**Subsurface Exploration and Foundation Analysis**
**Proposed New Kneupper Business Park**
**Kneupper Lane & Stevens Avenue**
**Converse, Texas**

**InTEC Project No. S241748**
October 14, 2024

**Wycoff Development & Construction, LLC**
310 Genesis Boulevard, Suite E
Webster, Texas 77598

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October 14, 2024

**Wycoff Development & Construction, LLC**
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Attention: **Mr. Ross Wycoff**
Email: ross@wycoffdevelopment.com

Re: Subsurface Exploration and Foundation Analysis
**Proposed New Kneupper Business Park**
Kneupper Lane & Stevens Avenue
Converse, Texas

**InTEC Project No. S241748**

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**

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LICENSED
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Vice President

## EXECUTIVE SUMMARY


The soil conditions at the location of **the proposed new Kneupper Business Park at Kneupper Lane & Stevens Avenue in San Antonio, Texas** were obtained from **12 borings drilled to depths of 20 to 35 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 silty clays, brown silty clays, tan silty clays, and marl.
- The results of our laboratory testing and engineering evaluation indicate that the underlying shallow clays are **moderately plastic to highly plastic in character.** Potential vertical movement on the order of **3 ½ to 4 ½ inches** is estimated at the boring locations.
- The proposed structures may be supported by straight shaft drilled piers or slab on grade type foundation systems.
- Ground water seepage 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.

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# INTRODUCTION

## General

This report presents the results of our subsurface exploration and foundation analysis for the **proposed new Kneupper Business Park at Kneupper Lane & Stevens Avenue in San Antonio, Texas.** This project was authorized by **Mr. Ross Wycoff.**

## 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 of the proposed structures. Our scope of services includes the following:

1) drilling and sampling of 12 borings – to depths of 20 to 35 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 analyses of the field and laboratory data in relation to the project requirements;
7) estimation of potential vertical movements;
8) preparation of recommendations for the design and construction phase of the project;
9) preparation of a written geotechnical engineering report for use by the members of the design team in their preparation of design, contract documents, and specifications.

The Scope of Services **did not include slope stability analysis 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 construction of the Kneupper Business Park at Kneupper Lane & Stevens Avenue in San Antonio, Texas. The project consists of four (4) new single-story commercial buildings each with an approximate footprint area of 20,000 sq ft. Proposed finish grade elevations and cut and fill information were not available for our review at the time of our investigation. Straight shaft drilled piers or slab on grade type foundations are anticipated.

A review of the aerial maps indicate that the site has been undeveloped with numerous trees and an old dry pond on the north east part of the tract. A review of the topographic map indicates that the site generally slopes from the east to the west.

# 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.

**Twelve soil test borings were drilled** at the approximate locations shown on the Boring Location Plan, Plate 1, included in the Illustration section of this report. These borings were **drilled to depths of 20 to 35 feet below the presently existing ground surface**. Boring locations were selected by the project geotechnical engineer and established in the field by the drilling crew using normal taping procedures.

## Drilling and Sampling

The soil borings were performed with a drilling rig equipped with a rotary head. Conventional solid stem augers were used to advance the holes 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.

## Field Tests and Water Level Measurements

Penetration Tests – During the sampling procedures, **Standard Penetration Tests were performed** in the borings 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 Logs

A field log was prepared for each boring. Each 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, these logs 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 13** included in the Illustration section. BA key to classification terms and symbols used on the logs is presented on **Plate 14.**

## 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 Standards**

| 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

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

# GENERAL SUBSURFACE CONDITIONS

## Soil Stratigraphy

The soils underlying the site may be grouped into **one to three generalized strata** with similar physical and engineering properties. The lines designating the interface between soil strata on the logs represent approximate boundaries. Transition between materials may be gradual. The soil stratigraphy information at the boring locations are presented in **Boring Logs, Plates 2 thru 13.**

The engineering characteristics of the underlying soils, based on 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 clays to tan and gray silty clays, brown to tan silty clays, and tan silty clays to marl are moderately plastic to highly plastic with tested liquid limit values varying from 19 to 68 and plasticity index values ranging from 06 to 48. The results of Standard Penetration Tests performed within these clays varied from 10 to greater than 50 blows per foot.

The description presented is of a generalized nature to highlight the major soil stratification features and soil characteristics. The Boring Logs should be consulted for specific information at each boring location.

Soil stratigraphy may vary between boring locations. **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.**

## 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 check 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 boreholes. Ground water levels will fluctuate with seasonal climatic variations and changes in the land use.

## 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 the 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,**

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

**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**. 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. **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.**

## DESIGN ENGINEERING ANALYSIS

### Foundation Design Considerations

Review of the borings and test data indicates that the factors presented below will affect the foundation design and construction at this site:

1) The underlying clays are moderately plastic to highly plastic and have a high swell potential.

2) Structures supported at shallow depths will be subjected to potential vertical movements on the order of **3 ½ to 4 ½ inches**.

3) If the finish grade elevation is higher than existing grade elevation, compacted crushed limestone select fill should be used to raise the grade.

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 structure had been estimated using Texas Department of Transportation Test Method 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 to fifteen feet in the project area.

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 ½ to 4 ½ inches was estimated at the existing grade elevation**. These PVR values will be realized if the subsoils are subjected to **moisture changes from average soil moisture conditions to wet soil moisture conditions.** TxDOT method of estimating potential vertical movements of the expansive clays is based on empirical correlations utilizing measured plasticity index values and assumed seasonal fluctuations in moisture content. Higher or lower PVR values may be estimated using other methods. However, the TxDOT method is the most widely used in the project area.

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.

The potential vertical movement estimated and stated is based on provision and maintenance of positive drainage to divert water away from the structure and the pavement areas. If the drainage is not maintained, ponding on water will occur, the wetted front may move below the assumed fifteen feet depth; the resulting potential vertical movement will be much greater than 2 to 3 times the stated PVR values. Utility line leaks may contribute water and cause similar movements to occur.

If the proposed finish grade elevation is higher than the existing grade level, compacted crushed limestone select fill should be used to raise the existing grade level. The select fill should be placed and compacted as recommended under *Select Fill* in the "Construction Guidelines" section in this report.

**Methods to Lower Vertical Movements**

The anticipated potential vertical movements at the building locations may be lowered using a variety of methods. These methods include but are not limited to: (1) removal of existing clays and replacement with compacted select fill, (2) moisture conditioning of on-site clays, (3) chemical stabilization of on-site clays, and (4) horizontal or vertical moisture barriers. Details for removal and replacement and moisture conditioning are presented here. One of the options in Table No. 2 may be considered to lower potential vertical movements at the building location. The site work recommendations presented here are based on the existing grade elevations.

**Table No. 2 – Options to Lower PVR**

| Option | Method to lower anticipated potential vertical movement |
|---|---|
| 1 | Removal of existing clays and Replacement by compacted select fill |
| 2 | Moisture conditioning on-site clays with select fill cap |

- The site work recommendations presented here are based on the existing grade elevations. It is our understanding that the site grading and Finish Floor Elevations have not been established at this time. The recommendations (depth of removal & replacement and moisture conditioning depth) presented here should be re-evaluated once the grading plan and the Finish Floor Elevation data are made available.

- The site work contractor should use the information provided in the Structural Notes section of the structural plan document. **The site work contractor should not select an option from this section.** If the structural notes refer to "preparing the building pad as per geotechnical report", please call InTEC for assistance.

**(1) Removal and Replacement**

The underlying clays may be removed to the stated depths from the existing grade elevation and replaced by compacted select fill. The depth options and the respective anticipated movements after selection of one of the depth options are presented in Table No. 3.

**Table No. 3 – Removal and Replacement Options**

| Depth of Clay Removal and Select Fill Replacement, Feet | Anticipated Potential Vertical Movement, Inches |
|---|---|
| 6 | 1 ½ |
| 7 | 1 |

**(2) Moisture Conditioning of On-site Clays**

The anticipated potential vertical movement may be lowered using moisture conditioned on site clays (thicknesses as presented in Table No. 4 in the following page) and capping with a select fill cap. The exposed clay subgrade after the minimum excavation depth as stated above should be scarified to a depth of 8 inches and re-compacted to a minimum of 92 percent of Standard Proctor (ASTM D 698) maximum dry density at moisture content value of +4 to +7 percentage points above the soil's optimum moisture content. The excavated clays should be moisture conditioned to water content in between Optimum Plus 4 and Optimum Plus 7 percentage points, cured, and be placed in 8-inch thick loose lifts, and uniformly compacted to the same criteria as noted above. Care should be taken that a lift is not allowed to dry or desiccate prior to placing a subsequent lift.

**Table No. 4 – Moisture Conditioning Options**

| Moisture Conditioned On-Site Clays, Feet | Select Fill Cap, Feet | Anticipated Potential Vertical Movements, Inches |
|---|---|---|
| 5 | 2 | 1 ½ |
| 6 | 2 | 1 |

Notes:

- Over-excavation:
    - Select fill: 3-ft outside the perimeter grade beams
    - Moisture conditioned soils: 5-ft outside the perimeter grade beams
- Compaction:
    - Select fill: 6 inch thick compacted lifts, compacted to a minimum of 95 percent of the maximum dry density at a moisture content between optimum minus 1 and optimum plus 3 percent (ASTM D 698).
    - Moisture conditioned soils: 6 inch thick compacted lifts, compacted to a minimum of 92 percent of the maximum dry density at a moisture content between optimum plus 4 and optimum plus 7 percent (ASTM D 698).
- The select fill should be placed and compacted as recommended under *Select Fill* in the "Construction Guidelines" section of this report. Each lift should be tested and approved by the geotechnical engineer before placement of the subsequent lifts.
- The moisture barrier such as a 6 mil plastic should cover the entire moisture conditioned pad area (as noted in the table above).
- The removal and replacement depth can vary across the site to achieve a desired potential vertical movement; this depth is based on a) existing grade elevation and b) finish grade elevation.
- 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 building pad outside the perimeter grade beams 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.

- Moisture conditioning of the existing clays with select fill cap will lower the anticipated potential vertical movements. If a severe drought condition prevails for an extended period of time, the conditioned clays may be getting drier to some extent and can cause shrinkage of the clays and downward movement of the structures. It the drier clays get wet again, it may result in swelling of these clays and cause upward movement of the structure. The drying and subsequent swelling of the conditioned clays may also happen for other reasons such as planting of trees nearby and uniformity of the injection. The result will be the same as discussed above. The magnitude of the movement depends on the extent of moisture change and the uniformity of the moisture conditioning.

- At the time of this investigation, moisture conditioning appear to be less expensive options. However, the accompanying risk of shrink / swell potential and its effect on the structure should be carefully evaluated by the owner in choosing this option.

- 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 building 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 buildings 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 presented 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 building and in the pavement areas. The surface water should be drained as fast as possible around the buildings and driveways. 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 our borings. If this happens, we should be informed. We may revise the foundation recommendations accordingly.

## Foundation Selection

**The type and depth of foundation suitable for a given structure primarily depends on several factors:** the subsurface conditions, the climatic conditions, site topography, site drainage, the function of the structure, the foot print area of the buildings, 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 straight shaft drilled piers or stiffened beam and slab foundation or post tensioned beam and slab foundation to support the proposed structures.

## DRILLED PIER FOUNDATION SYSTEM

### a) Drilled Piers

**Straight shaft drilled piers may be used to support the proposed structures. Piers founded at a depth of 25-ft below the existing grade elevation may be considered.** Piers founded at a depth of **25-ft** may be sized for an allowable end bearing capacity of **12,000 lbs per sq ft**. In addition, an allowable skin friction value of **875 lbs per sq ft** is recommended for the pier shaft portion which is embedded **below the depth of 15-ft** (for piers supported at 25-ft, skin friction value may be used for 10-ft length of the pier). Minimum pier diameter of **18 inches** is recommended.

Uplift 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 = 39 d - - - - - - (1)**

Fu = uplift force in kips due to swelling clays

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 compressive 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 of that shaft, whichever is maximum.

**The lower 10-ft length (for 25-ft piers) of the pier will provide uplift resistance**. An allowable skin friction value of **875 lbs per sq ft** may be used to evaluate the pull-out capacity. Lightly loaded piers may be founded at a depth deeper than 25-ft to generate higher uplift resistance. Please call InTEC for deeper pier depths. If L-Pile parameters are needed, please contact us.

Pier Spacing A minimum clear spacing between any two piers should not be less than **3d where d is the shaft diameter.** If the spacing between the piers is closer than 3d, stress concentrations will occur between the two piers. The concentrated stress may be higher than the allowable bearing capacity. Hence, these piers should be designed for a lower bearing capacity than the maximum allowable. For construction purposes, the minimum pier spacing may be as close as 3 feet provided the first pier has been drilled, concreted and the concrete had achieved its final set before the adjacent pier is drilled.

Grade Beams  A minimum 10-inch void space should be constructed beneath the grade beams to prevent uplift if the underlying clay expands. Cardboard carton forms may be used for this purpose.

Floor Slabs  Two alternatives are available for constructing floor slabs or lower level pavements in conjunction with a drilled pier foundation. The owner may select the alternative best satisfying the required performance criteria.

**Alternative No. 1: Floor slabs, or portions of the floor slab, which have high performance criteria and are movement sensitive in nature, should be structurally suspended above grade because of the anticipated ground movements.** A positive void space of at least 8 inches, preferably more, should be provided beneath the floor slab.

The crawl space should be drained properly with a positive drainage so that water will not accumulate in the crawl space. Excavated on-site material may be used to raise the grade in the crawl space area. However, it should be compacted to 95 percent of the TXDOT 114-E maximum density and tested. Select fill need not be used to raise the grade in the crawl space area.

**Alternative No. 2: Floor slabs within the superstructure may be ground supported provided** (a) the underlying clays are removed as stated in Table Nos. 5 and 6, (b) the owner (after discussing the resulting corresponding potential vertical movements and their effect on the performance of the structure with the project structural engineer and the project architect) determines that the anticipated movements will not adversely affect the performance of the proposed structure and (c) the recommendations in the “Construction Guidelines” section are followed.

Soil supported floor slabs could be doweled to the perimeter grade beams. Doweled slabs that are subjected to heaving will typically crack and develop a plastic hinge along a line which will be approximately 5 to 10 feet inside and parallel to the grade beams.

The floor slab may be cast independent of the grade beams, interior columns, and partitions. These slabs should experience cracking of lower magnitude, but may create difficulties at critical entry points such as doors. A “trip hazard” could result due to resulting differential movements at entryways and difficulty in opening and closing doors could develop.

## SHALLOW FOUNDATION SYSTEMS

**Stiffened grid type beam and slab foundation or post-tensioned slab-on-grade foundation** may be used to support the structures provided all of the items (a) thru (f) are followed:

(a) one of the options presented in the *Vertical Movements* section is used to lower the potential vertical movements,

(b) one of the depth options in the *Vertical Movements* section and the corresponding potential vertical movements are acceptable to the Structural Engineer and owner.

(c) 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 structure.

(d) if the underlying clays are removed to a selected depth option and replaced by compacted crushed limestone select fill, the bottom of the excavation should be thoroughly proof rolled,

(e) all the recommendations presented in the "Construction Guidelines" section are followed, and

(f) if any fill is placed within the building pad area, InTEC should be called to evaluate the effect of fill on the recommendations.

### b) Stiffened Grid Type Beam and Slab Foundation

It is desirable to design the foundations system utilizing the simplifying assumption that the loads are carried by the beams. Allowable Bearing Capacity values are presented for grade beams founded at a) a minimum depth of 24 inches below finish grade elevation and b) supported on or within natural undisturbed soils or compacted crushed limestone select fill. Stiffened grid type beam and slab foundation parameters are presented in Table No. 5 in the following page.

**Table No. 5 – Stiffened Grid Type Beam and Slab Foundation Parameters**

| Anticipated Vertical Movements | Net Allowable Bearing Capacity (psf) * | Unconfined Compressive Strength (tsf) | Design Plasticity Index Value | Soil Support Index | Climatic Rating |
|---|---|---|---|---|---|
| **PVR lowered to 1 ½ inches** | 1,800 | 0.9 | 31 | 0.68 | 17 |
| **PVR lowered to 1 inch** | 1,800 | 0.9 | 27 | 0.68 | 17 |

Methods to lower PVR are presented in Table No. 2.

Notes (*):

- The grade beams should be founded on or within conditioned soils or compacted select fill.
- The grade beams should have a minimum width of 12 inches.
- Allowable Bearing Capacity values as shown 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.

### c) Post-Tensioned Beam and Slab Foundation

Differential vertical movements should be expected for shallow type foundations at this site **due to expansive soil conditions which were encountered.** Differential vertical movements have been estimated for both the center lift and edge lift conditions for post-tensioned slab-on grade construction at this site. These movements were estimated using the procedures and criteria discussed in the Post-Tensioning Institute Manual entitled "Design of Post Tensioned Slabs-on-Ground-Third Edition Manual. The soil moisture content around the structure should be maintained at a consistent level. We recommend that the structural engineer consider the limitations and assumptions of the method by establishing minimum PTI design parameters for foundation system design to account for the relatively low Ym values produced when analyzing certain soil conditions. The estimated PTI differential movements (Ym) are provided in Table No. 6 in the following page.

**Table No. 6 – Post Tension Parameters**

| | PVR lowered to 1 ½ inches | PVR lowered to 1 inch |
|---|---|---|
| **Center Lift (Ym), Inches** | | |
| Grade beams founded at a depth of 24 inches below final grade elevation | 1.6 | 1.5 |
| Grade beams founded at a depth of 30 inches below final grade elevation | 1.4 | 1.3 |
| Grade beams founded at a depth of 36 inches below final grade elevation | 1.2 | 1.1 |
| | | |
| **Edge Lift (Ym), Inches** | | |
| Grade beams founded at a depth of 24 inches below final grade elevation | 2.1 | 1.9 |
| Grade beams founded at a depth of 30 inches below final grade elevation | 1.8 | 1.6 |
| Grade beams founded at a depth of 36 inches below final grade elevation | 1.5 | 1.3 |
| | | |
| **Center Lift (Em values, ft)** | 8.0 | 8.0 |
| **Edge Lift (Em values, ft)** | 4.0 | 4.0 |
| | | |
| **Allowable Bearing Capacity (psf)** | 1,800 | 1,800 |

Methods to lower PVR are presented in Table No. 2.

Notes:

- The grade beams should be founded on or within conditioned clays or compacted select fill.
- The grade beams should have a minimum width of 12 inches.
- Allowable Bearing Capacity values as shown 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.

The Post-Tensioning Institute (PTI) method incorporates numerous design assumptions associated with the derivation of required variables needed to determine the soil design criteria. The PTI method of predicting differential soil movement is applicable only when site moisture conditions are controlled by the climate alone (i.e. not improper drainage or water leaks). The performance of a slab and movement magnitudes can be significantly influenced by yard maintenance, water line leaks, and trees present before and after construction.

If proper drainage is not maintained, resulting potential vertical movements will be much greater than 2 to 3 times the anticipated vertical movements. Significant differences in fill thicknesses from one side of the structure to the other side may result in larger than anticipated differential movements. If such conditions exist or are anticipated, then the above recommended post tension parameters should be reevaluated.

**Moisture Barrier under Floor Slabs**

We recommend placement of a polyethylene moisture barrier under soil supported floor slab to reduce the possibility of moisture migration through the slab.

**Utilities**

Utilities which project through slab-on-grade floors should be designed with either some degree of flexibility or with **sleeves in order to prevent damage to these lines should vertical movement occur.**

Utility excavations may encounter ground water. The ground water can be handled by sump and pump method of dewatering systems.

Deep excavations may encounter ground water which may flow through the gravel fill surrounding the utility pipelines; this water may be near the ground surface at lower points in the subdivision. The gravel should be extended to a nearby creek or drainage feature and pressure head should be relieved by day lighting at the ground surface.

**Contraction, Control, or Expansion Joints**

**Contraction, control or expansion joints should be designed and placed in various portions of the structure.** Properly planned placement of these joints will assist in controlling the degree and location of material cracking which normally occurs due to soil movements, material shrinkage, thermal affects, and other related structural conditions.

**Retaining Wall and Lateral Earth Pressure**

InTEC is not aware of any retaining walls planned at this site. If retaining walls are planned, please contact InTEC. **Some of the grade beams and foundation walls may act as retaining structures in addition to transferring vertical loads. Some cantilever retaining walls may be needed at this site**. The equivalent fluid density values were evaluated for various backfill materials. These values are presented in Table No. 7.

**Table No. 7 – Equivalent Fluid Density Values**

| Backfill Material | Equivalent Fluid Density PCF<br>Active Condition | <br>At Rest condition |
|---|---|---|
| a. Crushed Limestone | 40 | 55 |
| b. Clean Sand | 45 | 60 |
| c. On-site clays | 75 | 95 |

These equivalent fluid densities do not include the effect of seepage pressures, surcharge loads such as construction equipment, vehicular loads or future storage near the walls.

If the basement wall or cantilever retaining wall can tilt forward to generate "active earth pressure" condition, the values under active condition should be used. For rigid non-yielding walls which are part of the building, the values "at rest condition" should be used.

The compactive effort should be controlled during backfill operations. Over-compaction can produce lateral earth pressures in excess of at-rest magnitudes. Compaction levels adjacent to below-grade walls should be maintained between 95 and 98 percent of standard Proctor (ASTM D 698) maximum dry density.

**The backfill behind the wall should be drained properly.** The simplest drainage system consists of a drain located near the bottom of the wall. The drain collects the water that enters the backfill and this may be disposed of through outlets in the wall called "weep holes". To ensure that the drains are not clogged by fine particles, they should be surrounded by a granular filter. In spite of a well-constructed toe drain, substantial water pressure may develop behind the wall if the backfill consists of clays or silts. A more satisfactory drainage system, consisting of a back drain of 12 inches to 24 inches width gravel may be provided behind the wall to facilitate drainage.

The maximum toe pressure for footings founded two feet below finish grade elevation should not exceed 1,600 pounds per square foot. An adhesion value of 600 pounds per square foot may be used under the wall footings to check against sliding. This adhesion value is applicable for retaining wall bases supported on the existing clay soils.

Some retaining wall bases may be supported on or within compacted select fill material. For these wall bases, a coefficient of sliding friction value of 0.4 is recommended.

If passive pressure is required at any location, we should be informed. We can provide the passive pressure for that particular condition after considering the rigidity and soil-structure interaction characteristics of that structure.

Some retaining wall bases may be supported on or within compacted select fill material. For these wall bases, a coefficient of sliding friction value of 0.4 is recommended. If passive pressure is required at any location, we should be informed. We can provide the passive pressure for that particular condition after considering the rigidity and soil-structure interaction characteristics of that structure.

## Seismic Design Criteria

The following seismic design criteria are presented based on the International Building code 2021 (IBC 2021) Section 1613 - Earthquake Loads. These criteria include the seismic site class and the spectral acceleration values and presented in Table No. 8.

**Table No. 8 – Seismic Design Criteria**

| | |
|---|---|
| **Site Class**<br>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 2021 | $S_s = 0.08g$ |
| **Maximum considered earthquake 1.0 sec** spectral response acceleration – Figure 1613.2.1(2) IBC 2021 | $S_1 = 0.031g$ |

Note: the 2021 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 borings.

# 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 removal of the organic material, preparation of the subgrade, and placement of select structural fill. The project geotechnical engineer 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. Each lift should be tested by InTEC geotechnical engineer or his representative prior to placement of the subsequent lift.

- Any underground utilities underneath the proposed commercial buildings should be located and removed. Otherwise, these existing utilities should be located and permanently sealed. The backfill should be installed in 6-inch lifts and compacted to a minimum 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.

- If cut and fill operations are performed underneath the buildings pad, a uniform fill thickness is recommended. Significant differences in fill thicknesses from one side of the structure to the other side may result in larger than anticipated differential movements.

- Voids caused by site preparation, such as removal of trees or demolition of any existing structures, should be compacted as described below:

## Compaction

Site grading plan is not available for review at this time. Any low areas or disturbed areas encountered during construction should be appropriately prepared and compacted. Any deleterious or wet materials should be removed and wasted. The fill placement in the low areas should not be in a "bowl shape". The sides of the fill area should be "squared up" and the excavated bottom should be proof rolled as described in *Proof Rolling* section of this report. On site material, with no deleterious material, may be used to raise the grade. After proof rolling operation, the fill should be placed in 6-inch lifts and compacted to a minimum of **95 percent of the maximum dry density as determined by ASTM D 698 test method within optimum and three percent above optimum moisture content.** Each lift should be tested by InTEC for compaction compliance and approved before placement of the subsequent lifts. The exposed subgrade should not be allowed to dry out prior to placing structural fill. It is recommended that any given lot does not straddle filled areas and natural areas to help reduce differential movement of the structures.

The excavation boundaries should be set such that buildings or pavement areas do not straddle fill and natural areas. The anticipated potential vertical movement may be significantly affected after the cut and fill operations are performed in this area.

## Proof Rolling

Proof rolling should be accomplished in order to locate and density any weak compressible zones under the commercial buildings and pavement areas and prior to placement of the select fill or base. 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 observation of the InTEC Geotechnical Engineer or his representative. 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.

**Proper site drainage should be maintained during construction so that ponding of surface run-off does not occur and cause construction delays and/or inhibit site access**.

**Select Fill**

Any select fill used under the buildings 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 and compacted to 95 percent of the maximum dry density as determined by test method ASTM D1557 at moisture content within 3 percent of the optimum water content. Each lift should be compacted and tested by InTEC representative to verify compaction compliance and approved before placement of the subsequent lifts.

**General Fill**

General fill materials may consist of clean on-site material, or any clean imported fill material. The purpose of a general fill is to provide soils with good compaction characteristics that will provide uniform support for any non-habitable structures that are not movement sensitive. The general fill material should be free of any deleterious material, construction debris, organic material, and should not have gravels larger than 6 inches in maximum dimension. The top two feet of fill material used underneath pavement areas should not have gravels larger than 3 inches in maximum dimension.

It should be understood that the use of the general fill may result in greater than anticipated potential vertical movements and differential movements. If the greater potential vertical movements or differential soil movements cannot be tolerated, then select fill material should be used and should conform to the Select Fill recommendations.

**General Fill Compaction**

The general fill materials should be placed in lifts not to exceed 8 inches thick and compacted to a minimum of 95 percent of the maximum dry density as determined by test method ASTM D 698 at a moisture content within 3 percent of the optimum water content. Each lift should be compacted and tested by a representative of a geotechnical laboratory to verify compaction compliance and approved before placement of the subsequent lifts.

**Drainage**

Ground water was not encountered in the borings at the time of drilling. However, minor ground water seepage may be encountered within the proposed buildings foundations 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.

**Temporary Construction Slopes**

Temporary slopes on the order of 1H to 1V may be provided for excavations through Strata I and II clays.

Fill slopes on the order of 1H to 1V may be used provided a) the fill materials are compacted as recommended and b) the slopes are temporary.

Fill slopes should be compacted. Compacting operations shall be continued until the slopes are stable but not too dense for planting on the slopes. Compaction of the slopes may be done in increments of 3 to 5-ft in fill height or the fill is brought to its total height for shallow fills.

**Permanent Slopes**

Maximum permanent slope of 1V to 3H is recommended in Stratum I clays. In areas where people walk on sloped areas, a slope of 1V to 5H is recommended.

## 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.

## 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.

## 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 a representative of InTEC and approved before placement of concrete.

Foundation concrete should not be placed on soils that have been disturbed by rainfall or seepage. If the bearing soils are softened by surface water intrusions during exposure or by desiccation, the unsuitable soils must be removed from the foundation excavation and replaced prior to placement of concrete.

Field Observations Each straight shaft drilled pier excavation must be monitored by a qualified individual of InTEC 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. No footings should be concreted without 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.

Casing **Ground water was not encountered in the borings at the time of drilling**. However, minor ground water seepage may be encountered 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. **Zones of sloughing soils may happen during pier construction, especially if gravel seams are encountered during pier drilling.** 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. **If groundwater inflow is too severe to be controlled by pumping, the concrete should be tremied to the full**

**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 clays and marl encountered 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 structures and the pavement. Landscaping, plumbing, and downspout drainage is also very important.** It is vital that all roof drainage be **transported away from the building** so that no water ponds around the buildings which can result in soil volume change under the buildings. **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 foundation soils. **All pavement and sidewalks within 10-ft of the structures should be sloped away from the structures to prevent ponding of water around the foundations.** Final grades within 10-ft of the structures 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 structures is essential.

In areas with pavement or sidewalks adjacent to the new structures, a positive seal must be provided and maintained between the structures 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.**

Several factors relate to civil and architectural design and/or maintenance which can significantly affect future movements of the foundation and floor slab systems:

1. Where positive surface drainage cannot be achieved by sloping the ground surface adjacent to the buildings, a complete system of gutters and downspouts should carry runoff water a minimum of 10-ft from the completed structures.

2. Planters located adjacent to the structures should preferably be **self-contained**. Sprinkler mains should be located a minimum of five feet from the building line.

3. Planter box structures placed adjacent to building should be provided with a means to assure concentrations of water are not available to the subsoils stratigraphy.

4. Large trees and shrubs should not be allowed closer to the foundations than a horizontal distance equal to roughly their mature height due to their significant moisture demand upon maturing.

5. Moisture conditions should be maintained **"constant" around the edge of the slabs.** Ponding of water in planters, in unpaved areas, and around joints in paving and sidewalks can cause slab movements beyond those predicted in this report.

6. **Roof drains should discharge on pavement** or be extended away from the structures. Ideally, roof drains should discharge to storm sewers by closed pipe.

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 building lines to prevent water traveling in the trench backfill and entering beneath the structures.

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 depth of active zone, and resulting **PVR will be much greater than 2 to 3 times the stated values under *Vertical Movements*. Utility line leaks may contribute water and cause similar movements to occur. If drainage is modified in the future or if additional landscaping is done in the vicinity of the structure or if a new structure is constructed in the vicinity of this structure, the effect of such improvements on this project structure should be reevaluated.**

<u>Dry Periods</u>

Close observations should be made around foundations during extreme dry periods to ensure that adequate watering is being provided to keep soil from separating or pulling back from the foundation.

## LIMITATIONS

The exploration and analysis of the subsurface conditions reported herein are considered sufficient to form a reasonable basis for the foundation design. **The recommendations submitted are based upon the available soil information and design details furnished by the Project Manager.**

**Twelve borings** were drilled at the project site. If deviations from the noted subsurface conditions are encountered during construction, they should be brought to the attention of the geotechnical engineer.

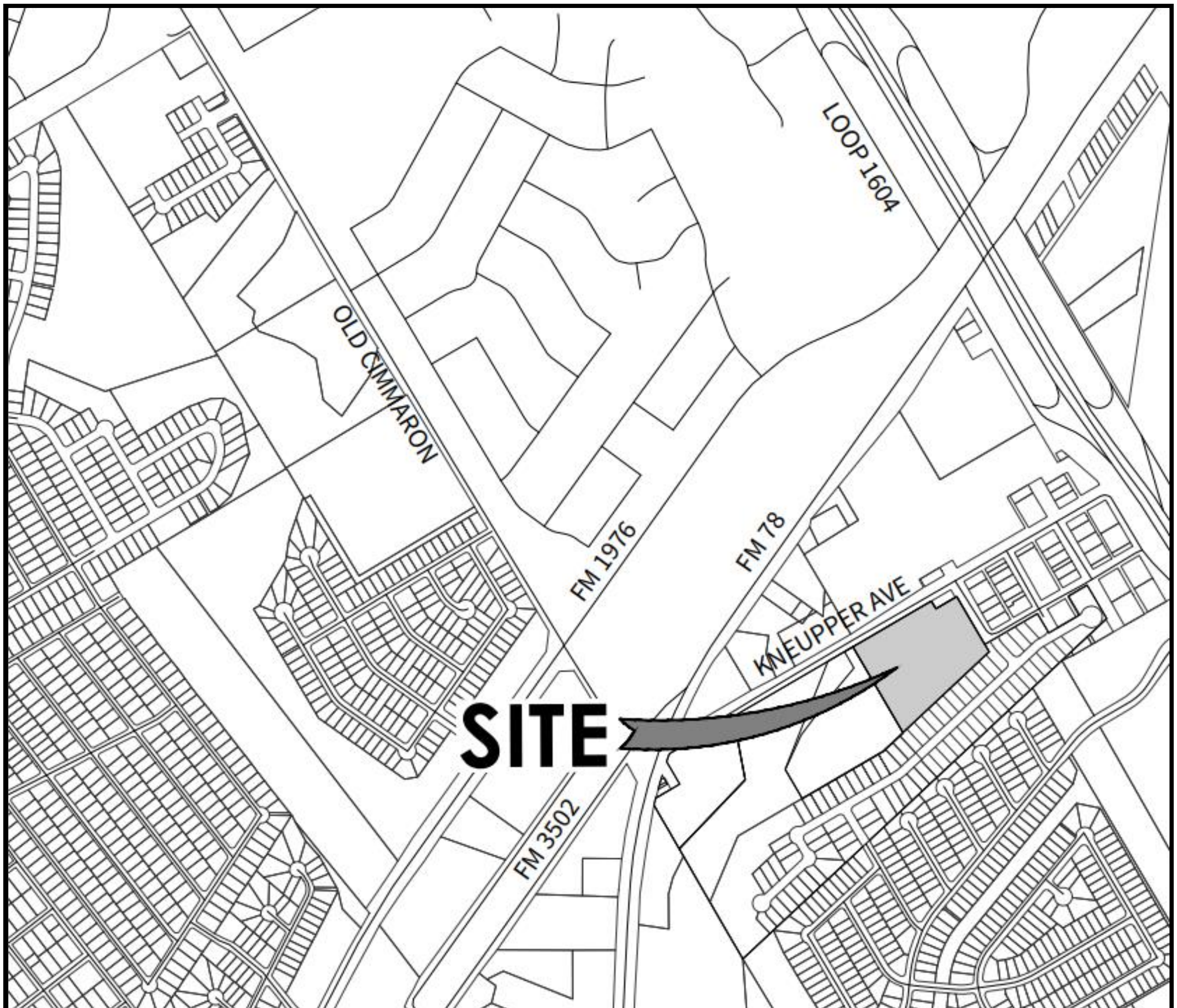
**The information contained in this report and on the Boring Logs are not intended to provide the contractor with all the information needed for proper selection of equipment, means and methods, or for cost and schedule estimation purposes. The contractor should conduct his own field investigation such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

Final plans for the proposed structure should be reviewed by the project geotechnical engineer so that he may determine if change in the foundation recommendations are required.

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

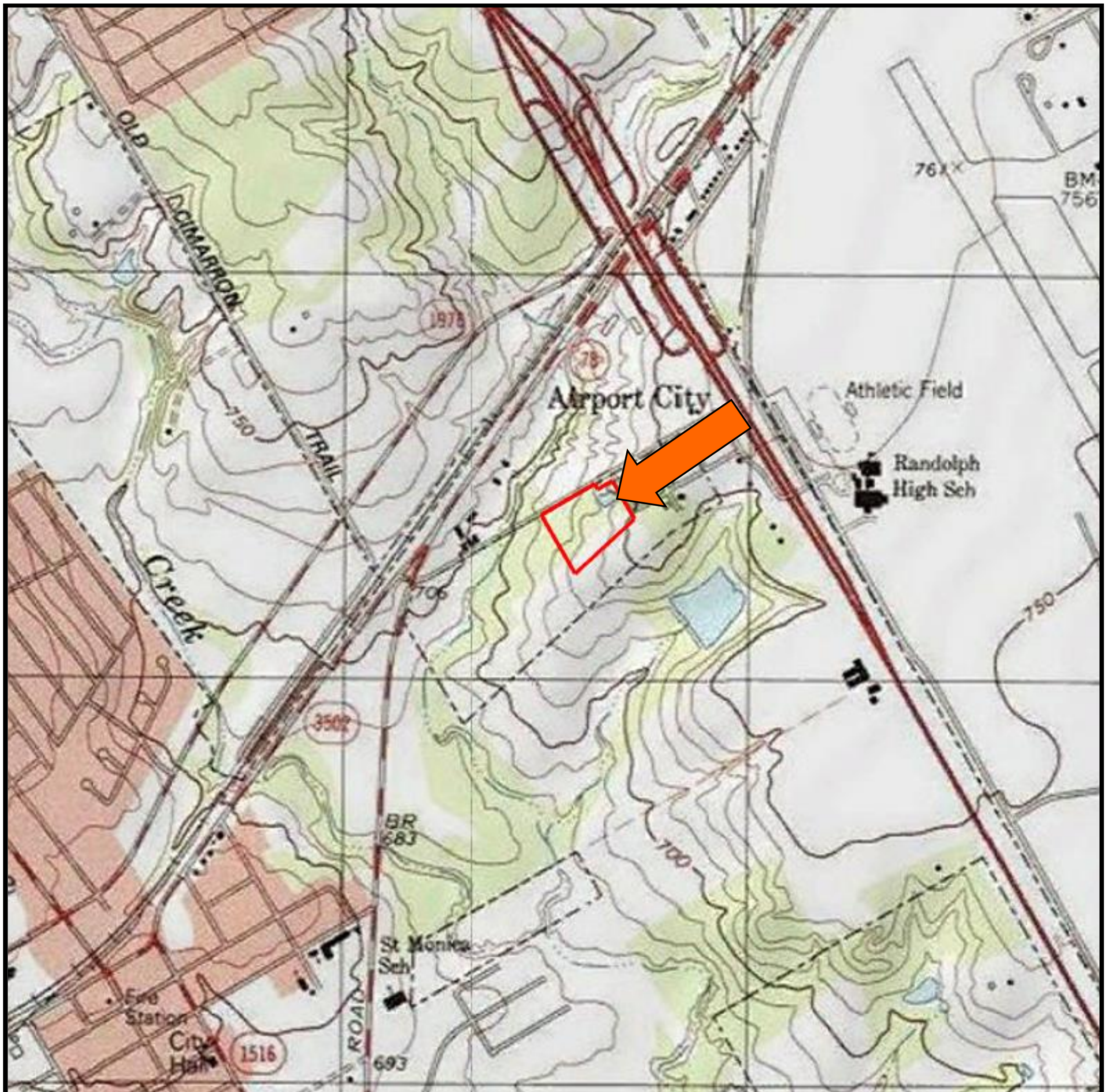
This report has been prepared for the exclusive use of **Wycoff Development & Construction, LLC** for the specific application to the foundation design of the **proposed new Kneupper Business Park at Kneupper Lane & Stevens Avenue in San Antonio, Texas.**

# Illustration Section




| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Vicinity Map** | |
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |


| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Aerial Map—Approximate Location** | |
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |




| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Topographic Map—Approximate Location** | |
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |


**Kknm—Navarro Group and Marlbrook Marl**

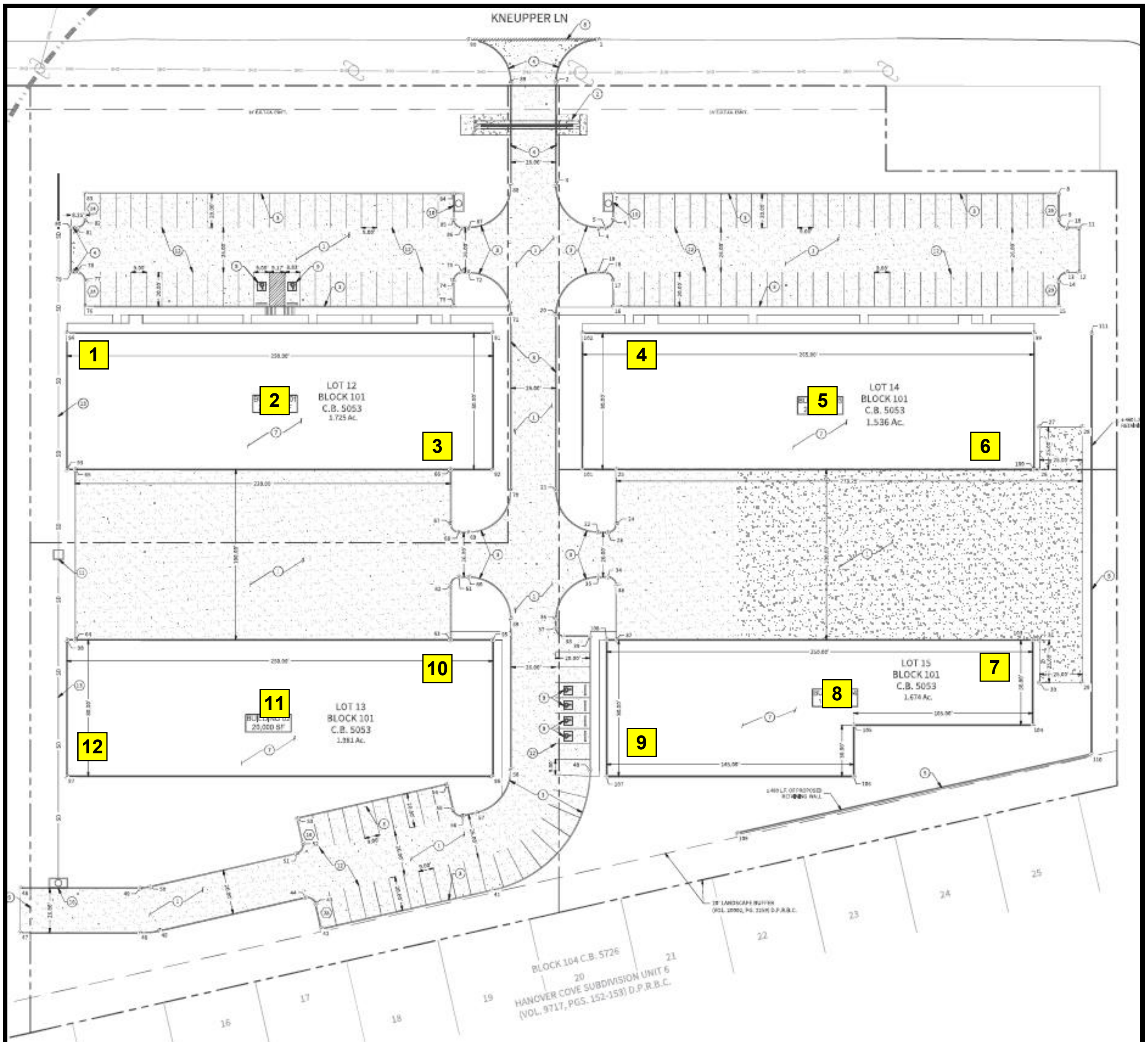
Navarro Group and Marlbrook Marl ("upper Taylor marl") undivided, Kknm. Upper part—marl, clay, sandstone, and siltstone; marl and clay, glauconitic, contain concretions of limonite and siderite; sandstone, fine grained, and siltstone, yellow brown, contain concretions of hard bluish-gray siliceous limestone 2 to 10 feet in diameter; sandstone beds have little lateral continuity and become more abundant westward; thickness up to 580 feet. Lower part—clay, dominantly montmorillonitic, unctuous, greenish gray to brownish gray; weathers to a very thick, black, clayey soil; thickness 400± feet. Total thickness 980± feet

| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Geologic Map—Approximate Location** | |
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |


Bexar County, 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 |
| HnC3—Heiden-Ferris complex, 3 to 5 percent slopes, severely eroded | | | | | | | | | | | | | | |
| Heiden, severely eroded | 80 | D | 0-14 | Clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 95-98-100 | 90-95-100 | 80-90-100 | 75-87-99 | 51-66-80 | 32-44-55 |
| | | | 14-25 | Clay, silty clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 95-98-100 | 90-95-100 | 80-90-100 | 75-87-99 | 51-66-80 | 32-44-55 |
| | | | 25-62 | Clay, silty clay | CH, CL | A-7-6 | 0- 0- 0 | 0- 0- 0 | 95-98-100 | 90-95-100 | 75-88-100 | 70-80-90 | 49-65-80 | 32-44-55 |
| | | | 62-80 | Clay | CH, CL | A-7-6 | 0- 0- 0 | 0- 0- 0 | 92-96-100 | 92-96-100 | 85-93-100 | 70-80-90 | 49-65-80 | 32-44-55 |
| Ferris, severely eroded | 15 | D | 0-8 | Clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 92-96-100 | 92-96-100 | 75-88-100 | 75-88-100 | 51-64-76 | 35-45-55 |
| | | | 8-59 | Clay, silty clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 92-96-100 | 92-96-100 | 75-88-100 | 72-86-100 | 51-65-78 | 35-46-56 |
| | | | 59-84 | Clay, silty clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 92-96-100 | 92-96-100 | 85-93-100 | 75-88-100 | 61-81-100 | 42-59-75 |
| HuC—Houston Black gravelly clay, 3 to 5 percent slopes | | | | | | | | | | | | | | |
| Houston black | 90 | D | 0-13 | Gravelly clay | GC, CH | A-7-6 | 0- 0- 0 | 0- 3- 4 | 65-69-77 | 46-57-74 | 41-55-74 | 35-48-67 | 63-70-76 | 38-44-49 |
| | | | 13-63 | Clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 98-98-99 | 96-96-98 | 85-92-98 | 74-81-89 | 63-70-71 | 38-44-49 |
| | | | 63-86 | Clay | CH | A-7-6 | 0- 0- 0 | 0- 0- 0 | 95-96-99 | 88-93-98 | 75-89-98 | 66-79-93 | 61-71-75 | 37-45-50 |

| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Soil Map—Approximate Location** | |
|---|---|---|
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |




| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | **Approximate Boring Locations** | |
| | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |

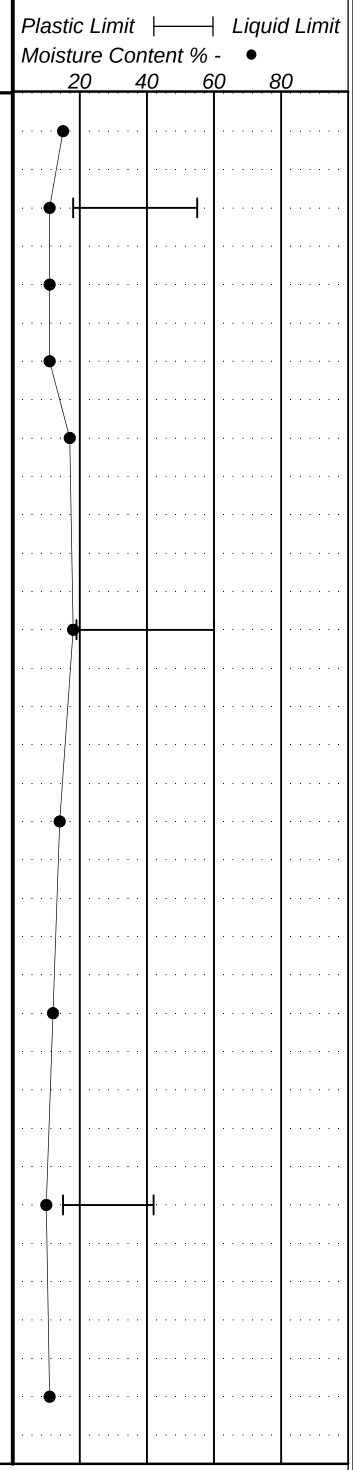
**PROJECT:** Kneupper Business Park
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** S241748
**DATE:** 10/03/2024

inTEC

# BORING NO. B-1

| 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 | Stiff Dark Brown Clay | | | | 14 | | | |
| | | SS | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay -with Gravel | | | | 20 | | 55 | 37 |
| 5 | | SS | | | | | 31 | | | |
| | | SS | | | | | 34 | | | |
| 10 | | SS | | | | | 39 | | | |
| 15 | | SS | | | | | 41 | | 60 | 41 |
| 20 | | SS | | | | | 44 | | | |
| | | | Hard Tan Silty Clay to Marl | | | | | | | |
| 25 | | SS | | | | | 80/11" | | | |
| 30 | | SS | | | | | 50/4" | | 42 | 27 |
| 35 | | SS | | | | | 50/2" | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

**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**

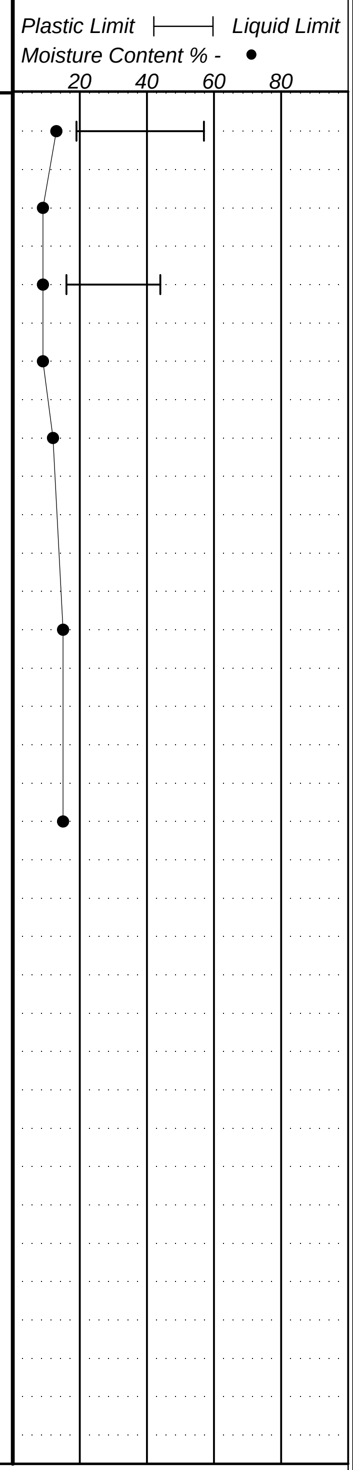
**PROJECT:** **Kneupper Business Park**
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** **S241748**
**DATE:** 10/03/2024

InTEC

## 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 | | SS | Very Stiff Dark Brown Clay | | | | 16 | | 57 | 38 |
| | | SS | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay<br>-with Gravel<br>-with Marl Seams | | | | 19 | | | |
| 5 | | SS | | | | | 21 | | 44 | 28 |
| | | SS | | | | | 29 | | | |
| 10 | | SS | | | | | 32 | | | |
| 15 | | AU | | | | | | | | |
| 20 | | AU | | | | | | | | |
| 25 | | | | | | | | | | |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 20

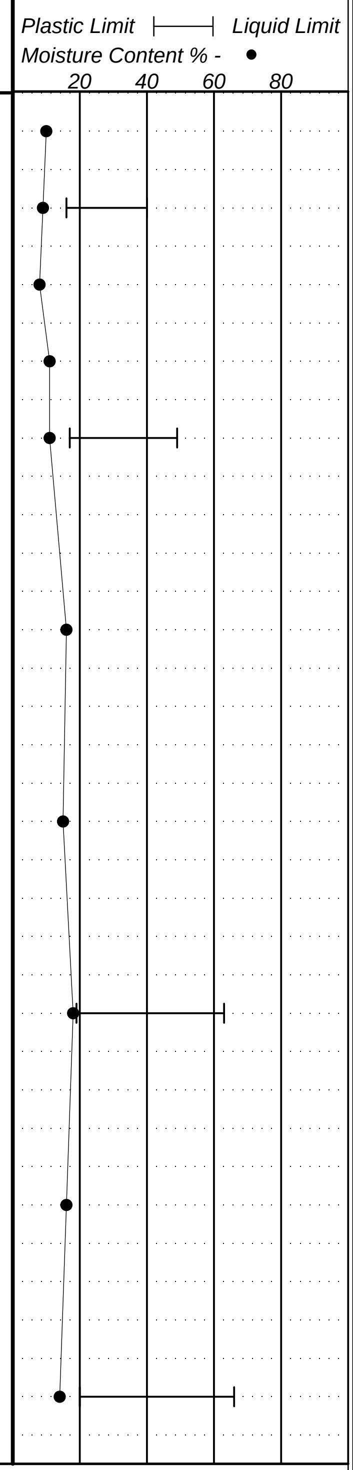
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT: Kneupper Business Park** **PROJECT NO: S241748**
**LOCATION:** Converse, Texas **DATE:** 10/03/2024
**CLIENT:** Wycoff Development & Construction, LLC

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 | | SS | Stiff Dark Brown Clay | | | | 14 | | | |
| | | SS | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay | | | | 22 | | 40 | 24 |
| 5 | | SS | | | | | 29 | | | |
| | | SS | | | | | 31 | | | |
| 10 | | SS | | | | | 26 | | 49 | 32 |
| 15 | | SS | | | | | 25 | | | |
| 20 | | SS | | | | | 24 | | | |
| 25 | | SS | | | | | 33 | | 63 | 44 |
| 30 | | SS | | | | | 46 | | | |
| 35 | | SS | | | | | 52 | | 66 | 46 |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

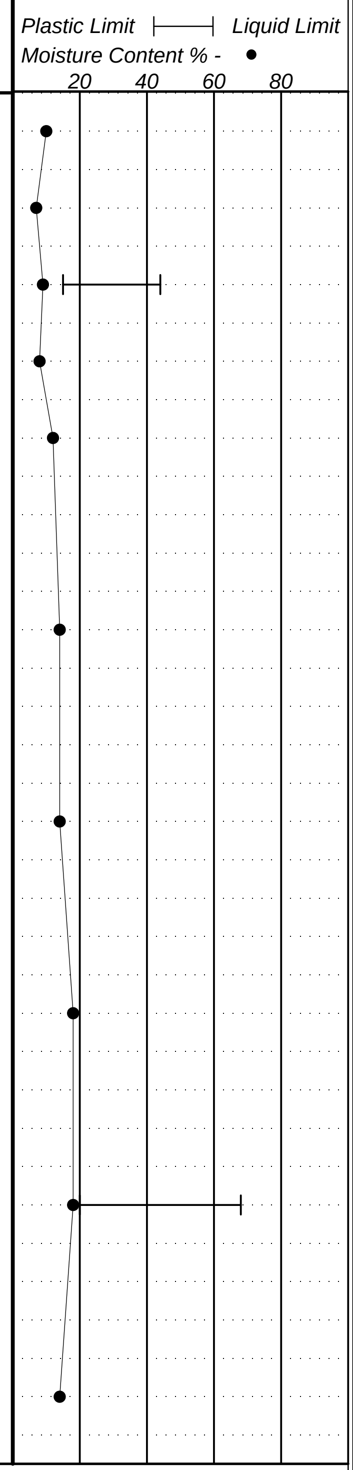
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** **Kneupper Business Park** **PROJECT NO:** **S241748**
**LOCATION:** Converse, Texas **DATE:** 10/03/2024
**CLIENT:** Wycoff Development & Construction, LLC

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 | Stiff Dark Brown Clay | | | | 12 | | | |
| | | SS | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay -with Gravel | | | | 23 | | | |
| 5 | | SS | | | | | 32 | | 44 | 29 |
| | | SS | | | | | 34 | | | |
| 10 | | SS | | | | | 35 | | | |
| 15 | | SS | | | | | 32 | | | |
| 20 | | SS | | | | | 30 | | | |
| 25 | | SS | | | | | 36 | | | |
| 30 | | SS | | | | | 43 | | 68 | 48 |
| 35 | | SS | | | | | 54 | | | |




**Notes:**

**Ground Water Observed:** No

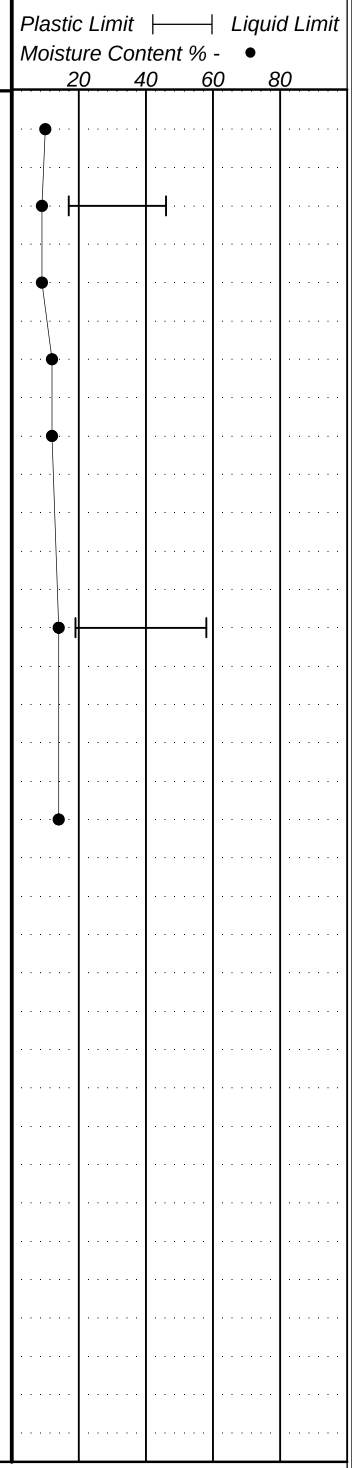
**Completion Depth (ft):** 35

**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:** **Kneupper Business Park** **PROJECT NO:** **S241748**
**LOCATION:** Converse, Texas **DATE:** 10/03/2024
**CLIENT:** Wycoff Development & Construction, LLC

**BORING NO. B-5**

| 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 | Stiff Dark Brown Clay | | | | 13 | | | |
| | | SS | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay | | | | 28 | | 46 | 29 |
| 5 | | SS | | | | | 42 | | | |
| | | SS | | | | | 38 | | | |
| 10 | | SS | | | | | 49 | | | |
| 15 | | AU | | | | | | | 58 | 39 |
| 20 | | AU | | | | | | | | |
| 25 | | | | | | | | | | |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 20

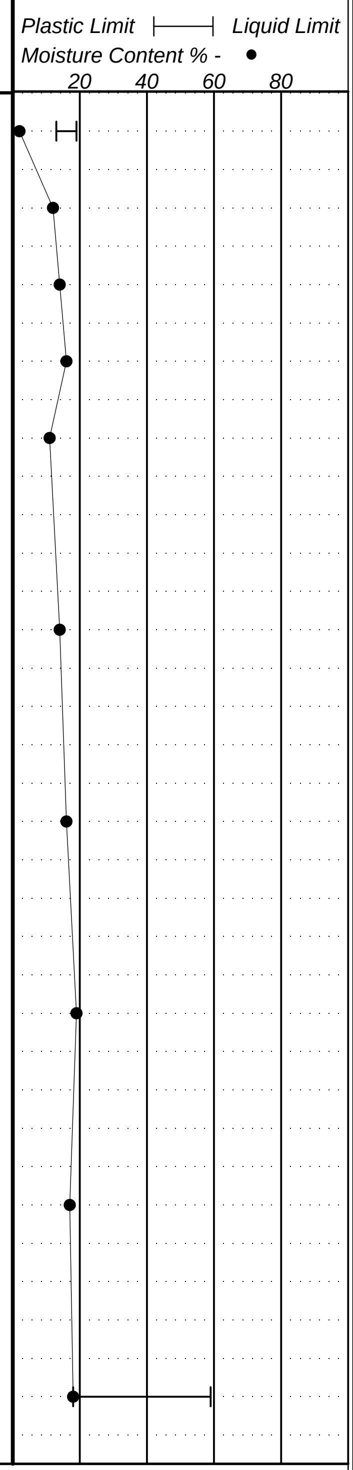
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** Kneupper Business Park
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** S241748
**DATE:** 10/03/2024

## BORING NO. B-6

| 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 | Very Stiff to Hard Tan Clay to Tan and Gray Silty Clay | | | | 20 | | 19 | 06 |
| | | SS | | | | | 18 | | | |
| 5 | | SS | | | | | 21 | | | |
| | | SS | | | | | 41 | | | |
| 10 | | SS | | | | | 48 | | | |
| 15 | | SS | | | | | 43 | | | |
| 20 | | SS | | | | | 34 | | | |
| 25 | | SS | | | | | 32 | | | |
| 30 | | SS | | | | | 44 | | | |
| 35 | | SS | | | | | 52 | | 59 | 41 |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

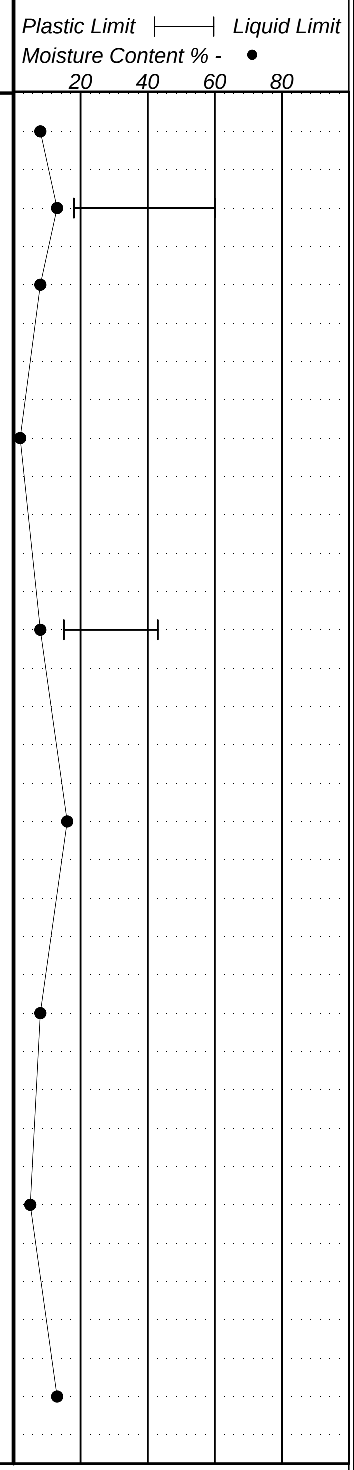
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** **Kneupper Business Park**
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** **S241748**
**DATE:** 10/03/2024

## BORING NO. B-7

| 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 | | Very Stiff to Hard Dark Brown Clay -with Gravel | | | | | | | |
| | SS | | | | | 22 | | | |
| | SS | | | | | 38 | | 60 | 42 |
| | | Brown to Tan Silty Clay -with Gravel to 10-ft | | | | | | | |
| 5 | SS | | | | | 50/2" | | | |
| | AU | | | | | | | | |
| 10 | | | | | | | | | |
| | AU | | | | | | | 43 | 28 |
| 15 | | | | | | | | | |
| | AU | | | | | | | | |
| 20 | | | | | | | | | |
| | SS | | | | | 57 | | | |
| 25 | | | | | | | | | |
| | AU | | | | | | | | |
| 30 | | | | | | | | | |
| | | -with Gray Clay Seams at 33-ft | | | | | | | |
| | SS | | | | | 50/3" | | | |
| 35 | | | | | | | | | |




**Notes:**

**Ground Water Observed:** No

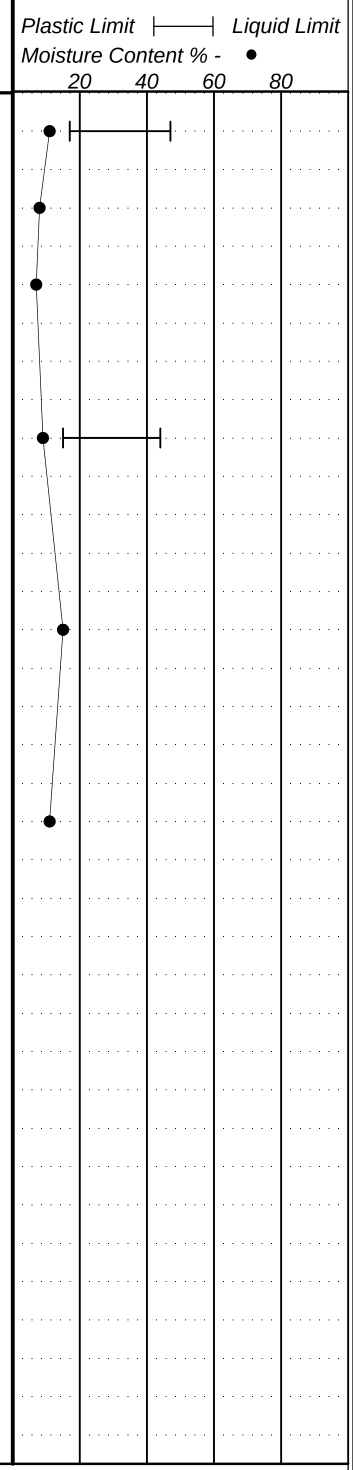
**Completion Depth (ft):** 35

**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:** **Kneupper Business Park** **PROJECT NO:** **S241748**
**LOCATION:** Converse, Texas **DATE:** 10/02/2024
**CLIENT:** Wycoff Development & Construction, LLC

## BORING NO. B-8

| 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 | Very Stiff Dark Brown Clay | | | | 15 | | 47 | 30 |
| | | SS | Hard Brown to Tan Silty Clay -with Caliche | | | | 44 | | | |
| 5 | | AU | | | | | | | | |
| | | AU | | | | | | | 44 | 29 |
| 10 | | | Hard Tan Clay | | | | | | | |
| 15 | | AU | | | | | | | | |
| 20 | | SS | | | | | 50/3" | | | |
| 25 | | | | | | | | | | |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 20

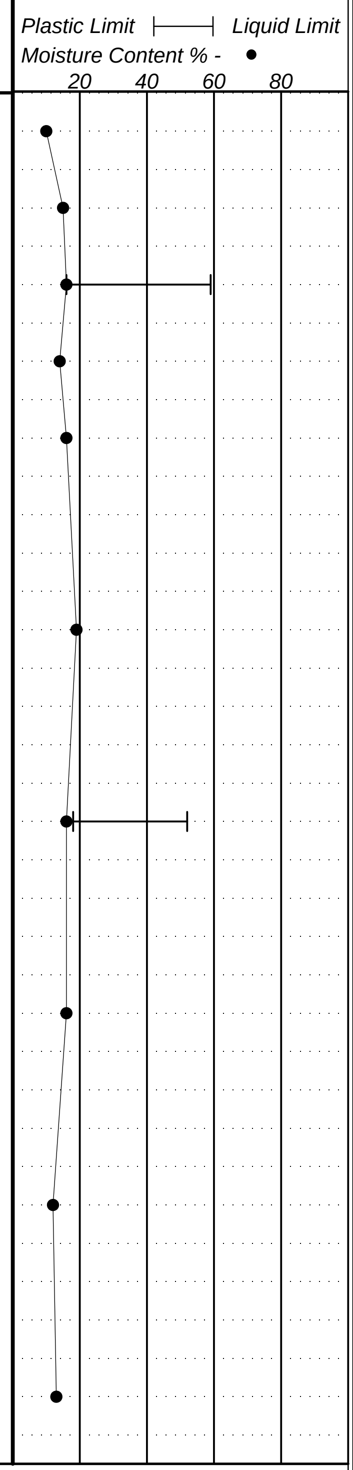
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** Kneupper Business Park **PROJECT NO:** S241748
**LOCATION:** Converse, Texas **DATE:** 10/03/2024
**CLIENT:** Wycoff Development & Construction, LLC

inTEC

## BORING NO. B-9

| 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 | Stiff to Hard Brown to Tan Clay | | | | 14 | | | |
| | | SS | | | | | 29 | | | |
| 5 | | SS | | | | | 30 | | 59 | 43 |
| | | SS | | | | | 33 | | | |
| 10 | | SS | Hard Tan Clay<br>-with Gray Clay Seams | | | | 44 | | | |
| 15 | | SS | | | | | 42 | | | |
| 20 | | SS | | | | | 46 | | 52 | 34 |
| 25 | | SS | | | | | 50 | | | |
| 30 | | SS | | | | | 46 | | | |
| 35 | | SS | | | | | 48 | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

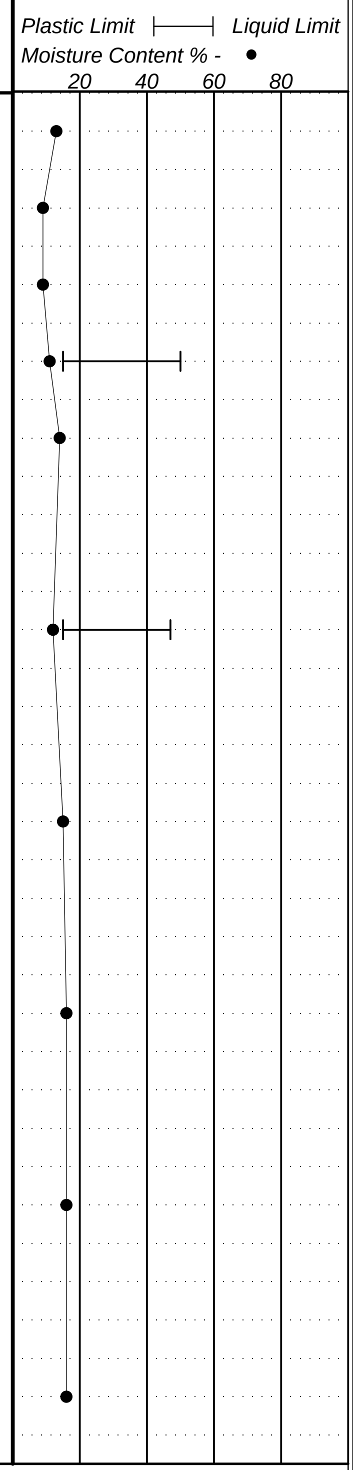
**S.S by P.P - Shear Strength in TSF**
**by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** **Kneupper Business Park**
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** **S241748**
**DATE:** 10/04/2024

**BORING NO. B-10**

| 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 | Stiff to Very Stiff Brown to Tan Silty Clay -with Gravel | | | | 10 | | | |
| | | SS | | | | | 23 | | | |
| 5 | | SS | Very Stiff to Hard Tan Clay -with Gray Clay Seams | | | | 25 | | | |
| | | SS | | | | | 29 | | 50 | 35 |
| 10 | | SS | | | | | 42 | | | |
| 15 | | SS | | | | | 46 | | 47 | 32 |
| 20 | | SS | | | | | 44 | | | |
| 25 | | SS | | | | | 33 | | | |
| 30 | | SS | | | | | 44 | | | |
| 35 | | SS | | | | | 50 | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

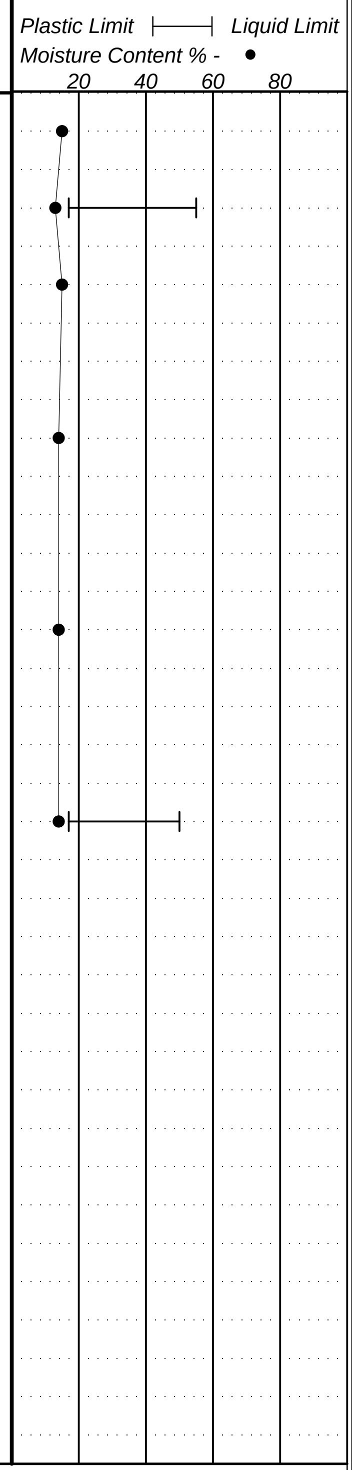
**S.S by P.P - Shear Strength in TSF by Hand Penetrometer**
**S.S. - Split Spoon Sample**
**S.T. - Shelby Tube Sample**
**HA - Hand Auger**
**AU - Auger Sample**

**PROJECT:** Kneupper Business Park
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

**PROJECT NO:** S241748
**DATE:** 10/02/2024

# BORING NO. B-11

| 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 | | | Stiff to Hard Brown to Tan Clay | | | | | | | |
| | | SS | | | | | 14 | | | |
| | | SS | | | | | 32 | | 55 | 38 |
| 5 | | SS | | | | | 50/5" | | | |
| | | | Hard Tan Clay<br>-with Gray Clay Seams | | | | | | | |
| 10 | | SS | | | | | 43 | | | |
| 15 | | SS | | | | | 50 | | | |
| 20 | | SS | | | | | 50 | | 50 | 33 |
| 25 | | | | | | | | | | |
| 30 | | | | | | | | | | |
| 35 | | | | | | | | | | |




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 20

**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:** **Kneupper Business Park**
**LOCATION:** Converse, Texas
**CLIENT:** Wycoff Development & Construction, LLC

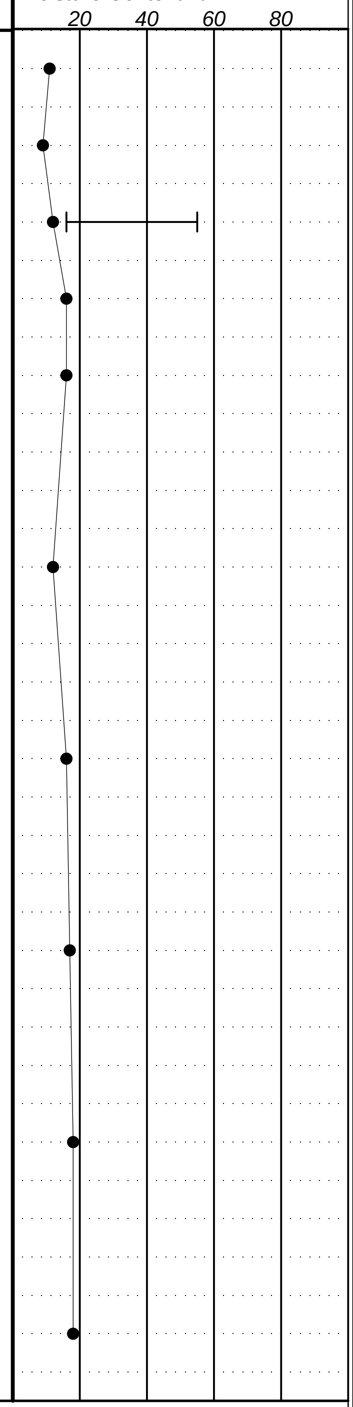
**PROJECT NO:** **S241748**
**DATE:** 10/01/2024

intEC

## BORING NO. B-12

| 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 | | Stiff to Very Stiff Brown to Tan Silty Clay | | | | | | | |
| | SS | | | | | 13 | | | |
| | SS | | | | | 24 | | | |
| | | Hard Tan Clay | | | | | | | |
| 5 | SS | | | | | 32 | | 55 | 39 |
| | SS | | | | | 48 | | | |
| | SS | | | | | 54 | | | |
| 10 | | | | | | | | | |
| | SS | | | | | 47 | | | |
| 15 | | | | | | | | | |
| | SS | | | | | 38 | | | |
| 20 | | | | | | | | | |
| | SS | | | | | 45 | | | |
| 25 | | | | | | | | | |
| | SS | | | | | 35 | | | |
| 30 | | | | | | | | | |
| | | -with Gypsum at 33-ft | | | | | | | |
| | AU | | | | | | | | |
| 35 | | | | | | | | | |

*Plastic Limit* ├──┤ *Liquid Limit*
*Moisture Content % -* ●




**Notes:**

**Ground Water Observed:** No

**Completion Depth (ft):** 35

**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**

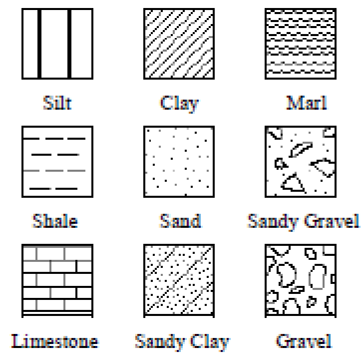
## KEY TO CLASSIFICATIONS AND SYMBOLS

### 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. |

| | | |
|---|---|---|
| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |

# Appendix

| Subsurface Exploration and Foundation Analysis<br>Proposed New Kneupper Business Park<br>Kneupper Lane & Steves Avenue<br>Converse, Texas | InTEC Project Number:<br>**S241748** | Date:<br>09/11/2024 |
|---|---|---|

# 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 not 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 Not 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 not 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 not building-envelope or mold specialists.***

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