GEOTECHNICAL INVESTIGATION

TRINITY LAKES PARK AND RIDE
HURST, TEXAS

AGG REPORT: DE19-305R1

JANUARY 30, 2020

PREPARED FOR:

TRANSYSTEMS
FORT WORTH, TEXAS

PRESENTED BY:

ALLIANCE
GEOTECHNICAL
GROUP

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ALLIANCE GEOTECHNICAL GROUP

- Geotechnical Engineering
- Environmental Consulting
- Construction Materials Engineering and Testing
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January 30, 2020

Mr. Chad G. Gartner, P.E.
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E-mail: cggartner@transystems.com
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Re: Geotechnical Investigation
Trinity Lakes Park and Ride
Hurst, Texas
AGG Report No. DE19-305r1

Dear Mr. Gartner:

Please find enclosed our report summarizing the results of the geotechnical investigation performed at the above referenced project. We trust the recommendations derived from this investigation will provide you with the information necessary to complete your proposed project successfully.

For your future construction materials testing and related quality control requirements, it is recommended that the work be performed by Alliance Geotechnical Group, Inc. in order to maintain continuity of inspection and testing services for the project under the direction of the Geotechnical Project Engineer.

We thank you for the opportunity to provide you with our professional services. If we can be of further assistance, please do not hesitate to contact us.

Sincerely,
ALLIANCE GEOTECHNICAL GROUP, INC.

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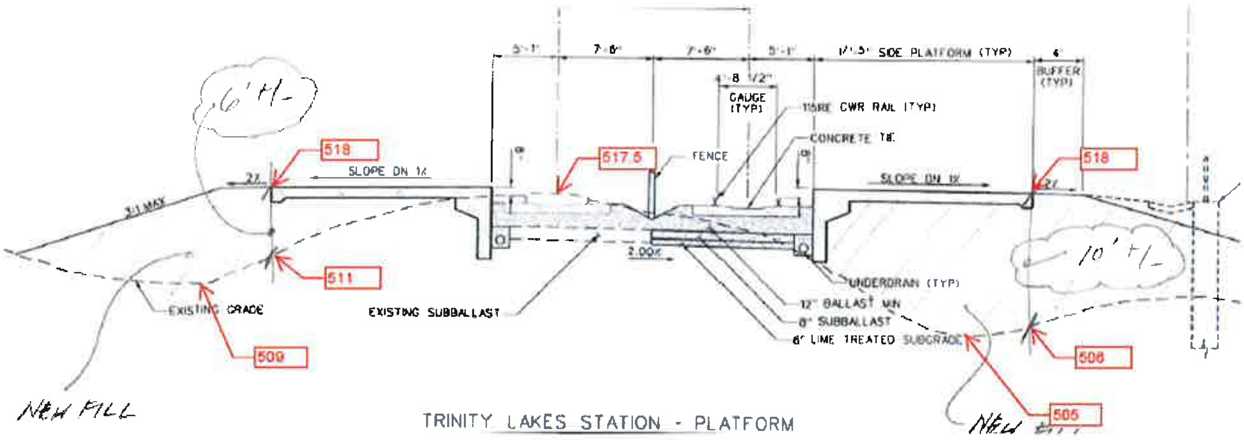
HURST, TEXAS

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The project consists of proposed new train platforms for the new Trinity Lakes Park and Ride. It is understood that the proposed new platforms will be located near the southeast corner of Loop 820 and W. Hurst Boulevard in Hurst, Texas. The platforms will be constructed on both sides of the railroad tracks.

Significant amounts of fill (up to 13 feet) will be required at the platforms. Current plans consist of constructing 3H:1V to 4H:1V slopes beyond these platforms. A typical section of the proposed new improvements is shown below.



1.2 PURPOSE AND SCOPE

The purposes of this geotechnical investigation were to: 1) explore the subsurface conditions at the site, 2) evaluate the pertinent engineering properties of the subsurface materials, 3) provide foundation recommendations for the proposed new platform structures, 4) provide recommendations for embankment construction, and 5) provide comments and recommendations for site grading and drainage. This report was prepared in general accordance with AGG's Proposal P19-0306E dated March 12, 2019.

2.0 FIELD INVESTIGATION

The field investigation consisted of drilling six (6) test borings (Borings B-1, B-2, B-3, B-3B, B-5 and B-6) across the project site. Three (3) test borings (Borings B-1 thru B-3) were located within the vicinity of the proposed new platform structures and were advanced to depths ranging from 50 to 65 feet below the existing ground surface. Two (2) test borings (Borings B-5 and B-6) were located in areas of proposed track improvements. These test borings were advanced to depths of 25 feet below the existing ground surface. One (1) test boring (Boring B-3B) was located on top of the existing railroad embankment in the vicinity of the proposed new railroad tracks adjacent to the proposed south platform.

Note: A forth structural test boring (Boring B-4) was originally planned but was unable to be drilled due to site access issues.

A truck-mounted auger drill rig was used to advance all of the test borings except for Boring B-3B. This boring was advanced using hand augering techniques. The borings were located at the approximate locations shown on the Plan of Borings (Figure 1).

Undisturbed samples of cohesive soils were obtained at intermittent intervals with standard, thin-walled, seamless tube samplers. These samples were extruded in the field, logged, sealed, and packaged to protect them from disturbance and maintain their in-situ moisture content during transportation to our laboratory.

Foundation bearing properties of the granular soils encountered in the borings were evaluated by the Standard Penetration test in conjunction with split spoon sampling. The Standard Penetration tests involves driving a standard 2 inch diameter sampler a total of one foot and recording the blow counts and driving distances for each 6 inch or 50 blow increment. The first 6 inch drive is for seating purposes. The results of the Standard Penetration tests are recorded at the respective testing depths on the Logs of Borings.

The rock encountered in the test borings was evaluated by the Texas Department of Transportation Penetrometer (TxDOT Cone) tests. The TxDOT Cone is driven with the resulting penetration in inches recorded for 100 blows. The results of the TxDOT Cone test are recorded at the respective testing depths on the Logs of Borings.

The results of the boring program are presented on the Logs of Borings, Figures 2 thru 10. A key to the descriptive terms and symbols used on the logs is presented on Figures 11 & 12.

3.0 LABORATORY TESTING

Laboratory tests were performed on representative samples of the soil to aid in classification of the soil materials. These tests included Atterberg limits tests, percent passing #200 sieve, moisture content tests and unit weight determinations. Hand penetrometer tests were performed on the clay soil samples to provide indications of the swell potential and the foundation bearing properties of the subsurface strata. An unconfined compressive strength test was performed on a selected soil sample in order to determine the foundation bearing properties. The results of our testing program are presented on the Logs of Boring (Figures 2 thru 10).

To provide additional information about the swell characteristics of these soils (at their in-situ moisture conditions), absorption swell tests were performed on selected samples of the clay soils. Results of the swell testing are provide on Figure 13.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 GENERAL SITE CONDITIONS

The project site is located within the southeast corner of Loop 820 and W. Hurst Boulevard in Hurst, Texas. The subject area is along the existing Trinity Railway Express (TRE) railroad track. The existing track is elevated above the adjacent grades by about 10 to 12 feet. The north side of the existing tracks is currently heavily wooded. See Plan of Borings (Figure 1) for site configuration, location and aerial view.

4.2 SITE GEOLOGY

As shown on the Dallas Sheet of the Geologic Atlas of Texas, the site is located in an area underlain by alluvial overburden soils underlain by the Grayson Marl & Main Street Limestone Formation. The alluvial overburden soils at this site consist of clays, silts, sands and gravel. The Grayson Marl & Main Street Limestone Formation typically consists of very hard limestone, calcareous clay, marl and calcareous shale. Soils derived from this formation are typically highly plastic clays exhibiting a high shrink/swell potential with variations in moisture content.

4.3 SIESMIC SITE CLASS

The International Building Code (IBC) was reviewed to determine the seismic site class of the subject property. In accordance to Table 1615.1.1 of the IBC, the subject site has a seismic site class of C for a very dense soil/soft rock profile.

4.4 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the borings, including descriptions of the various strata, their depths, and thicknesses, are presented on the Logs of Borings. Note that depth on all borings refers to the depth from the existing grade or ground surface present at the time of the investigation. Boundaries between the various soil types are approximate.

4.5 GROUNDWATER CONDITIONS

The borings were advanced using continuous flight auger methods. Advancement of the borings using these methods allows observation of the initial zones of seepage. Groundwater was generally encountered within the test borings at depths ranging from 8 to 20 feet below the existing ground surface. Shallower groundwater levels should be anticipated in all areas if construction occurs after periods of rain.

It is not possible to accurately predict the magnitude of subsurface water fluctuations that might occur based upon short-term observations. The subsurface water conditions are subject to change with variations in climatic conditions and are functions of subsurface soil conditions, and rainfall.

5.0 ANALYSES AND RECOMMENDATIONS

5.1 SOIL MOVEMENT

The subsurface exploration within the test borings revealed the presence of very dry and expansive sandy clay soils over sands. These sandy clay soils have high expansive potential due to the very dry moisture conditions due to tree root desiccation. The potential upward soil swell movements were performed using swell test results, pocket penetrometer readings, and moisture content tests to estimate the swell potential of the soil. The potential upward soil swell movements based upon the current very dry soil conditions and current grades have been estimated to range from 2 to 6 inches.

5.2 STRAIGHT SHAFT FOUNDATION SYSTEM

The proposed new platform structures could be supported by cased straight sided continuously reinforced shaft piers founded in the very hard gray unweathered limestone and very firm (relatively soft) to hard dark gray unweathered shale. Due to the presence of water bearing granular soils above the bearing stratum, all piers will have to be cased. The very hard gray unweathered limestone bearing stratum was first encountered at depths ranging from 26 to 28 feet below the existing ground surface at the platform test boring locations (Borings B-1 thru B-3). The depth and approximate elevation to the very hard gray unweathered limestone at each boring location is summarized below.

Boring Number	Depth to Very Hard Gray Unweathered Limestone	Elev. to Very Hard Gray Unweathered Limestone
B-1	26 feet	482.0 feet
B-2	35 feet	477.5 feet
B-3	28 feet	482.0 feet

The allowable end bearing pressure and side resistance pressures are provided in Table 1 and have been developed based on the assumption that a minimum 2 pier diameter clear spacing will be provided between piers. For piers touching, a 50% reduction in skin friction should be used. Where the clear spacing is 2D, no reduction is necessary. For spacing between 0 and 2D, a straight line interpolation should be used.

The skin friction values provided are for compression loading and for resistance to soil swell uplift. For other tension loads (sustained uplift), the allowable skin friction is 50% of the value indicated above. These foundations should be subject to settlements of less than 3/4 inch. Differential settlements should be limited to about one-half inch.

Note: The allowable bearing values within the upper 16 feet are due to the predominance of relatively soft dark gray shale to depths of 43 to 48 feet below existing grade (to elevations ranging from 462 to 465 feet).

TABLE 1. ALLOWABLE BEARING VALUES

SHAFT LOADING TYPE	BEARING STRATA	
	VERY HARD GRAY UNWEATHERED LIMESTONE AND VERY FIRM TO HARD DARK GRAY UNWEATHERED SHALE	
Axial End Bearing	UPPER 20 FEET OF PENETRATION	PENETRATION GREATER THAN 20 FEET
	12,000 psf **	50,000 psf
Skin Friction Side Resistance	2,000 psf *	8,000 psf*

* For all penetrations below temporary casing into very hard gray unweathered limestone and very firm to hard dark gray unweathered shale as verified by the AGG geotechnical team. The skin friction values provided are for compression loading and for resistance to soil swell uplift. For other tension loads (sustained uplift), the allowable skin friction is 50% of the value indicated above.

**A minimum penetration of 9 feet or 3 pier diameters into very hard gray unweathered limestone and very firm to hard dark gray unweathered shale bearing stratum (whichever is greater) is recommended to develop the allowable end bearing pressure. Larger penetrations may be required for soil swell uplift and for axial loading. Penetrations into gray weathered limestone / shale (identified by iron staining or tan colored seams) should not be counted on for the design penetrations during pier installations. The design penetrations should be counted on only for penetrations into continuous very hard gray unweathered limestone and very firm to hard dark gray unweathered shale below temporary casings.

Note 1: We recommend that an AGG Geotechnical Engineer be present at the start of the drill pier operations in order to identify the proper bearing stratum to field personnel.

Note 2: A minimum shaft diameter of 18 inches should be used for the straight shaft piers. In addition, we recommend that a maximum length to diameter ratio of 30 be used for design of the drilled shafts.

Note 3: In lieu of using cased straight shaft piers, auger cast piles could be considered to support the proposed platform structures. If auger cast piles are used, the auger cast piles should be designed by the Structural Engineer and/or piling specialist to penetrate into the bearing stratum. The allowable bearing values provided in Table 1 should be used for design of the auger cast piles. The auger cast piles should be constructed in general accordance with the recommendations within this report for straight shaft piers.

Note 4: We recommend that the first three (3) auger cast piles and at least 10% of the remaining auger cast piles thereafter be integrity tested. We recommend that the integrity testing be performed using sonic echo testing methods. These tests will assist in the evaluation of the integrity of the auger cast piles. We recommend that AGG be retained to perform the sonic echo testing on the auger cast piles.

5.2.1 ALLOWABLE LATERAL RESISTANCE

We understand that the proposed structures will possibly be subject to lateral loads. L-Pile parameters recommended for design are provided below. See group reductions below.

Note 1: L-Pile parameters for each soil and rock strata are provided below for individual laterally loaded drilled shafts. The parameters indicated below do not include reductions related to group effects (see below).

Neglect passive resistance within upper 5 feet below top of pier.

<table>
<tr><th colspan="6">FOR EXISTING FILL SOILS AND FILLS REQUIRED TO RAISE GRADES (below 5 feet)</th></tr>
<tr><th>Water Level Condition</th><th>Material Type for LPile</th><th>Undrained Cohesion (psf)</th><th>Total Soil Unit Weight (pcf)</th><th>Strain Factor E_{50}</th><th>Static Horizontal Modulus of Subgrade Reaction, k (pci)</th></tr>
<tr><td>Above Water</td><td>Soft Clay without Free Water</td><td>500</td><td>120.0</td><td>0.02</td><td>30</td></tr>
<tr><td>Below Water</td><td>Soft Clay with Free Water</td><td>500</td><td>58.0</td><td>0.02</td><td>30</td></tr>
<tr><th colspan="6">FOR NATIVE SANDY CLAY (above elevation 500 feet)</th></tr>
<tr><td>Below Water</td><td>Stiff Clay with Free Water</td><td>2,000</td><td>58.0</td><td>0.007</td><td>200</td></tr>
<tr><th colspan="6">FOR NATIVE SANDY CLAY AND SILTY CLAY (below elevation 500 feet)</th></tr>
<tr><td>Below Water</td><td>Stiff Clay with Free Water</td><td>750</td><td>58.0</td><td>0.01</td><td>100</td></tr>
</table>

<table>
<tr><th colspan="5">FOR NATIVE SANDS, SILTY SANDS AND CLAYEY SANDS</th></tr>
<tr><th>Water Level Condition</th><th>Material Type for LPile</th><th>Effective Soil Unit Weight (pcf)</th><th>Friction Angle</th><th>Static Horizontal Modulus of Subgrade Reaction, k (pci)</th></tr>
<tr><td>Below Water</td><td>Medium Sand with Free Water</td><td>63.0</td><td>28</td><td>20</td></tr>
</table>

Design Parameter	Design Values Upper 20 feet of Penetration into Very Hard Gray Unweathered Limestone and Very Firm to Hard Gray Unweathered Marl
Material Type	Weak Rock
Effective Unit Weight, pcf	65.0 below water
Young's Modulus, psi	700
Uniaxial Compressive Strength (PSF)	12,000
Rock Quality Index (RQD, %)	70
Stiffness Constant (Km)	0.0005

Design Parameter	Design Values Penetrations Greater than 20 feet into Very Hard Gray Unweathered Limestone
Material Type	Weak Rock
Effective Unit Weight, pcf	65.0 below water
Young's Modulus, psi	40,000
Uniaxial Compressive Strength (PSF)	50,000
Rock Quality Index (RQD, %)	90
Stiffness Constant (Km)	0.0005

Note 2: Reduction factors must be applied to account for group effects for laterally loaded piers. If the center to center spacing is less than 7D for laterally loaded piers, then an L-Pile study using the Ensoft Group Pile Program would be required to determine the appropriate reduction factors that must be applied for laterally loaded piers. This includes group effects in the direction of loading, side by side effects, and effects in skewed directions. Alliance Geotechnical Group could be retained to work with the structural engineer in performing the Group Pile Analyses. Otherwise, the Structural Engineer could use the information in the below paragraphs to conservatively approximate reductions for group effects.

In order to determine the appropriate reduction for group effects of a pier in a group, the reduction for side by side effects (R_{SS}), the reduction for in-line loading effects (R_{IL}), and the reduction for skewed effects (R_{SK}) will need to be determined for each adjacent pier and multiplied together to determine the reduction factor for the subject pier in the group. The reduction for in-line loading effects (R_{IL}) will consist of either leading pier (R_{LP}) or trailing pier (R_{TP}). Each of these reductions should be determined as recommended below.

Reduction due to Side by Side effects(R_{SS})

For 1D center to center spacing (piers touching), 64% of the lateral resistance should be used. Where the center to center spacing is 4D, no reduction is necessary. For a spacing between 1D and 4D, a straight line interpolation should be used.

Reduction due to Leading Pier (R_{LP})

For 1D center to center spacing (piers touching), 70% of the lateral resistance should be used. Where the center to center spacing is 4D, no reduction is necessary. For a spacing between 1D and 4D, a straight line interpolation should be used.

Reduction due to Trailing Pier (R_{TP})

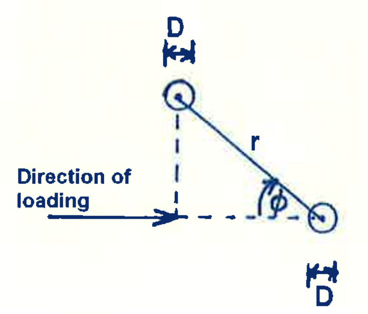
For 1D center to center spacing (piers touching), 47% of the lateral resistance should be used. Where the center to center spacing is 4D, 80% of the lateral resistance should be used. For a spacing between 1D and 4D, a straight line interpolation should be used between the 47% to 80% of lateral resistance. Where the center to center spacing is 7D, no reduction is necessary. For a spacing between 4D and 7D, a straight line interpolation should be used between the 80% and 100% of lateral resistance.

Reduction due to Skewed Pier (R_{SK})

The reduction factor for skewed piers (piers neither side-by-side or in line with the direction of loading) should be determined by the below formula.

$$R_{SK} = (R_{IL}^2 \cos^2 \varnothing + R_{SS}^2 \sin^2 \varnothing)^{1/2}$$

The reduction for in-line loading effects (R_{IL}) will consist of either leading pier (R_{LP}) or trailing pier (R_{TP}) whichever one is applicable to the loading direction. The center to center spacing is based upon r/D and Ø (where Ø is the angle between the direction of loading and the line between the piers as shown in the detail below).



Note 3: For all of the above reduction factors, if different size piers are used in a group, D used in the above formulas should be the largest pier diameter in the group.

5.2.2 DRILLED SHAFT SOIL INDUCED UPLIFT LOADS

All piers will be subject to uplift loads as a result of swelling within the overlying clays. Straight shafts should be designed by the Structural Engineer with adequate penetration lengths in order to have sufficient anchorage in resisting uplift forces generated by soil swelling. The piers should have sufficient continuous vertical reinforcing steel extending to the bottom of the piers to resist the computed net uplift loads (uplift less dead load).

The magnitude of the uplift loads varies with the shaft diameter, soil parameters, free water sources, and the depth of the active clays acting on the shaft. The uplift pressures can be approximated at this site by assuming a uniform uplift pressure of 1,500 pounds per square foot acting on the shaft perimeter for a depth of 12 feet assuming that over-excavation and moisture conditioning is performed to reduce the potential soil swell movements to one inch. Where moisture conditioning is not performed, the uplift pressure should be increased to 2,500 psf.

5.2.3 DRILLED SHAFT CONSTRUCTION CONSIDERATIONS

Groundwater was encountered within the platform test borings at depths ranging from 8 to 13 feet below existing grade. In addition, thick layers of caving water bearing granular soils are present above the bearing stratum. Therefore, all piers will have to be cased. Temporary casing should be properly seated and sealed within the bearing stratum to prevent seepage into the drilled shaft excavation. Care must be taken that a sufficient head of plastic concrete is maintained within the casing during extraction.

Concrete used for the shafts should have a slump of 6 to 8 inches and placed in a manner to avoid striking the reinforcing steel and walls of the shaft during placement. Complete installation of individual shafts should be accomplished within an 8-hour period in order to help prevent deterioration of bearing surfaces. The drilling of individual shafts should be excavated in a continuous operation and concrete placed as soon as practical after completion of the drilling. No shaft should be left open for more than 8 hours.

We recommend that Alliance Geotechnical Group be retained to observe and document the drilled pier construction. The engineer, or his representative, should document the shaft diameter, penetration, depth, casing installations and extractions, cleanliness, plumbness of the shaft, and the type of bearing material. Significant deviations from the specified or anticipated conditions should be reported to the owner's representative and to the structural engineer. The drilled pier excavation should be observed to verify the bottom of the excavation is dry and thoroughly cleaned of cuttings after completion.

Note: "Mushrooming" should not be allowed around piers, pier caps or grade beams.

5.2.4 GRADE BEAMS

Grade beams supported by straight shaft piers should be constructed over a void space. A minimum void space of 4 inches should be provided between the bottom of these members and the subgrade. This is assuming site modification work is performed to reduce the potential soil swell movements to one inch in preparation for slab-on-grade construction. If structural floors are used, a minimum 12 inch void space should be specified in these areas. Permanent retainer forms should be used. Structural cardboard forms are one acceptable means of providing this void beneath these members. Care must be exercised during concrete placement to avoid collapsing the cardboard void boxes. The cardboard carton forms should not be allowed to become wet or crushed prior to concrete placement. Permanent earth retainer forms should be used.

The exterior portions of the grade beams along the perimeter of the buildings should be carefully backfilled with on–site clayey soils unless specified otherwise below. The backfill soils should be placed at a moisture content between +1% and +4% above optimum. The fill should be compacted to 95 percent of maximum dry density as determined in accordance with ASTM D–698 (Standard Proctor).

Note: Prior to backfilling leave out areas along grade beams, it must be verified by AGG that <u>all</u> clay soil that has become dry along the leave-out areas during construction is removed and moisture conditioned prior to backfilling. Full depth select fill backfill is recommended along the interior of the grade beams.

5.3 PLATFORM FLOOR SLABS

The subsurface exploration within the test borings revealed the presence of very dry and expansive sandy clay over sands. The potential upward soil swell movements based upon the current very dry soil conditions and current grades have been estimated to range from 2 to 6 inches.

Due to the potential for slab movements, the floor slab either should be structurally supported and suspended above the site soils by a void space or should consist of a slab-on-grade that is placed on a low PI / flex base cap over moisture conditioned and compacted fill soils. We understand that the proposed north platform will be located relatively close to the existing tracks. Since deep excavation and moisture conditioning of the site soils cannot be performed unless an elaborate shoring system is used, we understand that a structurally supported floor slab will be used for the north platform. We understand that both structurally supported floor slab and slab-on-grade on moisture conditioned clay soils will be considered for the south platform.

5.3.1 STRUCTURALLY SUPPORTED FLOOR SLAB

Due to the potential for excessive slab movements, one method of minimizing the effects of soil movements in areas sensitive to movement would be the use of structurally supported floor slabs suspended above the underlying soils by a crawl space or a minimum twelve (12) inch void space with permanent earth retainer forms. The void space created between the bottom of the floor slab and the subgrade will serve to reduce distress resulting from swell pressures generated by the clays. If a structural floor is used, the void space should be drained. Any crawl space should be ventilated and drained. A suitable vapor barrier should be used below all floor slabs.

Note 1: A crawl space would be preferred and is recommended to allow access for maintenance. This would allow utility lines to be hung to the floor system and suspended above the expansive clays by at least 12 inches. Otherwise, soil swell movements beneath utility lines would cause leaks and crush the pipe at connections to floor fixtures unless appropriate details are used to prevent upward pipe movement (such as Plumbing Void Systems). All utility penetrations through or beneath grade beams or through floor slabs should be sleeved appropriately to accommodate large differential soil swell movement.

Note 2: Large differential upward piping movements should be anticipated where sub-floor piping transitions into highly expansive soil at the building perimeter. Flexible piping connections should be used at the building perimeter to accommodate large differential upward movements.

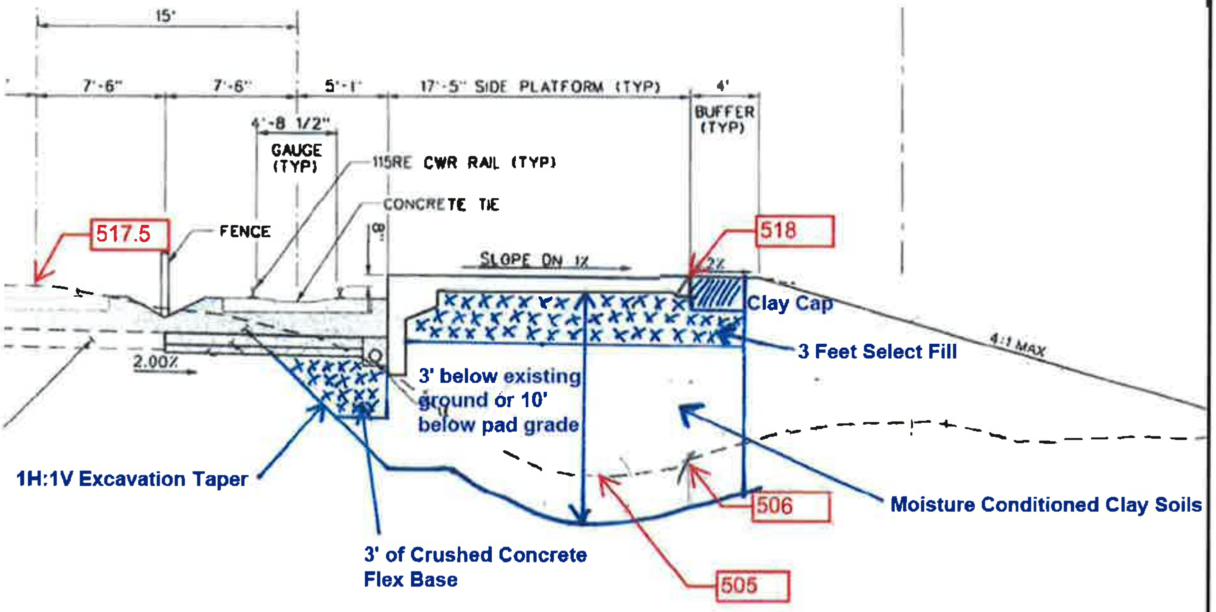
Note 3: Concrete cut-off collars should be used for below grade piping adjacent to the building perimeter to minimize water seepage into the crawl space via utility trenches.

5.3.2 SLAB-ON-GRADE FLOOR SLAB

A slab-on-grade floor system may be used for the south platform. Site preparation work will be required in order to lower the potential soil movements for the proposed south platform building. Recommendations for site preparation work to reduce the anticipated potential soil swell movements to on the order of one (1) inch are provided below.

1. Excavate to a depth of 3 feet below the existing ground surface or 10 feet below final pad grade, whichever is deeper. Wherever existing fill soils are present at base of cut, test pits and density testing will be required to verify adequate compaction (minimum 98% of ASTM D698) of these deeper existing fill soils as verified by AGG. If compaction levels are less than 98% of ASTM D698, the non-compact existing fill soils should be over-excavated and replaced in compacted lifts as specified below.

   Excavations should extend 3 feet beyond building lines and 3 feet beyond adjacent sidewalks and 3 feet beyond the proposed adjacent new track, whichever is greater. However, TRE will not allow excavations within 12 feet of the existing tracks. Therefore, over-excavation and moisture conditioning is limited at the proposed new tracks and only a portion of the tracks can be moisture conditioned. In order to maintain a 12 foot clear distance from the existing tracks, the excavation limits on the north side of the platform should extend to the platform edge and then be tapered upward on a 1H:1V excavation taper. A schematic detail of the required excavation and moisture conditioning is provided below.



   The excavation limits should extend at least 20 feet beyond building lines and then transitioned on 2H:1V slope at track entrances into the platform. These excavation extensions will be limited as described above and as shown in the above schematic.

We recommend that Alliance Geotechnical Group review the excavation plan for compliance with this report prior to construction bidding.

2. After excavation is approved by AGG, the upper 10 inches at the base of cut should be scarified, moisture conditioned and compacted as specified below in Item 3 below.

3. Fill to within 3 feet of final pad grade using on-site sandy clay, very sandy clay and clayey sand having a PI between 4 and 25. Clay soils having a PI in excess of 25 and silty sands having a PI of less than 4 should not be used as fill beneath the platform.

   Compact in maximum 8 inch lifts as specified below.

For clayey sand with PI between 4 and 15:	Compact at -2% to +2% of optimum to minimum of 98% of ASTM D 698.
For sandy clay with PI between 16 and 25:	Compact at +1% to +3% of optimum to minimum of 95% of ASTM D 698.

Note 1: Sandy clay fill placed below 10 feet of final pad grade should be compacted at optimum to +2% of optimum to minimum of 98% of ASTM D698.

Note 2: The moisture conditioned clay soils and select fill / crushed concrete flex base should be benched into the existing embankment and compacted in maximum 8 inch horizontal benched lifts. It is imperative that the interface between the existing embankment and the new fill soils be adequately compacted in horizontal benched lifts along the interface to eliminate any loose seam.

Note 3: The "targeted" compaction moisture contents are subject to the following QA/QC verifications by AGG.

Hand Penetrometer:	3.0 to 4.5 tsf (clayey sand) 2.5 to 3.75 tsf (sandy clay)
Soil Swell:	1% under select fill surcharge.

4. Fill to final pad grade with a minimum of 3 feet of low PI select fill and/or crushed concrete base. Crushed concrete base should be used for the entire 3 feet cap below the train tracks (to limits of 3 feet beyond the track edges). A minimum of 3 feet of crushed concrete base should be placed between the railroad ballast and the moisture conditioned clay soils. In non-train areas, the upper 8 inches of select fill should consist of crushed concrete base as a minimum to improve the subgrade support of the floor slab.

   The select fill should extend to the excavation limits (see Item 1 above). The material used as select fill should be a very sandy clay to clayey sand (uniform consistency free of clay clods) with a plasticity index between 5 and 14. The fill should be spread in loose lifts, less than 8 inches thick, and uniformly compacted to a minimum of 98 percent of ASTM Standard D 698 between –2 and +2 percentage points of the soil's optimum moisture content.

The crushed concrete should comply with TxDOT Item 247, Type D, Grade I and should be compacted in maximum 6 inch lifts at optimum to +2% above optimum to a minimum of 95% ASTM D 1557 (modified Proctor).

5. The upper 24 inches of fill in unpaved areas near the building should consist of compacted on–site clay to minimize water infiltration into the select fill (compact in 8 inch lifts to 95% ASTM D 698) at +1 to +4 above optimum.
6. The moisture condition of completed pad must be maintained until all slabs are in place as verified by AGG. This will be particularly important along the building perimeter where clay soils will be exposed after excavations for grade beams or tilt wall panels. The use of 6 inches of select fill above exposed clay excavations would reduce moisture losses during prolonged dry weather conditions.

Note 4: The moisture condition of the soils exposed along the perimeter must be verified by AGG prior to backfilling of grade beams. All backfill placed along the interior edge of grade beams should consist of select fill. Otherwise, large upward soil swell movements would occur.

Note 5: Anticipated settlements should be on the order of 0.5% to 1% of the total fill height.

Note 6: The above recommendations are to reduce the anticipated potential soil swell movements to on the order of one (1) inch for the platform structure. However, the soil swell movements could be higher on the north end of the proposed tracks and within the tapered excavation portion beneath the tracks. A hand auger test boring (Boring B-3B) was drilled on top of the existing railroad embankment in the vicinity of the proposed new tracks. Hand auger refusal occurred at a depth of 6 feet. The embankment fill soils encountered within the upper 6 feet consisted of a two foot layer of sandy gravel / crushed stone over moist sandy clay and clayey sand. These upper soils have a low to none expansive potential. However, it is possible that deeper expansive clay soils could be present beneath these low to none expansive soils.

All work should be performed in accordance with the Earthwork Guidelines (Section 9.0) of this report.

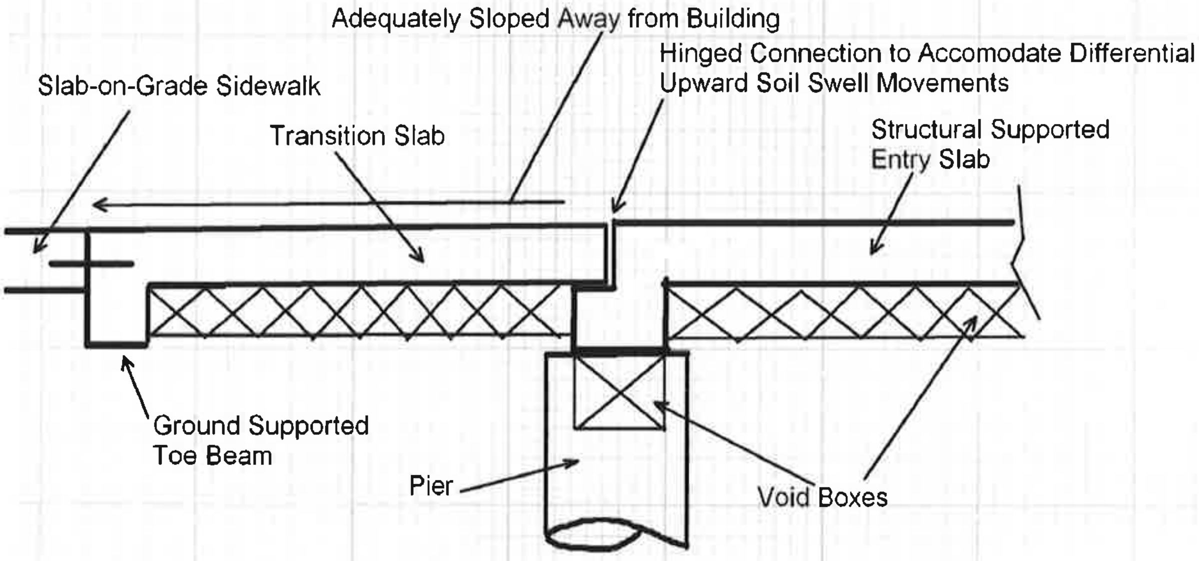
A polyethylene moisture barrier is recommended below slab-on-grade floor slabs where floor coverings or painted floor surfaces will be applied with products which are sensitive to moisture or if products stored on the building floors are sensitive to moisture. Procedures for installation of vapor barriers are recommended in ACI 302.

A floor slab supported on 36 inches of low PI select fill that is placed and compacted as specified above can be designed using a subgrade modulus value of 200 pci.

5.4 FLAT WORK AND PIPING CONSIDERATIONS

Provisions should be made for post-construction differential upward movement of adjacent sidewalks, flat work and piping. Site grading plans should include provisions for the effects of soil swell movements on access and entry slabs, adjacent sidewalks and all pavements. See Section 5.1 of this report. Utility line details and fixtures should consider the potential for differential movement beneath any piping. To prevent potential tripping hazards, access and entry slabs should be elevated above the adjacent sidewalks and pavement slabs (where possible).

If a structural supported floor slab is used, we recommend that all access and entry slabs also be structurally supported on drilled shafts and suspended above the active clays by a minimum 12 inch void space. To prevent potential tripping hazards, these access and entry slabs should be elevated above adjacent sidewalks and pavement slabs and provided with transition slabs over a 12 inch void space that are hinged at grade beam connections and provided with toe beams at connections to adjacent flatwork. A schematic detail of a transition slab is provided below.



Note: Transition slabs should have adequete length and should be sloped adequately to accommodate the soil swell indicated in the report. This is required so that a positive slope away from the building is maintained after soil swelling occurs.

All void spaces should be drained. We recommend that Alliance Geotechnical Group be retained to review the project drawings and specifications to ensure compliance with the geotechnical report.

6.0 UTILITY LINE CONSTRUCTION

6.1 TRENCH EXCAVATIONS

It is understood that open cut trench excavations will be performed along the entire alignment for new utility line installations. Information regarding the installation of new underground utilities has not been provided at this time. For the purpose of this report, we have assumed that the invert depths will be about 20 feet deep or shallower below the existing ground surface for the proposed utility lines.

As indicated on the Boring Logs (Figures 2 thru 9), soil conditions will vary considerably at the invert depths. Subsurface conditions anticipated to be encountered vary from existing fill soils to hard sandy clay to stiff silty clay to water bearing sands and gravels. Groundwater was generally encountered within the test borings at depths ranging from 8 to 20 feet below the existing ground surface. Shallower groundwater levels should be anticipated in all areas if construction occurs after periods of rain.

For trench excavations to any depth in unstable soil (sand, gravel, soft clay, existing fill, and/or submerged soil), or where seepage, or sloughing is observed in soil layers, it will be required to employ either sloped excavations or temporary bracing. It will be required to employ either sloped excavations or temporary bracing, regardless of the soil conditions encountered. General guidelines for the design of these two alternatives are discussed in the following sections.

6.2 OPEN CUTS

Recommended slope ratios for the respective soil conditions are presented graphically on Figure 14. Trench excavation should be cut back in a similar manner as described in Figure 14. It should be recognized that free standing slopes will be less stable when influenced by groundwater or saturated by rain. Surcharge loads, such as those resulting from excavation spoil, or construction equipment, should be placed no closer than five feet from the crest of the slope, or in accordance with OSHA regulations whichever is more stringent. Vehicle traffic should be maintained at least five feet from the edge of the crest.

Excavation may encounter non-compact fill soils placed during previous construction of underground utilities or construction of existing embankments. If encountered, these fill soils should be sheeted, shored and braced, or laid back on slopes no steeper than 1.5(H): 1(V) for short term (less than 8 hours) and no steeper than 2(H): 1(V) for long term conditions. The contractor will need to take measures to avoid undermining and damaging the existing underground utilities, railroad tracks and other structures.

6.3 TRENCH BRACING

Where site limitations require excavations to have vertical side walls, an internal bracing system will be necessary. Bracing may consist of timber or steel shoring or manufactured steel trench braces. The lateral pressure distribution to be used in the design of trench bracing may be determined as presented on Figure 15. It should be recognized that pressures are not included from hydrostatic pressures, surcharge loads, or traffic live loads at trench side walls, dynamic loads, and vibration, which if present, must be included in bracing design. In lieu of a shoring system, a trench shield consisting of a prefabricated rigid steel unit adequate to withstand anticipated lateral pressures may be used.

6.4 DEWATERING

Groundwater was generally encountered within the test borings at depths ranging from 8 to 20 feet below the existing ground surface. Shallower groundwater levels should be anticipated in all areas after periods of rain in the form of seepage through pervious soil layers.

In areas where groundwater is encountered, a system of ditches, sumps and/or well points, and pumping will be required to provide groundwater control. The design of the actual dewatering system required is the contractor's responsibility. This includes the control of tail-water flow through previous backfilled sections.

Note: Due to the potential for pressurized aquifer conditions in some areas along the alignment, dewatering must be performed to at least 3 feet below excavation cuts prior to trench excavations to prevent disturbance and softening of the supporting soils below the pipe. The dewatering should be performed to several feet (at least 3 feet) below the proposed bottom of the trench excavations in order to minimize softening of the supporting soils.

6.5 CONSTRUCTION CONSIDERATIONS

The following guidelines are presented to aid in the development of the excavation plans:

- Surface areas behind the crest of the excavations should be graded so that surface water does not pond within 15 feet of the crest, nor drain into the excavation.
- Heavy material stockpiles should not be placed near the crest of slopes per OSHA requirements. Similarly, heavy construction equipment should not pass over or be parked within 5 feet of the crest.
- The crest of slopes should be continually monitored for evidence of movement or potential problems. Freestanding slopes will become less stable when influenced by groundwater or saturation by rain.
- Identify other sources that might affect trench stability.
- Identify underground utilities prior to the start of excavation.
- Inspect trench excavations prior to the start of each work shift by qualified.
- Continuously monitor trench excavations by qualified personnel during construction.
- Immediately inspect trench excavations following a rain event or other water intrusion by qualified personnel.
- Inspect trench excavations by qualified personnel when changing soil conditions are encountered or after any occurrence that could have affected trench stability.
- Test and monitor for atmospheric hazards (i.e. low oxygen levels, hazardous fumes, toxic gases) within trench excavations.

6.6 TRENCH BACKFILL

The excavated soils can be used for trench backfill. Use of rock fragments greater than six (6) inches in any dimension should be prohibited since attaining uniform moisture and density without voids would be difficult. The backfill should be placed in thin compacted lifts as specified below. The fill materials should be free of surficial vegetation or debris.

Sandy clay soils, silty clay soils, very sandy clay and clayey sand having a PI between 4 and 25 should be used as utility trench backfill. The backfill should be placed in 8 inch horizontal loose lifts and compacted to at least 95% ASTM D698 at -2% to +2% above optimum moisture. Sands and silty sands having a PI of less than 4 should not be used as fill.

For fill depths below 15 feet or where it is desired to reduce post-construction settlements, the compaction level should be increased to a minimum of 98% ASTM D698, at -2% to +2% of optimum moisture. Anticipated settlements should be on the order of 1% of the total fill height.

7.0 EMBANKMENT CONSTRUCTION

Current plans consist of constructing 3H:1V slopes beyond the platforms. We further understand that a retaining wall (to be designed and constructed by others) will be constructed on the south side of the south platform. This retaining wall is to be constructed in the future after the subject platforms and embankments have been constructed. A temporary 3H:1V to 4H:1V embankment slope will be constructed south of the proposed south platform until this retaining wall is constructed by others.

It should be noted that according to TxDOT's Geotechnical Manual (Table 7-1 on page 7-3), embankments with 3H:1V slopes should have a PI of less than 20. Therefore, we recommend that the embankment be constructed with fill consisting of low PI select fill. The low PI select fill should be placed in horizontal benched lifts and should extend 5 feet beyond the toe of the embankment and 10 feet beyond the top of the embankment. Care should be taken to ensure that compaction is achieved between the interface of the existing embankment and the low PI select fill.

The material used as select fill should be a very sandy clay to clayey sand with a PI between 8 and 18. On-site soils meeting the criteria for low PI select fill may be used and on-site soils that are properly blended (using a pulvermixer) to achieve the criteria for low PI select fill may be used.

The fill should be spread in horizontal benched lifts, less than 8 inches thick, and uniformly compacted to a minimum of 98 percent of ASTM Standard D698 between -2% and +2% above the soils optimum moisture content. Compaction must be achieved along the benched excavation interface. Compacted fill should be placed beyond final lines and grades to allow the slope to be trimmed of loose soil during final grading. In this manner, compacted fill will be exposed along the final embankment slope rather than loose non-compacted fill. Compaction along sloped grades should not be allowed.

If it is desired to reduce the amount of select fill, on-site sandy clay soils (having a PI between 4 and 25) may be used in the deeper fill if compacted in 8 inch benched lifts as specified above. If the on-site sandy clay soils are used as fill, they should be capped with 5 feet of low PI select fill.

Each lift of fill should be placed over a compacted subgrade prior to any moisture loss of the compacted soil. The surface of each compacted fill lift should be watered after compaction is achieved to maintain a moist compact condition until the next lift is placed. Fill soils should be spread in loose lifts, less than 8 inches thick, and uniformly compacted to a minimum of 98 percent of ASTM D 698 between -2% and +2% of the soil's optimum moisture content.

8.0 TRACK SUBGRADE PREPARATION

We understand that a new track will be constructed adjacent to and south of the existing track. We further understand that 12 inches of ballast and 8 inches of subballast will be placed beneath the railroad tracks. We recommend that the subballast be placed directly on either lime/cement stabilized subgrade or on crushed concrete flex base.

8.1 SUBGRADE PREPARATION

It is recommended that provisions be made in the contract documents to provide for proofrolling after the required cuts have been performed and prior to any filling. The entire subgrade should be proofrolled. Proofrolling can generally be accomplished using a heavy (25 ton or greater total weight) pneumatic tired roller making several passes over the areas. Where soft, loose or compressible zones are encountered, these areas should be removed to a firm subgrade. Wet or very moist surficial materials may need to be undercut and either dried or replaced with proper compaction or replaced with a material which can be properly compacted. Any resulting void areas should be backfilled to finished subgrade in 8 inch compacted lifts compacted to 98% ASTM D 698 at -1% to +2% above optimum moisture content.

After proofrolling is performed and any soft, loose or compressible zones are removed and replaced, compact upper 8 inches of subgrade to 98% ASTM D698 as specified above. Then fill to track subgrade in accordance with Section 7.0 of this report.

8.2 TRACK SUBGRADE STABILIZATION

Surficial soils along the proposed new pavements are anticipated to vary from expansive clay and sandy clay soils to low-plastic sands. The clay soils react with hydrated lime which serves to improve their support value at higher moisture levels and provides a firm, uniform subgrade beneath the paving. Cement stabilization is used to improve the support value of sandy soils. Due to the highly variable subgrade soils, it is recommended that subgrade be first stabilized with lime followed by cement stabilization. These recommendations are provided below.

Based on results from Atterberg limits testing, six (6) to eight (8) percent hydrated lime by dry weight (43 to 58 pounds per square yard per 8-inch depth) will be required to stabilize the existing clay subgrade. The actual lime requirement will depend upon the actual subgrade soils exposed at final grade and should be determined at the time of construction.

The lime should be thoroughly mixed and blended with the active subgrade soil (TxDOT Item 260). We recommend that this lime stabilization extend 2 feet beyond exposed pavement edges, where possible, in order to reduce the effects of shrinkage during extended dry periods.

Note: After final grading has been achieved, depth checks and PI verification checks should be performed to verify that the specified depth of stabilization is present and that the PI has been reduced to 15 or less.

Project specifications should allow a curing period between initial and final mixing of the lime/soil mixture. After initial mixing, the lime treated subgrade should be lightly rolled and maintained at or within 5 percentage points above the soil's optimum moisture content until final mixing and compaction. We recommend a 3-day curing period for these soils. The following gradation requirements are recommended for the stabilized materials prior to final compaction:

	Percent
Minimum Passing 1 3/4" Sieve	100
Minimum Passing 3/4" Sieve	85
Minimum Passing No. 4 Sieve	60

All non-slaking aggregates retained on the No. 4 sieve should be removed prior to testing.

After the subgrade has been properly lime-stabilized, the lime treated soils should then be cement stabilized. It is anticipated that 8 percent cement by dry weight (57 pounds per square yard per 8-inch depth) is required to stabilize the lime treated soils.

The cement should be thoroughly mixed and blended with the top 8 inches of the subgrade and the mixture compacted to a minimum of 98 percent of maximum dry density as determined in accordance with ASTM D 698, within 2 percentage points of the soil's optimum moisture content. We recommend that stabilization extend 2 feet beyond exposed pavement edges, where possible, in order to reduce the effects of shrinkage during extended dry periods. The cement stabilization should be performed in accordance with TxDOT Item 275.

Fine grading filling should not be allowed after curing per TxDOT Item 275. The surface should be thoroughly rolled with a pneumatic tire roller and "tight-bladed" by a power grader removing all loosened soil and cement from the pavement area. Concrete should be placed directly on top of undisturbed cement stabilized soils.

The following gradation requirements are recommended for the cement stabilized materials before final compaction:

	Percent
Minimum Passing 1 3/4" Sieve	100
Minimum Passing 3/4" Sieve	85
Minimum Passing No. 4 Sieve	60

All non-slaking aggregates retained on the No. 4 sieve should be removed before testing.

Note: Cement application, final mixing, gradation acceptance, and compaction of each area treated must all be accomplished within a two (2) hour period. See Item 275 for curing requirements, etc.

The stabilized subgrade should be protected and moist cured or sealed with a bituminous material for a minimum of 7 days or until the pavement materials are placed. Pavement areas should be graded to prevent ponding and infiltration of excessive moisture on or adjacent to the pavement areas.

8.3 RECYCLED CRUSHED CONCRETE BASE

In lieu of performing lime/cement stabilization, eight (8) inches of recycled crushed concrete base could be used. The flex base should be compacted at optimum to +2% of optimum to a minimum of 95% Modified Proctor density (ASTM D1557). The base materials should comply with TxDOT Item 247, Type D, Grade 1. We recommend that the base materials extend at least two feet beyond the pavement edges (where possible).

Prior to placing the crushed concrete flex base, the subgrade should be proofrolled prior to subgrade compaction. Proofrolling should be performed in accordance with Section 8.1 of this report. The upper eight (8) inches of pavement subgrade should be scarified and compacted prior to placing crushed concrete flex base. The upper eight (8) inches of the subgrade soils should be compacted at -1% to +2% of optimum moisture to a minimum of 98% Standard Proctor density (ASTM D 698). If a rain event occurs prior to placement and compaction of the flex base, the subgrade should be aerated and re-tested prior to paving.

9.0 EARTHWORK GUIDELINES

9.1 SITE GRADING AND DRAINAGE

All grading should provide positive drainage away from the proposed structure and should prevent water from collecting or discharging near the foundations. Water must not be permitted to pond adjacent to the structures during or after construction. Otherwise, differential soil swell movements, and resulting differential foundation movements could exceed the estimates contained within this report.

Leave outs for drilled shafts or around the perimeter of the structures should not be allowed to collect and hold water. These leave outs should be pumped out as needed.

Surface drainage gradients should be designed to divert surface water away from the platforms and edges of pavements. Surface drainage gradients of sidewalks, pavements, and landscaping, within 20 feet of the buildings should be constructed with maximum slopes allowed by local codes and maintained after occupancy (5% minimum in landscaping areas and 2% minimum for flatwork subject to ADA requirements). If flatter grades are used, a permanent horizontal moisture barrier should be used for a distance of 15 feet from the platforms (20 mil poly sheeting with UV protection attached to the building and covered with

16 inches of on-site compacted clay (compacted in 8 inch lifts above optimum to a minimum of 95% of ASTM 698).

Provisions should be made for large post-construction differential upward movement of adjacent flat work. Site grading plans should include provisions for the effects of soil swell movements on access and entry slabs, adjacent sidewalks, and all pavements. See Section 5.1 of this report.

The roofs should be provided with gutters and downspouts to prevent the discharge of rainwater directly onto the ground adjacent to the building foundations. Downspouts should discharge directly onto well-drained areas or drainage swales, if possible. Roof downspouts and surface drain outlets should discharge into erosion–resistant areas. Water permitted to pond in planters, open areas, or areas with unsealed joints next to the structures can result in excessive slab or pavement movements as indicated in this report.

Exterior sidewalks and pavements will be subject to large post construction movement as indicated in this report. These potential movements should be considered during preparation of the grading plan. Flat grades should be avoided. Where concrete pavement is used, joints should also be sealed to prevent the infiltration of water. Since post construction movement of pavement and flat work will occur, joints particularly around the building should be periodically inspected and resealed where necessary.

9.2 PROOFROLLING AND SUBGRADE PREPARATION

Prior to placing fill in non-building areas, the exposed subgrade in areas to receive fill should be stripped and proofrolled using a fully loaded dump truck. Soft areas should be undercut and replaced with compacted on–site soils. The surface should then be scarified to a depth of 8 inches and recompacted to a minimum of 95 percent of the maximum density as determined by ASTM D 698 between optimum and +3% of its optimum moisture content.

9.3 ON-SITE CLAY FILL PLACEMENT IN PAVEMENT AND LANDSCAPING AREAS

The on–site surficial clays may be used for general grading and filling. The fill materials should be free of surficial vegetation or debris. Clay materials should be spread in loose lifts, less than 8 inches thick and uniformly compacted to a minimum of 95 percent of the maximum density as determined by ASTM D 698 (Standard Proctor) between optimum and +3% above its optimum moisture content. The upper 8 inches of subgrade soil within pavement areas should be compacted at -2% to +2% of optimum moisture to a minimum of 98% Standard Proctor density (ASTM D 698).

9.4 ON-SITE CLAY FILL PLACEMENT IN BUILDING AREAS

On-site clay soils may be used as fill in building areas. All fill should be placed and compacted in maximum 8-inch lifts. Prior to fill placement, the subgrade should be proofrolled and compacted to a depth of 8 inches. See Section 5.3.2 of this report.

9.4.1 MOISTURE CONDITIONING PRIOR TO COMPACTION

Each layer shall be leveled with approved equipment. After spreading, each layer shall be thoroughly manipulated by plowing, discing, or other approved methods of the full depth of the layer being placed to insure uniform density and moisture distribution for proper compaction. The moisture content at the time of compaction shall be within the range specified in these special provisions. If the material is too dry, it shall be moistened by watering before and during manipulation, to properly condition the material for compaction. If the material is too wet, the compaction operation shall be delayed until the moisture content has been reduced to within satisfactory compaction range.

Because of time of completion limitations, thoroughly processing of the on-site clay soils will be required during manipulation if the moisture content is below optimum at the time of placement. Each fill lift should be processed until the soil mixture is free of large clods to allow uniform moisture distribution and uniform compaction within the entire fill lift. This is particularly important if highly plastic clay soils are to be used as fill in the building pads. The amount of processing and reworking required to achieve uniform moisture conditions can be reduced by pre-wetting the onsite soils prior to placement.

9.4.2 QUALITY ASSURANCE REQUIREMENTS

As a quality control measure, pocket penetrometer (P.P.) tests shall be performed with each field density test during construction as further verification that proper moisture conditioning is being achieved within the clay fill soils. Proper moisture conditioning should be indicated with hand penetrometer readings within the following ranges.

Hand Penetrometer:	3.0 to 4.5 tsf (clayey sand)
	3.0 to 4.0 tsf (sandy clay)

Similarly, P.P. tests should be performed on each Proctor Compaction Point in the laboratory for correlation and verification of the desired P.P. range with respect to Proctor moisture, density and swell (with verification that volumetric swell is less than 1% at the targeted moisture content). Prior to construction, it should be confirmed that the "targeted moisture contents" recommended in this report will result in an average volumetric swell of less than 1% within the upper 10 foot. This swell testing should be performed in conjunction with all proctor compaction testing. If the targeted moisture contents result in an average swell of over 1% or in a P.P. value outside the range indicated above, AGG should be contacted to determine if the intent of the geotechnical design is being achieved with respect to required swell reduction and bearing capacity.

9.5 SELECT FILL PLACEMENT IN BUILDING AREAS

The material used as select fill should be a sandy clay to clayey sand (uniform consistency free of clay clods) with a plasticity index between 5 and 14. The fill should be spread in loose lifts, less than 8 inches thick, and uniformly compacted to a minimum of 98 percent of ASTM Standard D 698 between -2% and +2 percentage points above the soil's optimum moisture content. Increased compaction to 98% is recommended due to heavy floor loading. The upper 24 inches of fill in unpaved areas near the building should consist of compacted on–site clay to minimize water infiltration into the select fill (compacted in 8 inch lifts at +1% to +4% to 95% ASTM D698).

10.0 TREE EFFECTS

It should be recognized that concrete slabs (platform floor slabs, sidewalks, pavements, etc.) will be subject to long term settlement due to ground shrinkage caused by moisture absorption of tree root systems. To minimize long term settlements, trees and deep rooted shrubs should not be planted within 35 feet of the platforms and critical flatwork areas sensitive to movement in order to minimize settlements caused by ground shrinkage associated with moisture absorption of the tree root systems. Also, the area beneath the un-pruned mature tree drip lines and to limits of at least 10 feet beyond the drip line should not be paved. The area beneath the drip line should be landscaped and irrigated.

However, we understand that current plans consist of planting trees within landscaping leave-outs within the proposed platform structures. We recommend that an arborist be contacted regarding the installation, effectiveness, and use of root barriers and irrigated tree wells for the south platform tree areas to minimize future slab-on-grade foundation and flatwork settlements due to ground shrinkage caused by tree root absorption of the moisture conditioned clay soils. In addition, consideration should be given to filling full depth with low PI select fill in lieu of using moisture conditioned clay soils. The low PI select fill would still be subject to long-term ground shrinkage due to tree root absorption but it would be much less than what would occur if moisture conditioned clay soils were used.

11.0 FIELD SUPERVISION

Many problems can be avoided or solved in the field if proper inspection and testing services are provided. It is recommended that all pier excavations, footing excavations, proofrolling, site and subgrade preparation, subgrade stabilization and pavement construction be monitored by a qualified engineering technician. Density tests should be performed to verify compaction and moisture content of any earthwork. Inspection should be performed prior to and during concrete placement operations. Alliance Geotechnical Group employs a group of experienced, well-trained technicians for inspection and construction materials testing who would be pleased to assist you on this project.

12.0 LIMITATIONS

The professional services, which have been performed, the findings obtained, and the recommendations prepared were accomplished in accordance with currently accepted geotechnical engineering principles and practices. The possibility always exists that the subsurface conditions at the site may vary somewhat from those encountered in the test borings. The number and spacing of test borings were chosen in such a manner as to decrease the possibility of undiscovered abnormalities, while considering the nature of loading, size, and cost of the project. If there are any unusual conditions differing significantly from those described herein, Alliance Geotechnical Group, Inc. should be notified to review the effects on the performance of the recommended foundation system.

The recommendations given in this report were prepared exclusively for the use of the client, their client and their consultants. The information supplied herein is applicable only for the design of the previously described development to be constructed at locations indicated at this site and should not be used for any other structures, locations, or for any other purpose.

We will retain the samples acquired for this project for a period of 30 days subsequent to the submittal date printed on the report. After this period, the samples will be discarded unless otherwise notified by the owner in writing.

FIGURES

ALLIANCE GEOTECHNICAL GROUP

Project No: **DE19-282**	**PLAN OF BORINGS**	
	TRINITY LAKES PARK AND RIDE HURST, TEXAS	**Figure No:** **1**

LOG OF BORING B-1

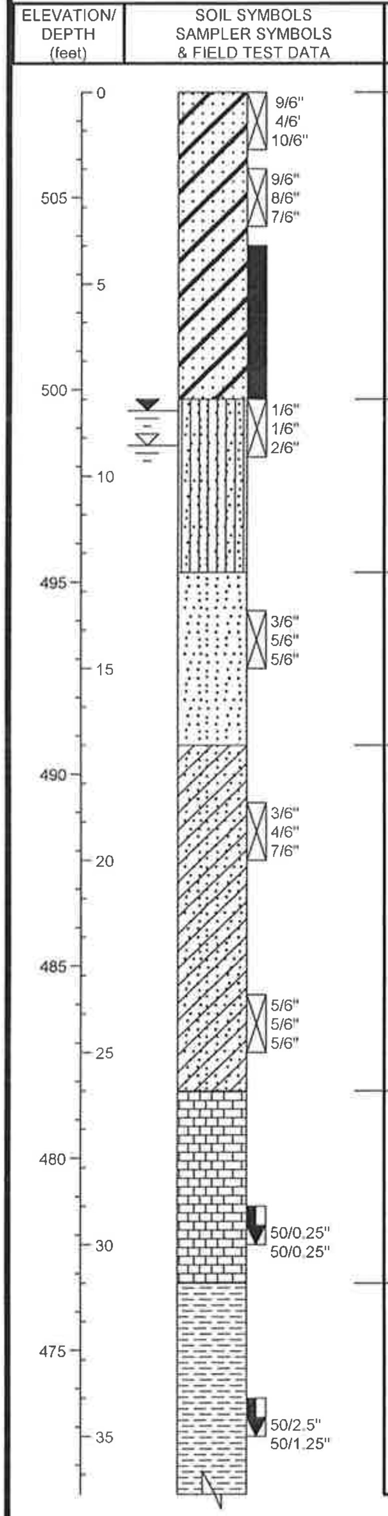
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **08/22/2019** Elev.: **507.75** Location: **See Figure 1**

Depth to water at completion of boring: **9.2'**

Depth to water when checked: **at 24 hours** was: **8.3'**

Depth to caving when checked: was:



DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Reddish brown sandy CLAY	5						4.5++		
	8						4.5++		
	11	39	15	24	62	127	4.5++		
							4.5++		
	15						4.5+		
-water seepage at 8'	14					121	3.5	7.0	7.3
Reddish brown silty SAND	17			NP	18				
Tan medium coarse to coarse SAND	15	17	14	3	7				
Tan and light gray clayey SAND w/ sand layers	20								
Very hard gray LIMESTONE w/ hard shale layers									
Very firm to hard dark gray SHALE									

Notes:

FIGURE:2

LOG OF BORING B-1

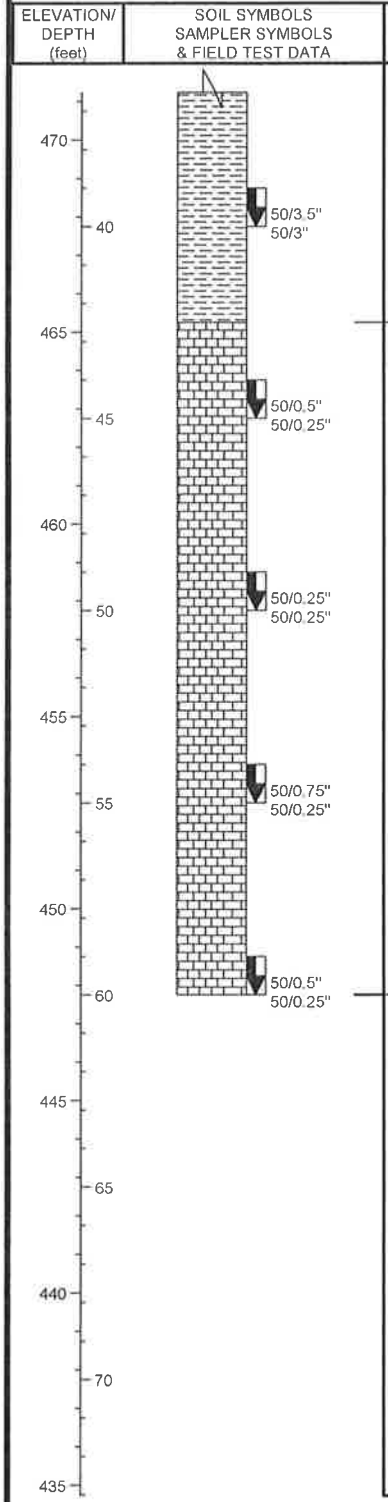
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **08/22/2019** Elev.: **507.75** Location: **See Figure 1**

Depth to water at completion of boring: **9.2'**

Depth to water when checked: **at 24 hours** was: **8.3'**

Depth to caving when checked: was:



DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Very firm to hard dark gray SHALE									
Very hard gray LIMESTONE w/ hard shale layers									
Boring terminated at 60'									

Notes:

FIGURE:3

Alliance Geotechnical Group, Inc.

LOG OF BORING B-2

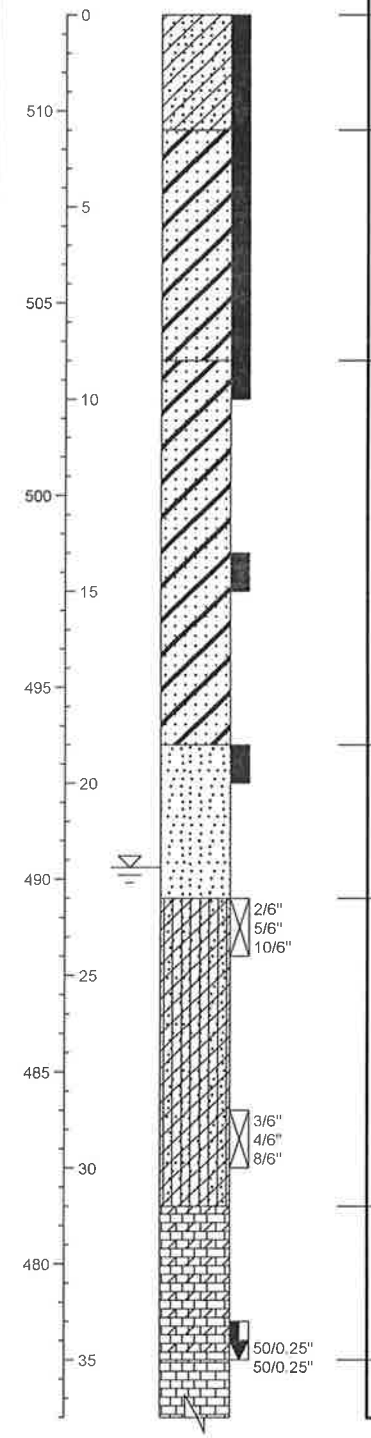
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **09/16/2019** Elev.: **512.5** Location: **See Figure 1**

Depth to water at completion of boring: **22.2'**

Depth to water when checked: was:

Depth to caving when checked: was:



DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P PEN tsf	UNCON ksf	Strain %
Dark brown and brown clayey SAND w/ gravel and sand layers (FILL)	15						4.0		
							4.25		
	7	16	15	1					
Reddish brown very sandy CLAY							4.5+		
	12	26	12	14	58	120	4.5++		
							4.5++		
							4.5++		
	13						4.5+		
							4.5+		
Light gray and tan sandy CLAY	15	33	12	21			4.5+		
	17					114	4.5+		
-seepage at 18' during drilling									
Tan fine to medium coarse SAND w/ clay seams	21			NP	14				
Light gray and tan clayey silty SAND	20				49				
Hard to very hard tan weathered LIMESTONE, fractured									
Very hard gray LIMESTONE									

Notes:

FIGURE:4

LOG OF BORING B-2

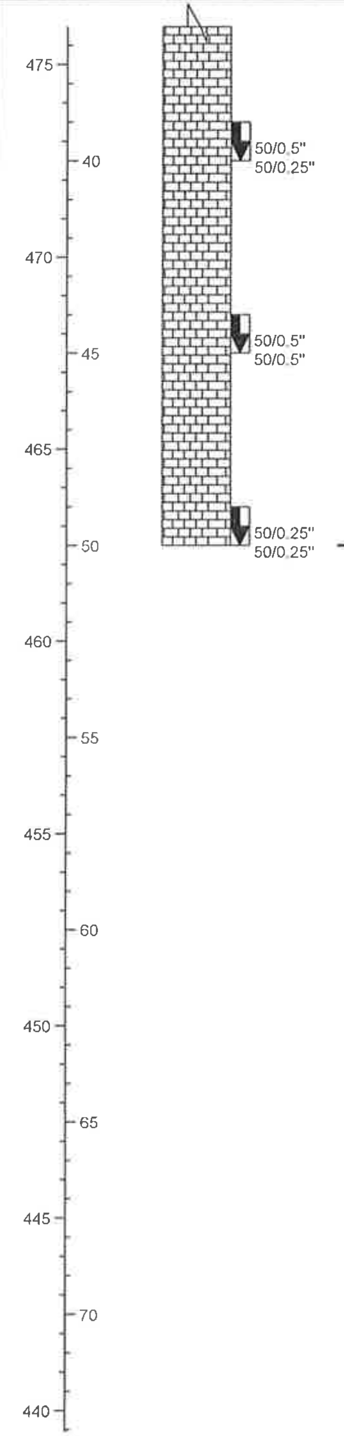
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **09/16/2019** Elev.: **512.5** Location: **See Figure 1**

Depth to water at completion of boring: **22.2'**

Depth to water when checked: was:

Depth to caving when checked: was:



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
	50/0.5" 50/0.25" 50/0.5" 50/0.5" 50/0.25" 50/0.25"	Very hard gray LIMESTONE									
		Boring terminated at 50'									

Notes:

FIGURE:5

Alliance Geotechnical Group, Inc.

LOG OF BORING B-3

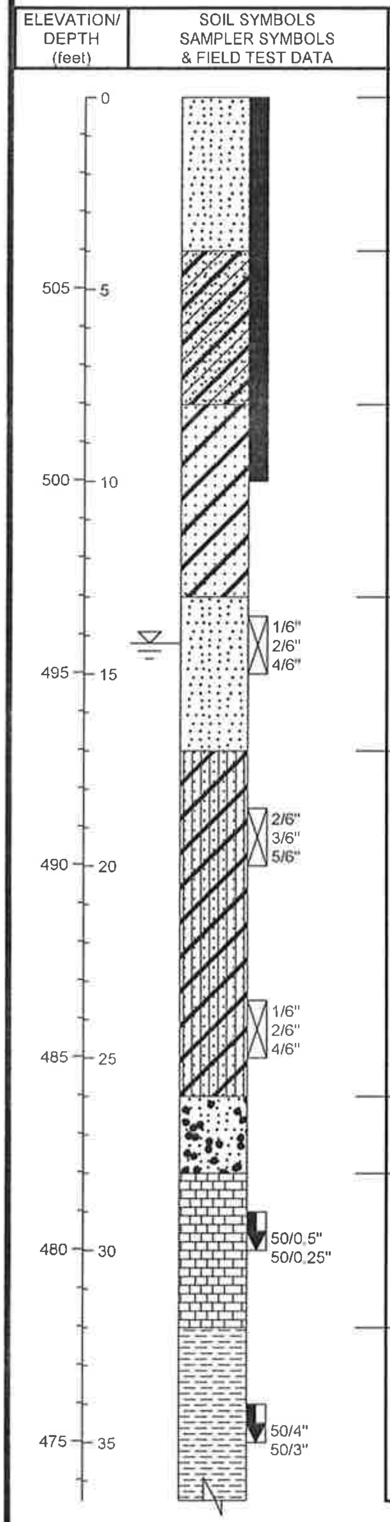
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **08/22/2019** Elev.: **509.95** Location: **See Figure 1**

Depth to water at completion of boring: **14.2'**

Depth to water when checked: was:

Depth to caving when checked: was:



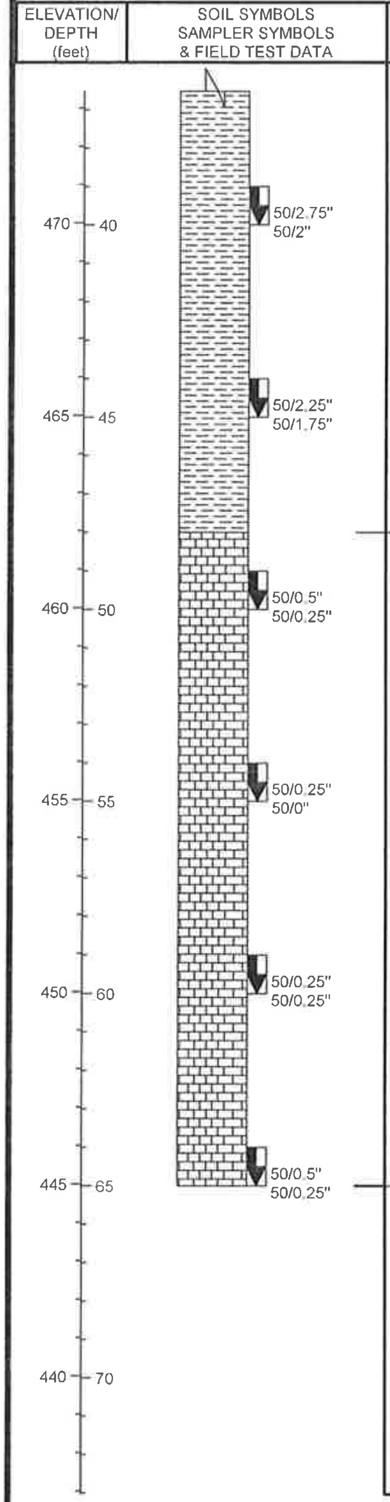
DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Brown SAND w/ gravel pieces (FILL)									
Reddish brown and tan very sandy CLAY to clayey SAND w/ gravel pieces and sand layers (FILL)	12				48		4.5		
	19	30	14	16		94	1.0		
	15						2.0		
	15					116	2.0		
Light brown sandy CLAY w/ calcareous nodules	16	49	15	34		114	4.5		
							4.5		
-water seepage at 13' during drilling									
Tan fine SAND	18			NP	9				
Tan silty sandy CLAY w/ sand layers	20	24	13	11	60				
Tan SAND and GRAVEL									
Very hard gray LIMESTONE w/ hard shale layers									
Very firm to hard dark gray SHALE w/ very hard limestone layers									

Notes:

FIGURE:6

LOG OF BORING B-3

Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**
Date: **08/22/2019** Elev.: **509.95** Location: **See Figure 1**
Depth to water at completion of boring: **14.2'**
Depth to water when checked: was:
Depth to caving when checked: was:

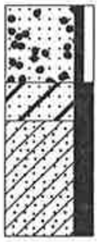


DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Firm dark gray <u>SHALE</u> w/ very hard limestone layers									
Very hard gray <u>LIMESTONE</u> w/ hard shale layers -large shale layers below 51'									
Boring terminated at 65'									

Notes: **FIGURE:7**

LOG OF BORING B-3B

Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**
Date: **01/12/2020** Elev.: **515** Location: **See Figure 1**
Depth to water at completion of boring: **Dry**
Depth to water when checked: was:
Depth to caving when checked: was:

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
515 — 0		Sandy GRAVEL and Crushed STONE (FILL)									
		Moist reddish brown and tan sandy CLAY w/ trace gravel (FILL)							1.5		
		Reddish brown clayey SAND w/ clay seams (FILL)	13	22	13	9	37	121	2.0		
510 — 5											
		Hand Auger terminated at 6'									
505 — 10											
500 — 15											
495 — 20											
490 — 25											
485 — 30											
480 — 35											

Notes:

FIGURE:8

LOG OF BORING B-5

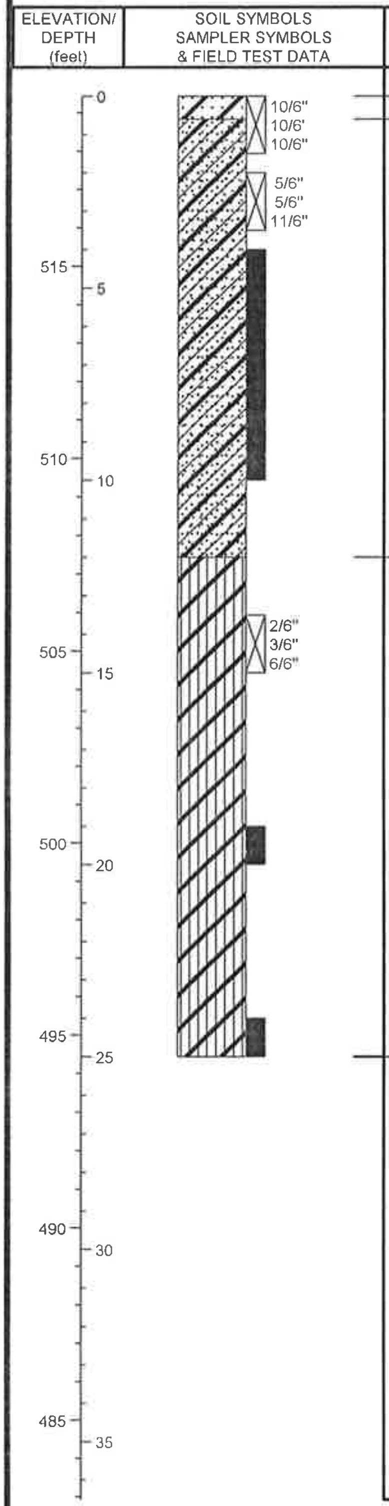
Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **08/26/2019** Elev.: **519.44** Location: **See Figure 1**

Depth to water at completion of boring: **Dry**

Depth to water when checked: was:

Depth to caving when checked: was:



DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Reddish brown sandy CLAY w/ crushed concrete fragments (FILL)									
Reddish brown sandy CLAY to clayey SAND	9	29	14	15					
	11	39	15	24		126	4.5++		
							4.5++		
	11						4.5++		
	9						4.5++		
							4.5++		
	8						4.5++		
Tan and reddish brown silty CLAY w/ sand seams									
	17	30	15	15	78				
	19				72		1.25		
	19						1.25		
Boring terminated at 25'									

Notes:

FIGURE:9

LOG OF BORING B-6

Project: **Trinity Lakes Park and Ride - Hurst, Texas** Project No.: **DE19-305**

Date: **08/21/2019** Elev.: **501.62** Location: **See Figure 1**

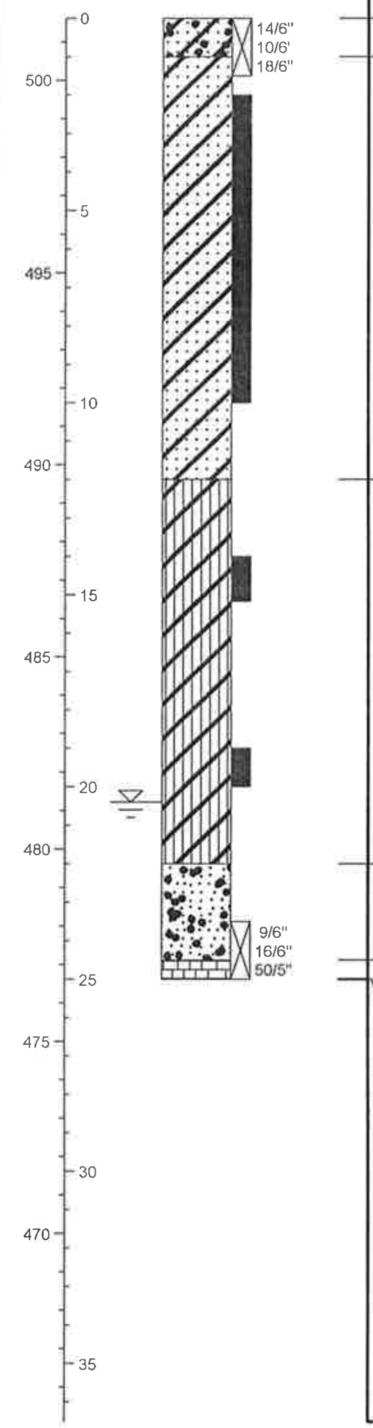
Depth to water at completion of boring: **20.4'**

Depth to water when checked: was:

Depth to caving when checked: was:

ELEVATION/ DEPTH (feet)

SOIL SYMBOLS SAMPLER SYMBOLS & FIELD TEST DATA



DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
Reddish brown sandy CLAY mixed with GRAVEL (FILL)	6								
Reddish brown sandy CLAY									
	10	39	16	23		126	4.5++		
							4.5++		
	8						4.5++		
							4.5++		
	13						4.5++		
							4.5++		
							4.5++		
	13						4.5+		
Light gray and tan silty CLAY									
	32	40	13	27	87		2.0		
	24						2.25		
-water seepage at 22' during drilling									
Gray SAND and GRAVEL									
	10								
Very hard gray LIMESTONE									
Boring terminated at 25'									

Notes: **FIGURE:10**

KEY TO LOG TERMS & SYMBOLS

Strata symbols

Symbol	Description
▨	CLAY, sandy
▥	SAND, silty
░	SAND
▨	SAND, clayey
▦	LIMESTONE
▤	SHALE
▨	SAND, clayey silty
▦	LIMESTONE, weathered
▨	Sandy CLAY, Clayey SAND
▨	CLAY, silty sandy
▒	SAND, GRAVEL
▒	GRAVEL, sandy
▨	CLAY, silty
▨	Sandy Gravelly CLAY

Misc. Symbols

Symbol	Description
▼	Water table when checked
▽	Water table at boring completion
⌇	Boring continues

Soil Samplers

Symbol	Description
☒	Standard Penetration Test

Notes:

1. Exploratory borings were drilled on dates indicated using truck mounted drilling equipment.

2. Water level observations are noted on boring logs.

3. Results of tests conducted on samples recovered are reported on the boring logs. Abbreviations used are:
   DD = natural dry density (pcf)
   MC = natural moisture content (%)
   Uncon.= unconfined compression (tsf)
   P.Pen.= hand penetrometer (tsf)
   LL = liquid limit (%)
   PL = plastic limit (%)
   PI = plasticity index
   -200 = percent passing #200

4. Rock Cores
   REC = (Recovery) sum of core sample recovered divided by length of run, expressed as percentage.
   RQD = (Rock Quality Designation) sum of core sample recovery 4" or greater in length divided by the run, expressed as percentage.

FIGURE:11

KEY TO LOG TERMS & SYMBOLS

Symbol	Description
<u>Soil Samplers</u>	
■	Thin Wall Shelby Tube
◧	Auger
▼	THD Cone Penetration Test

FIGURE:12

SWELL TEST RESULTS

BORING NO.	DEPTH (FEET	UNIT WEIGHT (pcf)	ATTERBURG LIMITS			IN-SITU MOISTURE CONTENT	FINAL MOISTURE CONTENT	LOAD (psf)	% VERTICAL SWELL
			LL	PL	PI				
B-1	4-5	127.4	39	15	24	11.2	15.0	563	5.3
B-2	4-5	119.8	26	12	14	11.9	12.6	563	0.5
B-2	14-15	113.9	-	-	-	16.7	17.2	1,813	0.1
B-3	8-9	114.1	49	15	34	15.6	17.4	1,063	1.6
B-3B	4-5	120.8	22	13	9	13.2	13.2	563	0
B-5	4-5	125.5	39	15	24	10.5	12.7	563	2.1
B-6	2-3	126.1	39	16	23	10.4	15.3	313	6.5

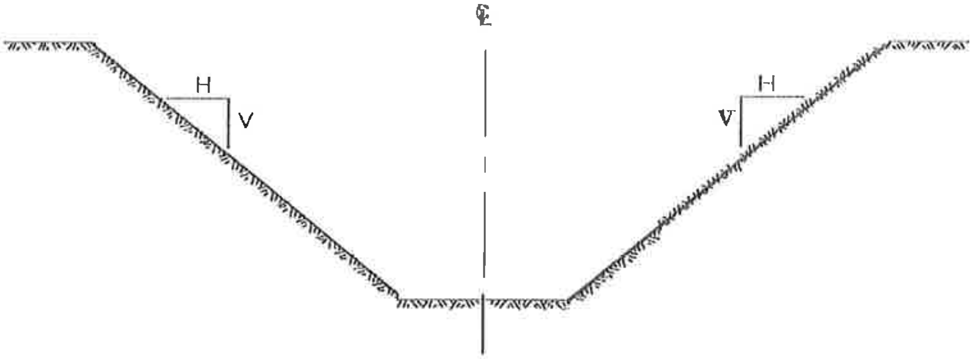
PROCEDURE:

1. Sample placed in confining ring, design load (including overburden) applied, free water with surfactant made available, and sample allowed to swell completely.
2. Load removed and final moisture content determined.

SWELL TEST RESULTS
TRINITY LAKES PARK AND RIDE
HURST, TEXAS
ALLIANCE GEOTECHNICAL GROUP
DE19-305 | **Date: 10/07/2019** | **FIGURE: 13**

RECOMMENDED SLOPE RATIOS

	Short Term (under 8 hours)		Long Term (over 8 hours)	
SOIL / ROCK	**H**	**V**	**H**	**V**
Existing fill materials, sand, gravel, silty sand, clayey sand, very sandy clay, and soft clayey soils (hand penetrometer of 0.5 to 0.9 tsf)	1-½	1	2	1
Submerged soils and/or fractured rock (weathered limestone) from which water is seeping *	1-½	1	2	1
Stiff to hard sandy clay and silty clay above existing groundwater level	1	1	1	1



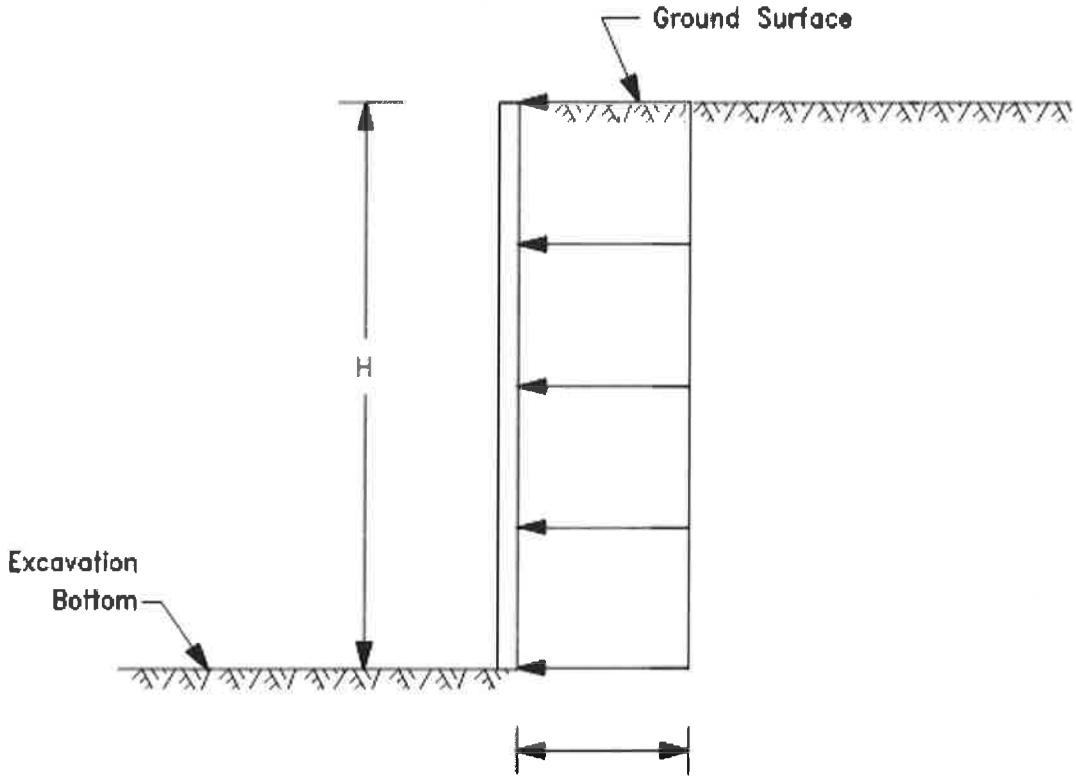
* In accordance with the best interpretation of OSHA regulations, submerged soil is defined as water bearing granular soils, fissured clay soils, or fractured rock (tan fractured weathered limestone, or fractured gray limestone) from which groundwater is seeping.

NOTE: Recommended slope ratios may be subject to reduced stability under the influence of groundwater or saturation by rain. Recommended slope ratios are designed for safety only of temporary excavations and are not designed to prevent limited sloughing during construction.

ALLIANCE GEOTECHNICAL GROUP

TRINITY LAKES PARK & RIDE **HURST, TEXAS**	**RECOMMENDED SLOPE RATIOS**	**FIGURE** **14**
DATE: October 7, 2019	**PROJECT NO: DE19-305**	

LATERAL EARTH PRESSURES FOR INTERNALLY BRACED EXCAVATIONS



$s_h = k\ g\ H$

WHERE:

s_h = Lateral Earth Pressure, psf.

g = Saturated Unit Weight of Soil
Use 130 pcf for Clay.
Use 140 pcf for Limestone.

H = Height of Excavation, ft.

k = Earth Pressure Coefficient;
Use 0.40 for Existing Fill Soils, Gravel, Sand, Silty Sand, Clayey Sand, & Very Sandy Clay
Use 0.35 for Sandy Clay and Silty Clay

NOTES:

1) If water is not allowed to drain from behind shoring or bracing, full hydrostatic pressure must be considered.
2) Surcharge loads and traffic live loads, if present, must also be considered.

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TRINITY LAKES PARK AND RIDE HURST, TEXAS	LATERAL EARTH PRESSURES	Figure 15
DATE: 10-08-2019	PROJECT NO: DE19-305	