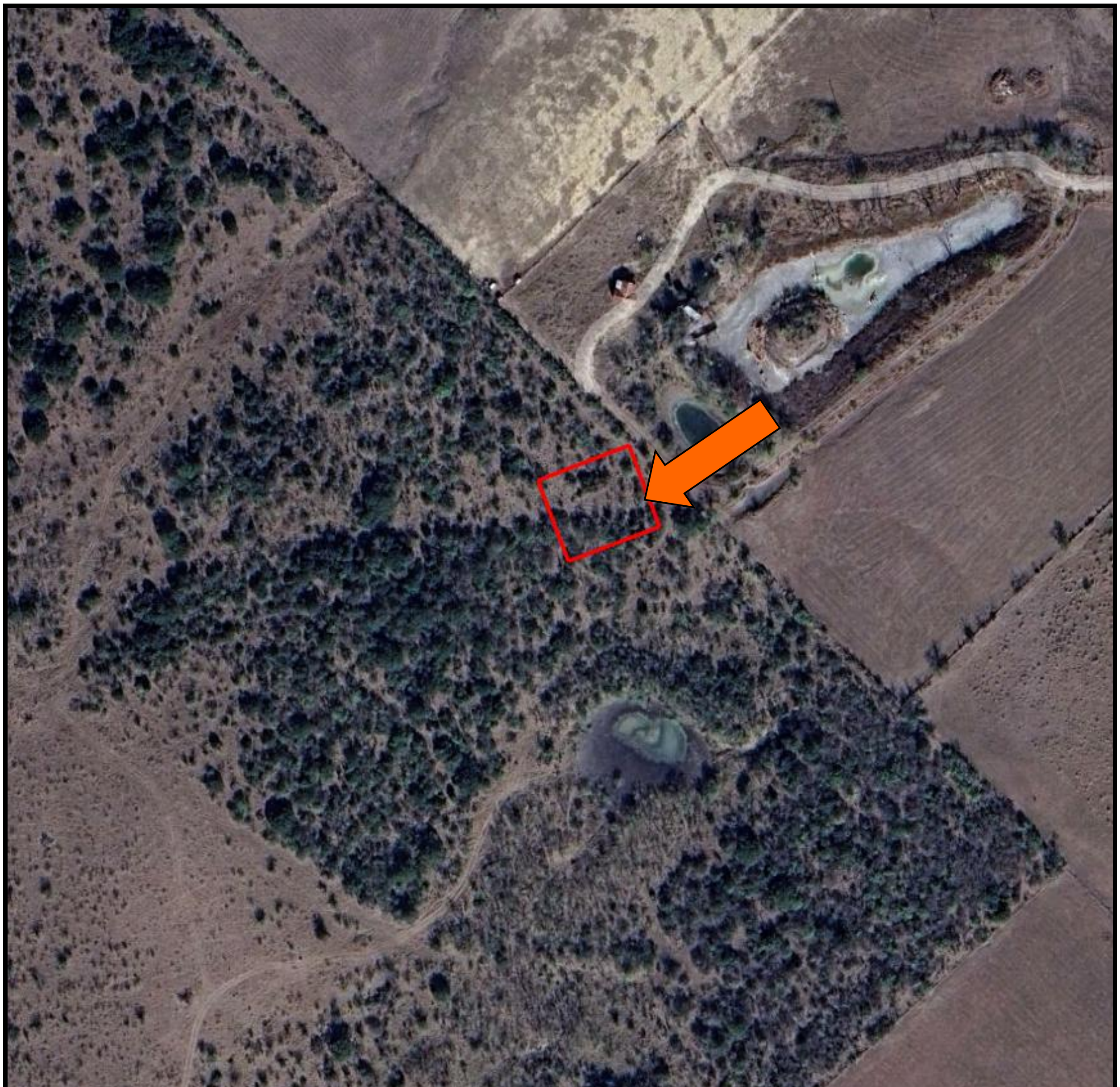
The project geotechnical engineer declares that the findings, recommendations, or professional advice contained herein have been made and this report prepared in accordance with generally accepted professional engineering practice in the fields of geotechnical engineering and engineering geology. InTEC should be engaged to review the recommendations presented in this report if cut and fill operations are performed or if any changes are made to drainage conditions. No other warranties are implied or expressed.

This report has been prepared for the exclusive use of **Southstar Communities** and their design team for the design evaluation for the **proposed new Lift Station #3 at Mayfair Subdivision in New Braunfels, Texas.**

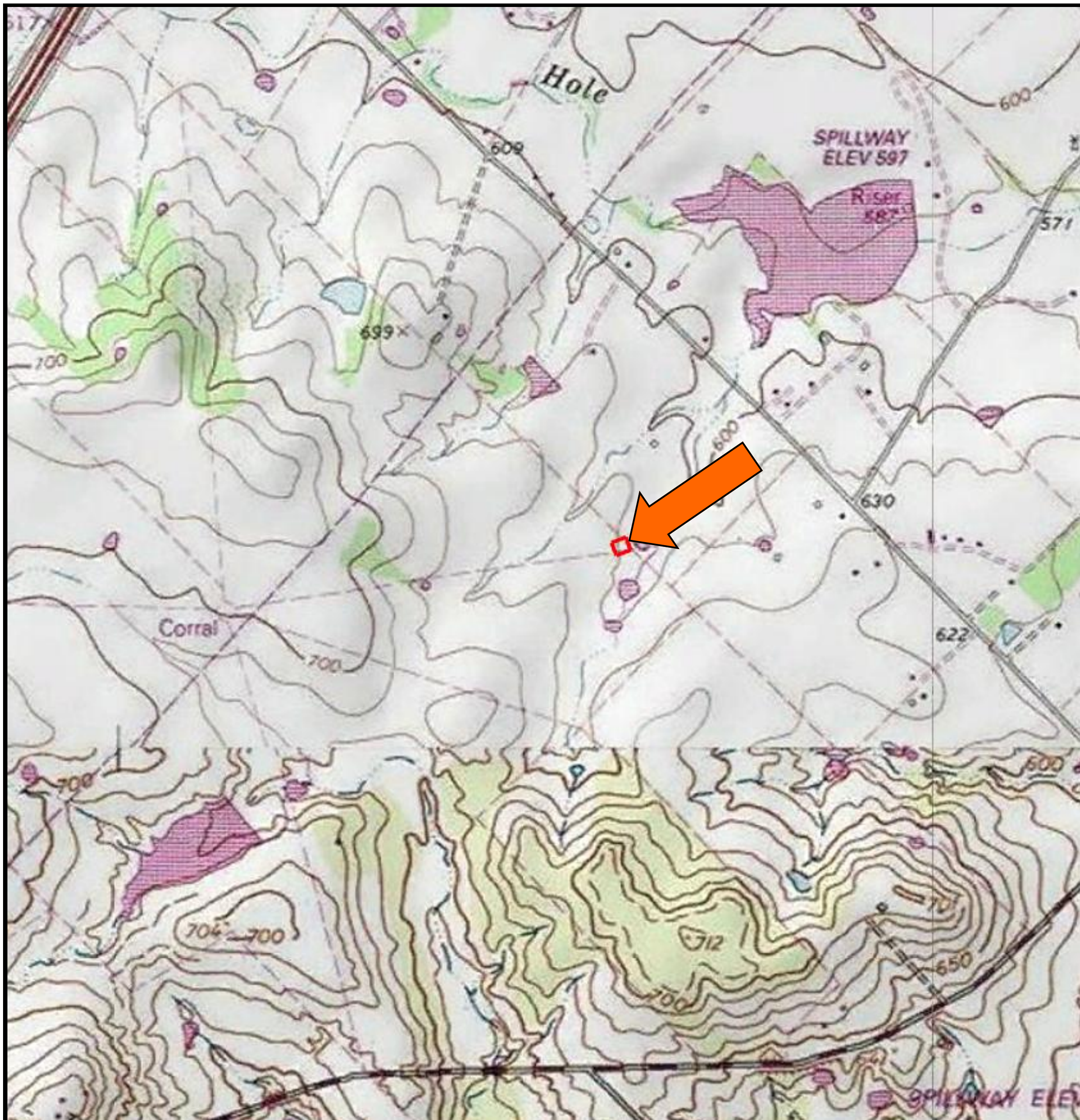
Illustration Section



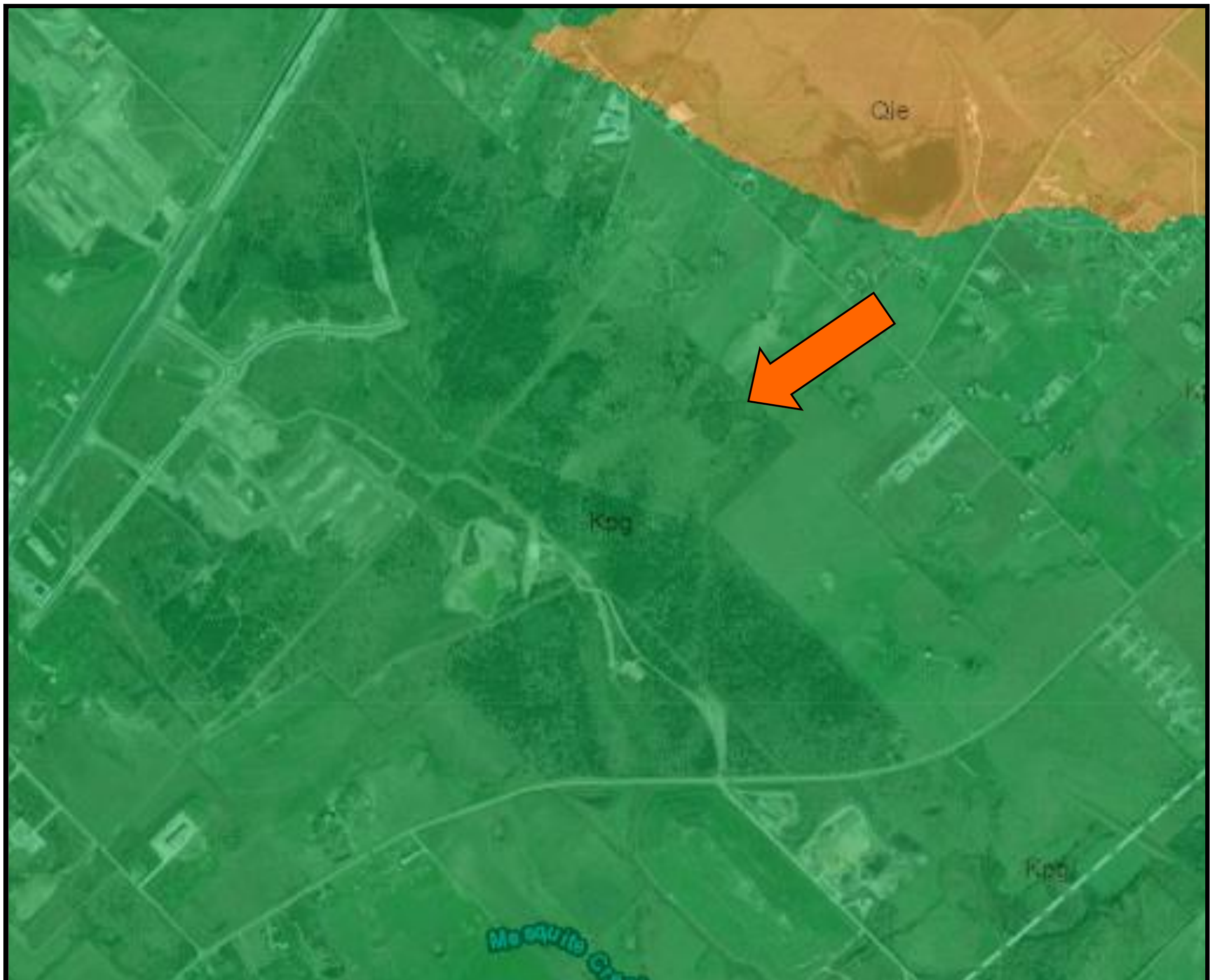
Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Vicinity Map**	
	InTEC Project Number: **S251705**	Date: 01/29/2025



Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Aerial Map—Approximate Location**	
	InTEC Project Number: **S251705**	Date: 01/29/2025



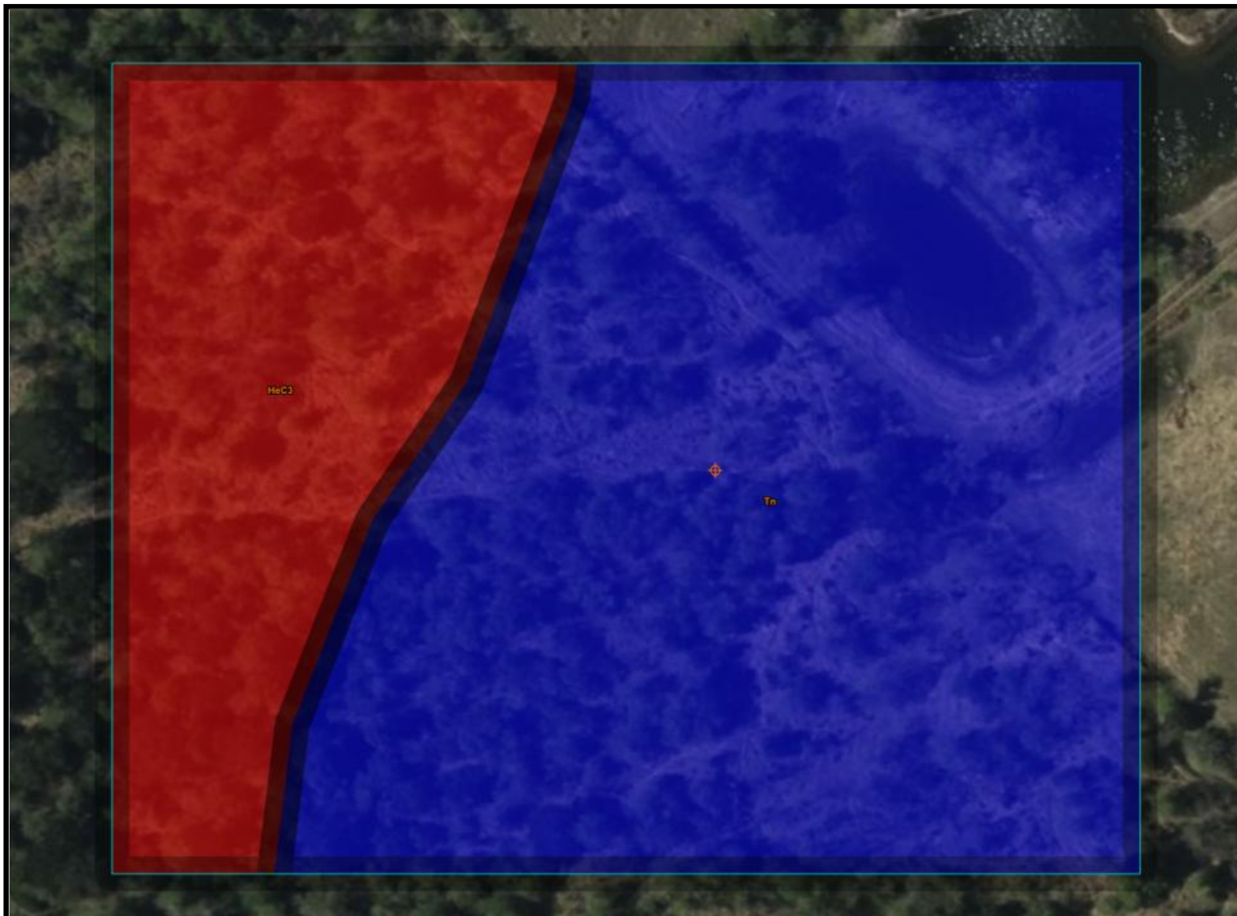
Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Topographic Map—Approximate Location**	
	InTEC Project Number: **S251705**	Date: 01/29/2025



Kpg—Pecan Gap Chalk

chalk and chalky marl, more calcareous westward, very light yellow to yellowish brown, weathers to form moderately deep soil, seldom exposed; Exogyra ponderosa common; thickness 100-400 feet, thins westward to eastern Medina County where it is overlain by Anacacho Limestone, beyond this point included with Austin Chalk

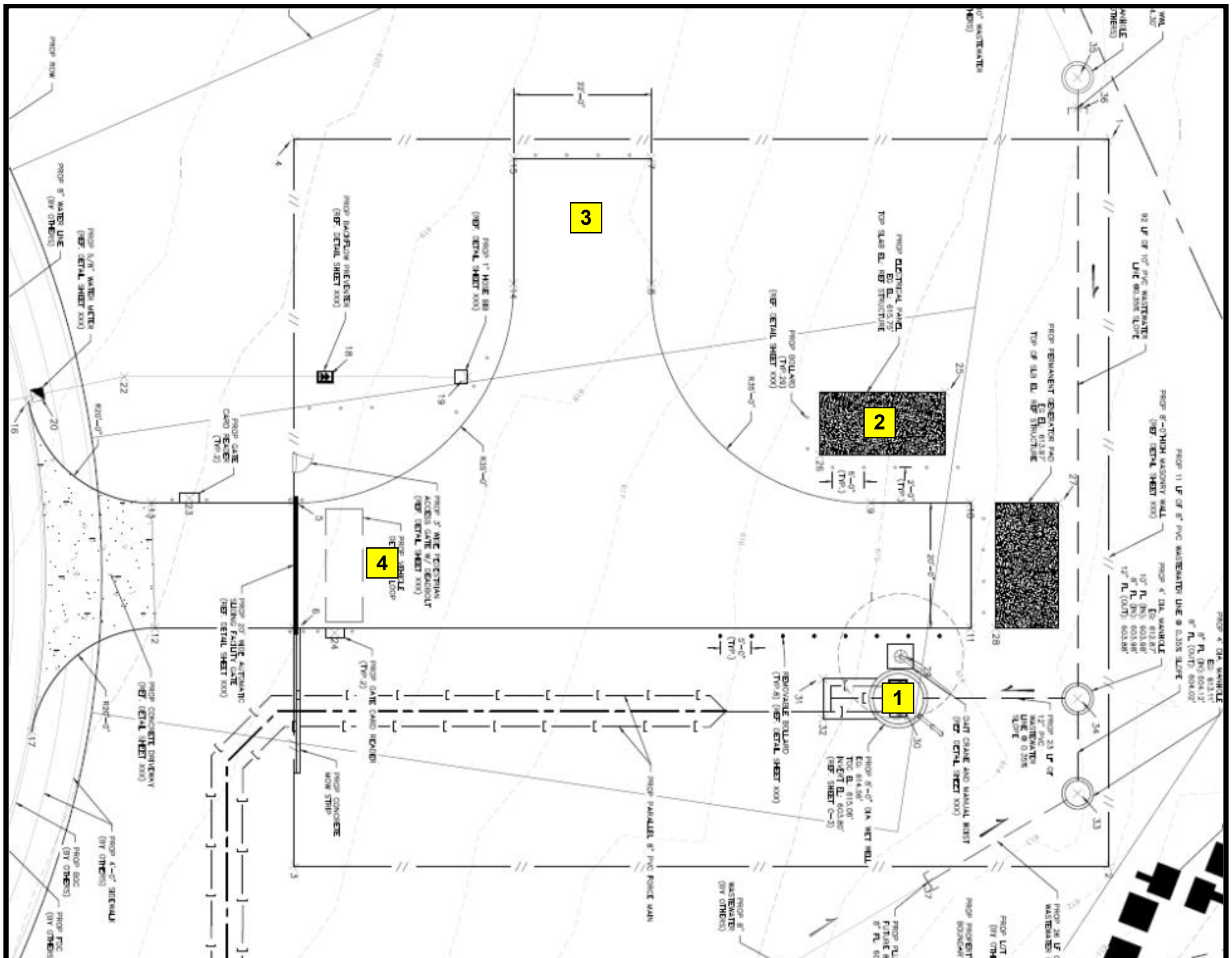
Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Geologic Map—Approximate Location**	
	InTEC Project Number: **S251705**	Date: 01/29/2025



Comal and Hays Counties, Texas

Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
HeC3—Heiden clay, 3 to 5 percent slopes, eroded														
Heiden, moderately eroded	85	D	0-13	Clay	CH	A-7-6	0- 0- 0	0- 0- 0	96-98-100	90-96-100	80-94-100	65-81- 94	50-60 -80	30-40-55
			13-22	Silty clay, clay	CH	A-7-6	0- 0- 0	0- 0- 0	96-98-100	90-96-100	80-94-100	65-81- 98	50-60 -80	30-40-55
			22-58	Silty clay, clay	CH	A-7-6	0- 0- 0	0- 0- 0	96-98-100	90-96-100	80-94-100	65-81- 98	50-60 -80	30-40-55
			58-80	Clay	CH	A-7-6	0- 0- 0	0- 0- 0	98-100-100	97-100-100	86-98-100	71-86- 95	50-70 -80	30-45-55
Tn—Tinn clay, 0 to 1 percent slopes, frequently flooded														
Tinn	85	D	0-17	Clay	CH	A-7, A-7-6	0- 0- 0	0- 0- 0	100-100-100	96-98-100	84-91-100	73-79- 91	61-66 -76	37-41-49
			17-57	Silty clay, clay	CH	A-7, A-7-6	0- 0- 0	0- 0- 0	100-100-100	96-98-100	81-91-100	70-79- 91	58-66 -76	35-41-49
			57-80	Silty clay, clay	CH	A-7, A-7-6	0- 0- 0	0- 0- 0	100-100-100	92-96-100	78-89-100	67-78- 91	58-66 -76	35-41-49

Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Soil Map—Approximate Location**	
	InTEC Project Number: **S251705**	Date: 01/29/2025



Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	**Approximate Boring Locations**	
	InTEC Project Number: **S251705**	Date: 01/29/2025

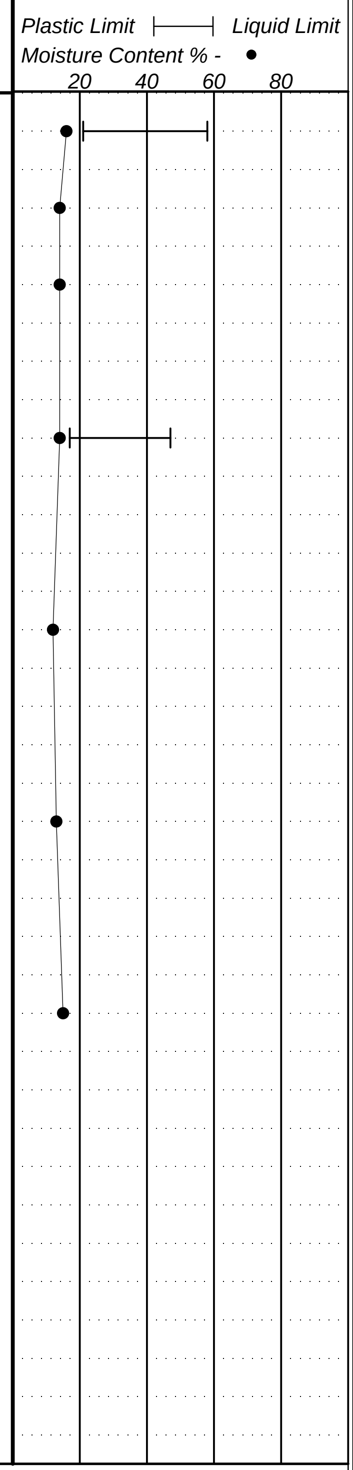
PROJECT: Lift Station #3 - Mayfair Sub
LOCATION: New Braunfels, Texas
CLIENT: Southstar Communities

PROJECT NO: S251705
DATE: 04/11/2025

intec

BORING NO. B-1

DEPTH (feet)	SAMPLES	SOIL DESCRIPTION	% MINUS 200 SIEVE	UNIT DRY WT IN PCF	S.S. BY P.P	BLOWS PER FOOT	SHEAR STRENGTH TSF	LIQUID LIMIT	PLASTICITY INDEX
0		Dark Brown Clay							
	SS					12		58	37
	SS					18			
5	SS					21			
		Tan and Gray Clay							
	SS					28		47	30
10									
		Tan to Tan and Gray Silty Clay -with Caliche -with Marl Seams							
15	SS					50			
20	SS					50/3"			
25	AU								
30		Refusal at 29-ft							
35									



Notes:

Ground Water Observed: No

Completion Depth (ft): 29

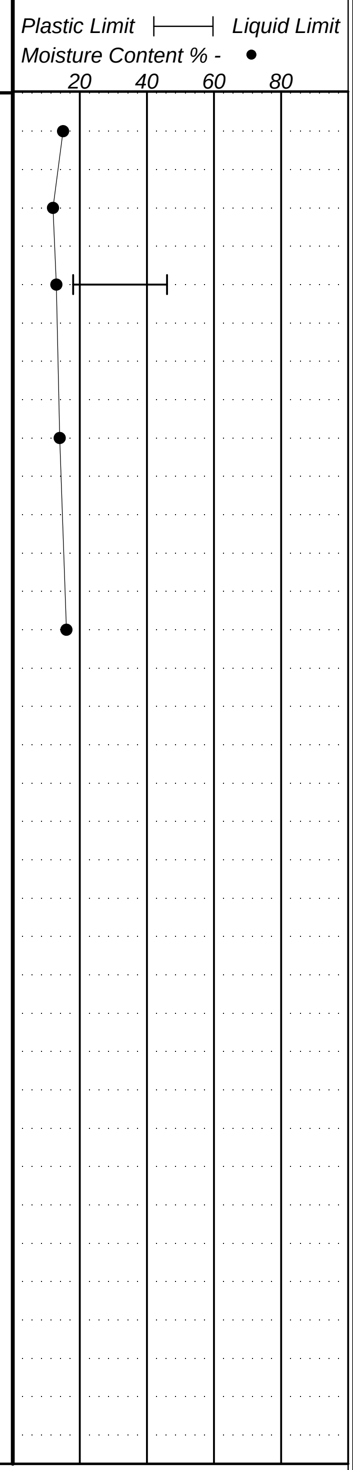
S.S by P.P - Shear Strength in TSF by Hand Penetrometer
S.S. - Split Spoon Sample
S.T. - Shelby Tube Sample
HA - Hand Auger
AU - Auger Sample

PROJECT: Lift Station #3 - Mayfair Sub
LOCATION: New Braunfels, Texas
CLIENT: Southstar Communities

PROJECT NO: S251705
DATE: 04/11/2025

BORING NO. B-2

DEPTH (feet)	SYMBOL	SAMPLES	SOIL DESCRIPTION	% MINUS 200 SIEVE	UNIT DRY WT IN PCF	S.S. BY P.P	BLOWS PER FOOT	SHEAR STRENGTH TSF	LIQUID LIMIT	PLASTICITY INDEX
0			Dark Brown Clay							
		SS					17			
		SS					20			
5		SS	Tan to Tan and Gray Silty Clay -with Caliche				41		46	28
		SS					69/10"			
10										
		SS					50/6"			
15										
20										
25										
30										
35										



Notes:

Ground Water Observed: No

Completion Depth (ft): 15

S.S by P.P - Shear Strength in TSF by Hand Penetrometer
S.S. - Split Spoon Sample
S.T. - Shelby Tube Sample
HA - Hand Auger
AU - Auger Sample

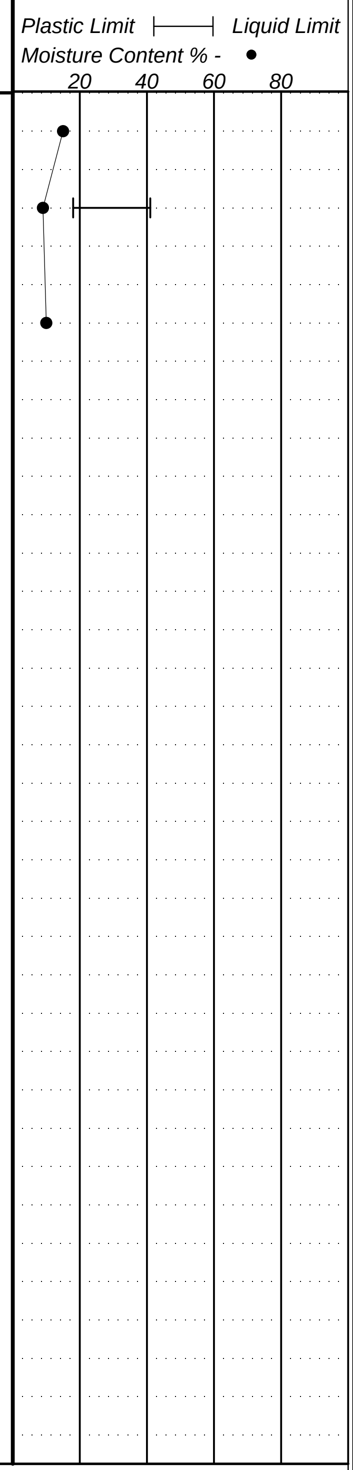
PROJECT: **Lift Station #3 - Mayfair Sub**
LOCATION: New Braunfels, Texas
CLIENT: Southstar Communities

PROJECT NO: **S251705**
DATE: 04/11/2025

intec

BORING NO. B-3

DEPTH (feet)	SYMBOL	SAMPLES	SOIL DESCRIPTION	% MINUS 200 SIEVE	UNIT DRY WT IN PCF	S.S. BY P.P	BLOWS PER FOOT	SHEAR STRENGTH TSF	LIQUID LIMIT	PLASTICITY INDEX
0			Dark Brown Clay							
		SS					15			
		SS	Tan to Tan and Gray Silty Clay				24		41	23
5										
		SS					36			
10										
15										
20										
25										
30										
35										



Notes:

Ground Water Observed: No

Completion Depth (ft): 7

S.S by P.P - Shear Strength in TSF by Hand Penetrometer
S.S. - Split Spoon Sample
S.T. - Shelby Tube Sample
HA - Hand Auger
AU - Auger Sample

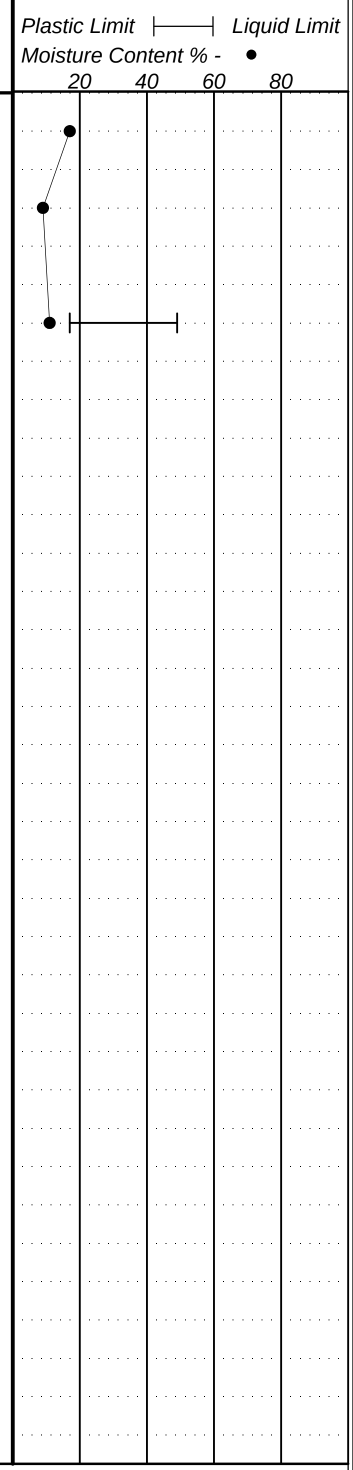
PROJECT: **Lift Station #3 - Mayfair Sub**
LOCATION: New Braunfels, Texas
CLIENT: Southstar Communities

PROJECT NO: **S251705**
DATE: 04/11/2025

intec

BORING NO. B-4

DEPTH (feet)	SYMBOL	SAMPLES	SOIL DESCRIPTION	% MINUS 200 SIEVE	UNIT DRY WT IN PCF	S.S. BY P.P	BLOWS PER FOOT	SHEAR STRENGTH TSF	LIQUID LIMIT	PLASTICITY INDEX
0		SS	Dark Brown Clay				14			
		SS	Tan Silty Clay -with Caliche and Gravel				36			
5		SS					42		49	32
10										
15										
20										
25										
30										
35										



Notes:

Ground Water Observed: No

Completion Depth (ft): 7

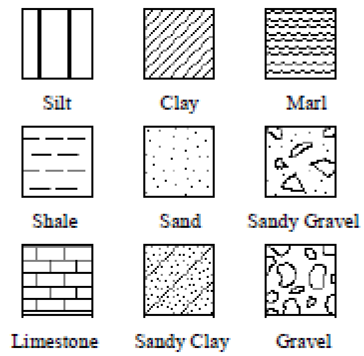
S.S by P.P - Shear Strength in TSF by Hand Penetrometer
S.S. - Split Spoon Sample
S.T. - Shelby Tube Sample
HA - Hand Auger
AU - Auger Sample

KEY TO CLASSIFICATIONS AND SYMBOLS

Soil Fractions

Component	Size Range
Boulders	Greater than 12"
Cobbles	3" - 12"
Gravel	3" - #4 (4.76mm)
Coarse	3" - ¾"
Fine	¾" - #4
Sand	#4 - #200 (0.074mm)
Coarse	#4 - #10 (2.00mm)
Medium	#10 - #40 (0.42mm)
Fine	#40 - #200 (0.074mm)
Silt and Clay	Less than #200

Soil or Rock Types
(Shown in symbols column)
(Predominate Soil Types Shown Heavy)



TERMS DESCRIBING SOIL CONSISTENCY

Description (Cohesive Soils)	Unconfined Compression TSF	Blows/Ft. Std. Penetration Test	Description (Cohesionless Soils	Blows/Ft. Std. Penetration Tests
Very Soft	0.25	<2	Very Loose	0 – 4
Soft	0.25 – 0.50	2 – 4	Loose	4 – 10
Firm	0.50 – 1.00	4 – 8	Medium Dense	10 – 30
Stiff	1.00 – 2.00	8 – 15	Dense	30 – 50
Very Stiff	2.00 – 4.00	15 – 30	Very Dense	50
Hard	>4.00	>30		

SOIL STRUCTURE

Calcareous	Containing deposits of calcium carbonate; generally nodular.
Slickenside	Having inclined planes of weakness that are slick and glossy in appearance.
Laminated	Composed of thin layers of varying color and texture.
Fissured	Containing shrinkage cracks frequently filled with fine sand or silt. Usually more or less vertical.
Interbedded	Composed of alternate layers of different soil types.
Jointed	Consisting of hair cracks that fall apart as soon as the confining pressure is removed.
Varved	Consisting of alternate thin layers of sand, silt or clay formed by variations in sedimentations during the various seasons of the year, of often exhibiting contrasting colors when partially dried. Each layer is generally less than ½" in thickness.
Stratified	Composed of, or arranged in layers (usually 1 inch or more)
Well-graded	Having a wide range of grain sizes and substantial amount of all intermediate particle sizes.
Poorly or Gap-graded	Having a range of sizes with some intermediate sizes missing.
Uniformly-graded	Predominantly of one grain size.

Appendix

Subsurface Exploration and Foundation Analysis Proposed New Lift Station #3 Mayfair Subdivision New Braunfels, Texas	InTEC Project Number: **S251705**	Date: 01/29/2025

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org