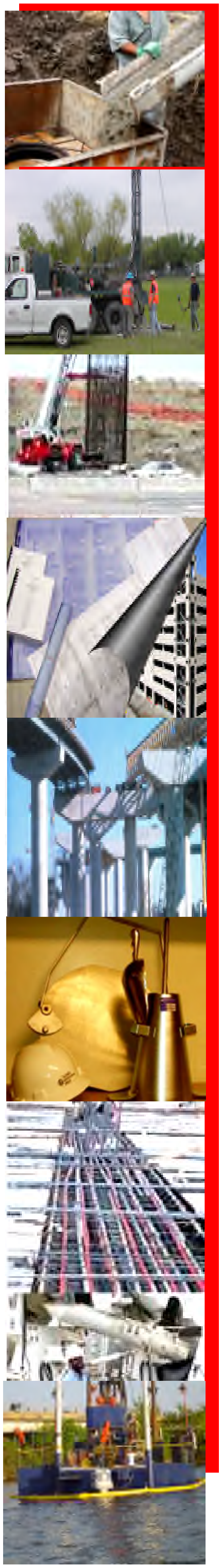

GEOTECHNICAL INVESTIGATION

PAVEMENT RECONSTRUCTION
TURNER WARNELL ROAD
ARLINGTON, TEXAS

AGG REPORT: DE19-335R2

JANUARY 8, 2020

PREPARED FOR:

HUITT-ZOLLARS, INC.

PRESENTED BY:

ALLIANCE GEOTECHNICAL GROUP

Geotechnical Engineering – Construction Services – Construction Materials Engineering Testing
3228 Halifax Street - Dallas, TX 75247 Ph. 972.444.8889 FX. 972.444.8893

January 8, 2020

Ms. Katie D. McCarty, P.E.
Associate
Huitt-Zollars, Inc.
1717 McKinney Avenue Suite 1400
Dallas, TX 75202

Phone: (214) 871-3311 Ext. 10100
E-mail: kmccarty@huitt-zollars.com

Re: Geotechnical Investigation
Pavement Reconstruction
Turner Warnell Road
Arlington, Texas
AGG Report No. DE19-335R2

Dear Ms. McCarty:

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 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

ALLIANCE GEOTECHNICAL GROUP STATE OF TEXAS MICHAEL DAINE ROLAND 96043 LICENSED PROFESSIONAL ENGINEER F-1970

Logan Tucker, E.I.T.
Staff Engineer

Michael D. Roland, P.E.
Vice President

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PAVEMENT RECONSTRUCTION
TURNER WARNELL ROAD (S. COOPER STREET TO MATLOCK ROAD)
ARLINGTON, TEXAS

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The project consists of reconstructing approximately 8,000 linear feet of Turner Warnell Road from S. Cooper Street to Matlock Road in Arlington, Texas. The proposed alignment is a two lane divided roadway. The existing roadway conditions consist of distressed pavement with longitudinal and lateral cracks and patched areas. The reconstructed pavement will consist of a new concrete roadway with curb and gutter and sidewalks.

In addition to the new pavement reconstruction, it is understood that new water and wastewater lines will be installed. We understand that the proposed utility line will have invert depths of 15 feet or shallower. We further understand that the proposed underground utility lines will be installed using open cut construction along the entire alignment.

1.2 PURPOSE AND SCOPE

The purposes of this geotechnical investigation were to: 1) explore the subsurface conditions at the site, 2) provide boring logs that present subsurface conditions encountered including water level observations and laboratory test results, 3) provide comments on the presence and effect of expansive soils on the subjected roadways, 4) provide pavement subgrade preparation and pavement section recommendations, and 5) provide recommendations for open cut construction and trench backfill recommendation for the proposed new water and wastewater lines. This report was prepared in general accordance with AGG Proposal No. P19-0209E dated February 11, 2019.

2.0 FIELD INVESTIGATION

The field investigation consisted of drilling a total of nine (9) test borings (Boring B-1 thru B-9) along the proposed Turner Warnell Road reconstruction alignment. The test borings were drilled to depth of 20 feet below the existing ground surface. A truck-mounted drilling rig was used to advance these borings and to obtain samples for laboratory evaluation. The borings were located at the approximate locations shown on the Plan of Borings (Figure 1).

Undisturbed samples of the 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.

The relative densities of the subsurface soils were also 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 18 inches 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 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 Figure 11.

3.0 LABORATORY TESTING

The 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 dry unit weight determinations. Hand penetrometer tests were performed on the soil samples to provide indications of the swell potential and the foundation bearing properties of the subsurface strata. Unconfined compressive tests were also performed on select clay samples to evaluate the foundation bearing capacity. The results of these tests are presented on the Logs of Borings (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 (Figure 12). In addition, to determining the potential swell that exists at the in-situ moisture condition, two samples were air dried prior to performing the swell testing in order to simulate dry conditions. Lime series tests were performed on selected soil samples in order to determine the optimum amount of lime for stabilization (see Figure 14). In addition, soluble sulfate testing was performed on selected samples to determine the potential for sulfate/lime induced heave (see Figure 13).

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 GENERAL SITE CONDITIONS

As described above, pavement reconstruction is to be performed along Turner Warnell Road in Arlington, Texas. The existing asphalt pavement is in poor condition with cracking in many areas. There are medium to large trees present adjacent to the existing pavement in a few areas. Some large trees have drip lines that extend over the pavement. See Plan of Borings (Figure 1) for pavement alignment and aerial view.

4.2 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the borings, including descriptions of the various strata and their depths and thickness, 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.

4.3 SITE GEOLOGY

As shown on the Geologic Atlas of Texas, the site is located underlain by the Woodbine Formation and the Eagle Ford Formation. The Woodbine Formation typically consists of shale, sandstone, and cemented sand interbedded with clay seams. The Eagle Ford Formation typically consists of shale, sandstone, and limestone interbedded with clay seams. Soils derived from the Eagle Ford are typically plastic clays exhibiting high shrink/swell potential with variations in moisture content. These Eagle Ford shaley clay soils typically have very high soluble sulfate levels which are very corrosive to buried concrete and can cause sulfate/lime induced heave. In addition, unmapped alluvial deposits were also encountered within the upper soils. Alluvial soils typically consist of clay, silt, sand and gravel.

4.4 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 seepage was generally not encountered during drilling the drilling operation. However, ground water was encountered at depths ranging from 13 to 18.4 feet below the existing grade within two of the test borings (Borings B-6 and B-7). The borings were immediately backfilled and the pavement patched prior to moving to the next boring location. Therefore, delayed water level readings were not obtained. See the Logs of Borings for the groundwater level.

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, rainfall and water levels within nearby creeks, ponds, and adjacent drainage ditches.

5.0 ANALYSIS AND RECOMMENDATIONS

5.1 SOIL MOVEMENTS

The subsurface exploration generally revealed the presence of deep active clay soils. The clay soils will have a moderate to high shrink/swell potential depending upon the soil moisture condition at the time of construction. Potential soil swell movement calculations were performed using swell test results, pocket penetrometer readings, and moisture content tests to estimate the swell potential of the soil. The potential soil swell movement values are based upon current soil moisture conditions and current grades at the test boring locations.

Potential soil swell heave within a typical 10 foot deep “active zone” has been estimated to be in the range of 2 to 3.5 inches beneath the existing pavement (outside of the tree influenced areas). However, within tree influenced areas, the potential soil swell heave has been estimated to range from 6 to 8 inches of “active zone” soil swell heave. Therefore, relatively large “active zone” differential movements of up to 6 inches are present between tree influenced areas and non-tree areas. Large differential pavement movements (up to 6 inches) are also possible between existing pavement areas and pavement widening areas especially if construction is performed after extended periods of dry weather (whereby the subgrade soils in the areas to be widened would be dry and highly expansive due to exposure to prolonged periods of dry weather).

In addition to swelling within a typical 10 foot deep “active zone”, the potential for additional “deep-seated” swell exists at this site. The assumed “active zone” swell values are upward soil movements that could occur due to typical seasonal moisture changes and soil swelling within the upper ten (10) feet as measured from finished floor grade. The deep-seated swell values are additional upward soil movements that could occur due to moisture changes and soil swelling below a typical ten (10) foot deep “active-zone”.

Deep-seated swell could occur due to groundwater fluctuations or free water sources such as ponding water conditions, percolation of water in landscaped areas, leaking sprinkler lines and/or leaking utility lines that are not detected and repaired in an expedient manner. At this site, the deep-seated swell is estimated to range between less than 1 inch to 3 inches. The risk of differential deep seated swell below pavements is generally not a high risk due to the low probability of deep water percolation below 10 foot depths but could occur if a free water source occurs over an extended period of time. Measures to minimize deep seated swell associated with free water sources are provided in Appendix A to this report.

5.2 OPTIONAL SITE MODIFICATION TO REDUCE SOIL MOVEMENTS

As mentioned above, differential upward pavement movements could occur at this site due to soil swelling. If this magnitude of differential pavement movement is not acceptable, site preparation work will have to be performed (especially within tree influenced areas and pavement widening areas) in order to lower the potential differential movements to acceptable levels in areas sensitive to differential movements. If it is required for the differential soil swell movement for the proposed new pavements to be reduced, moisture conditioning will be required. Moisture conditioning can be accomplished by excavation and moisture conditioning of the in-situ soils in compacted lifts. An Alliance Geotechnical Group Engineer should be contacted for site preparation work recommendations in order to reduce the soil swell movements to acceptable levels if it is desired to reduce differential upward pavement movements.

Note 1: Over-excavation will possibly cut large roots and thereby damage or kill the few existing trees along the alignment. Also, wherever moisture conditioning is performed near tree areas, the new pavement will be subject to large settlements due to soil shrinkage from tree root moisture absorption.

Note 2: See Section 6.9 regarding tree effects.

6.0 PAVEMENT RECOMMENDATIONS

The required pavement sections depend on the traffic volume and the frequency of heavy truck traffic. The pavement designs in this report are based upon City of Arlington's daily traffic volume (26,304,908 ESAL). The design ESAL of 26,304,908 was based on the total traffic volume for a 50-year design life. The design ESAL is based on the Average Daily Traffic (ADT) of 22,030 (See Appendix B). The traffic data was retrieved from the City of Arlington's Traffic Count website (See Appendix B). A 1.0% heavy duty traffic and 1.5% annual traffic growth rate was assumed to evaluate the design ESAL.

The heavy duty traffic comprises of 60% Single Unit trucks and 40% WB-50 trucks. The loading configuration for the WB-50 truck consist of 8 kips front single-axle and two 32 kips tandem-axles. The loading configuration for the single-unit truck consist of 8 kips front single-axle and 32 kips back single-axle.

The pavement section recommendations provided below were designed based upon AASHTO Guide for Design of Pavement Structures using WinPAS 12 computer program.

6.1 STABILIZATION WITH HYDRATED LIME / CEMENT

The subsurface exploration revealed surficial materials consisting of plastic clay soils having a moderate to high shrink/swell potential. These clay soils react with hydrated lime, which serves to improve their support value and provide a firm, uniform subgrade beneath the paving. We understand that the City of Arlington prefers 14 inches of lime stabilization followed by 12 inches of cement stabilized subgrade beneath their 120' Divided Roadway pavements. These recommendations are provided below.

Soluble sulfate testing (see Figure 13) resulted in sulfate levels ranging from 120 to 440 ppm on the upper soil layers. Soluble sulfate levels of up to 8,600 ppm were encountered within the deeper soils. These deeper high sulfate soils will possibly get mixed within the pavement subgrade during utility line installations and thereby cause a risk of sulfate induced heave. It should be noted that the clay and shaley clay soils of the Eagle Ford Formation are notorious for having very high soluble sulfates levels. **Due to the risk of sulfate induced heave, AGG doesn't recommend performing lime-stabilization at this site**.

A risk of sulfate heave occurs when sulfate levels are in excess of 3,000 ppm. If stabilization is performed, we recommend that additional soluble sulfate testing be performed on the pavement subgrade once final grading of the pavement has been achieved to verify that that sulfate levels are below 3,000 ppm. If the risk of sulfate induced heave is acceptable and lime-stabilization is to be considered for this project, we recommend that the following be performed.

Based upon the results of PI and lime series testing (see Figure 14), eleven (11) percent hydrated lime by dry weight (140 pounds per square yard per 14-inch depth) should be anticipated to stabilize the existing clay subgrade. The lime stabilization should be performed in two 7-inch lifts (70 pounds per square yard per 7-inch depth). The actual lime requirement will depend upon the actual subgrade soils exposed at final grade and should be determined at the time of construction along with sulfate content determinations.

After excavation is performed to a depth of 7 inches below pavement subgrade, lime should be thoroughly mixed and blended with the upper 7 inches at base of cut. After allowing for proper curing time and blended with the active subgrade soil (TxDOT Item 260), the mixture should be compacted to a minimum of 98 percent of maximum dry density as determined in accordance with ASTM D698, within -2% to +4% of the soil's optimum moisture content. After adequate curing, seven inches of native clay soils (with low sulfate contents) should be placed on top of the lime stabilized soils and should be lime stabilized as described above. We recommend that this lime stabilization extend 1 foot beyond exposed pavement edges, if possible, in order to reduce the effects of shrinkage during extended dry periods. Depth checks and PI verification checks should be performed on each lift 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 wet cured (mellowed) at a minimum of +5% above optimum moisture (or wetter) for 3 days prior to final mixing and compaction. We recommend a 3 day curing period for these soils. The subgrade must be kept wet by regular watering during the mellowing period. 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 to a total depth of 14 inches, the lime stabilized subgrade should then be cement stabilized to a depth of 12 inches. It is anticipated that eight (8) percent cement by dry weight (86 pounds per square yard per 12-inch depth) would be required for stabilization. As an alternative, the optimum percent of cement could be determined in accordance with TEX-120-E. A minimum strength of 200 pounds per square inch at 7 days should be achieved.

The cement should be thoroughly mixed and blended with the top 12 inches of the lime stabilized subgrade and the mixture compacted to a minimum of 95 percent of maximum dry density as determined in accordance with ASTM D698, within 2 percentage points of the soil's optimum moisture content. We recommend that stabilization extend to 1 foot beyond exposed pavement edges, if possible, in order to reduce the effects of drying 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 2 hour period. See TxDOT 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.

6.2 RECOMPACTED PAVEMENT SUBGRADE

In lieu of performing stabilization, the concrete pavement thickness should be increased by two (2) inches and placed directly on compacted subgrade soils. The upper 8 inches of subgrade soil should be compacted at -1% to +2% of optimum moisture to a minimum of 98% Standard Proctor density (ASTM D698). Prior to placing fill and subgrade compaction, the subgrade should be proofrolled. Proofrolling should be performed in accordance with Section 6.3 of this report.

Only on-site soil (comparable to the underlying subgrade soil) should be used for fine grading the pavement areas. After fine grading, the subgrade should again be watered if needed and re-compacted in order to re-achieve the moisture and density levels discussed above and provide a tight non-yielding subgrade.

Sand should not be allowed for use in fine grading the pavement areas. Sand should be specifically prohibited beneath pavement areas during final grading since these more porous soils can allow water in flow, resulting in heave and strength loss of subgrade soils. The subgrade moisture content and density must be maintained until paving is completed. The subgrade should be watered just prior to paving to assure concrete placement over a moist subgrade. If a rain event occurs prior to paving, the subgrade should be aerated and re-tested prior to paving.

6.3 PROOFROLLING AND FILL PLACEMENT

After the existing pavement is removed and prior to placing fill and/or performing lime stabilization, the exposed subgrade should be proof-rolled. Proof-rolling can generally be accomplished using a heavy (25 ton or greater total weight) pneumatic tired roller making several passes over the areas. The proof-rolling operations should be performed under the direction of an AGG Geotechnical Engineer. Where soft or compressible zones are encountered, these areas should be removed to a firm subgrade as determined by AGG. Any resulting void areas should be backfilled to finished subgrade in 8 inch compacted lifts as specified below.

After completion of proof-rolling, the ground surface should then be scarified to a depth of 8 inches and recompacted to a minimum of 98 percent of the maximum density as determined by ASTM D698 between -1% and +2% of its optimum moisture content.

6.4 PAVEMENT SECTION RECOMMENDATIONS

The pavement section recommendations provided below were designed based upon AASHTO Guide for Design of Pavement Structures using WinPAS 12 computer program and based upon design ESAL of 26,304,908. The pavement design was based on the City of City of Arlington's traffic counts.
A summary of the inputs are provided below:

Design ESAL 18-kip:	26,304,908*
Initial Serviceability:	4.5
Terminal Serviceability:	2.25
Modulus of Rupture:	588 psi (4,000 psi concrete)
Elasticity Modulus:	3,932,000 psi

Effective k-value:	300 psi/in–lime & cement stabilization
	50 psi/in – recompacted subgrade
Reliability Level:	85%
Standard Deviation:	0.39
Load Transfer J:	2.7 (adequate edge support)
Drainage Coefficient:	1.0

* *The design ESAL is based on the Average Daily Traffic (ADT) of 20,030. The traffic data was retrieved from the City of Arlington's Traffic Count website. The design ESAL is also based on 1% Heavy Duty Traffic and based on the assumption of 1.5% annual traffic growth rate.*

Based on the City of Arlington's 120' Divided Roadway standard design criteria, the pavement section would result in 9 inches of concrete pavement with a compressive strength of 3,600 psi at 28 days and the subgrade shall be mixed to a depth of 14 inches for lime stabilization and then cement shall be mixed to a depth of 12 inches. The City of Arlington's pavement standard would result in a design life of 23 years. AGG was requested to provide pavement recommendations for a 50-year design life for Turner Warnell Road. Table 1 presents the recommended pavement sections for the proposed pavement reconstructions for a 50-year design life.

TABLE 1 - RECOMMENDED PAVEMENT SECTIONS
TURNER WARNELL ROAD

26,304,908 18-kip ESAL
PCC SECTION
11 inches Portland Cement Concrete (4,000 psi Concrete) *
14 inches Lime Stabilized Subgrade (Section 6.1) and
12 inches Cement Stabilized Subgrade (Section 6.1)
OR
13 inches Portland Cement Concrete (4,000 psi Concrete) *
8 inches Recompacted Subgrade (Section 6.2)

**4,000 psi (at 28 days) for machine finish. For hand finish, increase concrete strength to 4,500 psi at 28 days. A minimum 28-day flexural strength of 588 psi.*

NOTE: The above pavement designs are based upon adequate pavement edge support being provided. If adequate pavement edge support is not provided, the above pavement thicknesses should be increased by one (1) inch.

Concrete quality will be important in order to produce the desired flexural strength and long term durability. We recommend that the concrete have 5% entrained air plus or minus 1%.

The concrete should be placed at a slump of 4 inches plus or minus 1 inch for hand pours and a slump of 2 inches plus or minus 1 inch for machine finish pours.

The sulfate levels are in excess of 3,000 ppm as shown by the results of the soluble sulfate testing. Therefore, sulfate resistant concrete mix designs utilizing fly ash should be considered for concrete placed directly on clay compacted subgrade. If crushed “Chico” stone flex base is used, sulfate-resistant concrete would not be required. The sulfate resistant mix design should include the type and amount of cement and the type and amount of fly ash proposed. Since Type V cement is not locally available, we recommend that a fly ash/cement mix design utilizing Type II cement and 25% Type F fly ash with a low C3A concentration and a maximum water/cement ratio of 0.45 (or an approved equal) be used for concrete in contact with these site soils due to its resistance to sulfate attack. We recommend that additional ACI requirements for Class 3 exposure (sulfates that exceed 20,000 ppm) be considered for implementation at this site. We recommend that these ACI guidelines be considered during design.

Proper joint placement and design is critical to pavement performance. Load transfer at all joints and maintenance of watertight joints should be provided. Control joints should be sawed as soon as possible after placing concrete and before shrinkage cracks occur. All joints including sawed joints should be properly cleaned and sealed as soon as possible to avoid infiltration of water.

Our previous experience indicates that joint spacing on 12 to 15 foot centers have generally performed satisfactorily. It is our recommendation that the concrete pavement be reinforced with a minimum of No. 4 bars placed on chairs on approximately 18–inch centers in each direction.

6.5 PAVEMENT CONSIDERATIONS

All joints and pavements should be inspected at regular intervals to ensure proper performance and to prevent crack propagation. The soils at the site are active and differential heave within the paving areas will occur. See Section 5.1 of this report. The service life of paving may be reduced due to water infiltration into subgrade soils through heave induced cracks in the paving section. This will result in softening and loss of strength of the subgrade soils. A regular maintenance program to seal paving cracks will help prolong the service life of the paving. The life of the pavement can be increased with proper drainage. Areas should be graded to prevent ponding adjacent to curbs or pavement edges. Granular backfill materials, which could hold water behind the curb, should not be permitted.

Compacted clay soils should be used behind the curb. Flat pavement grades should be avoided.

The site soils along the alignment are corrosive to both metals and to buried concrete. High sulfate levels should be anticipated due to sulfate concentrations within seams and pockets within this formation. Soluble sulfate testing (see Figure 13) resulted in sulfate levels as high as 8,600 ppm. All flowable fill, grout, and all below grade concrete in contact with the native sulfate rich soils including concrete piping should be designed for Class 3 exposure per ACI. A locally available sulfate resistant mix design for below grade concrete would include 25% Type F Flyash in combination with Type II cement. There are other ACI requirements where sulfate levels exceed 20,000 ppm.

6.6 SITE GRADING AND DRAINAGE

All grading should provide positive drainage away from the proposed roadways and should prevent water from collecting or discharging near the pavements. Water must not be permitted to pond adjacent to or near the pavements during or after construction. Otherwise, differential upward soil swell movements will be exacerbated.

Joints in the concrete pavements should be sealed to prevent the infiltration of water. Since post construction movement of pavement may occur, joints should be periodically inspected and resealed along with pavement cracks that will occur.

6.7 TREE EFFECTS

The roots of large mature trees can absorb large amounts of moisture from the supporting soils to depths of over 15 feet. The lateral limits of tree root influence extend at least 5 feet beyond the unpruned drip line (and to much greater distances when the ground beneath the drip lines is paved and/or if multiple trees are present in the area).

To reduce future settlement after reconstruction, root barriers and/or irrigated tree wells could be considered for any existing tree or any proposed new trees to be planted. An arborist or landscape architect should be contacted regarding the required depth of the root barrier and whether or not this is a viable solution. Root barriers and/or irrigated tree wells should be considered for existing small trees near the roadway and for new trees to be planted along the roadway.

Otherwise, a thickened concrete pavement section should be considered for use near tree influenced areas. An additional 1 to 2 inches of concrete (over the required design thickness) could be used near the tree areas to provide additional rigidity to reduce differential deflections caused by post construction shrink/swell movements. Additional steel reinforcement could be used to further stiffen the pavement. Larger bars on a closer spacing and two mats of steel should be considered. A Structural Engineer should be consulted regarding the most cost effective reinforcement design for the thickened sections.

If the pavement is thickened and stiffened as described above, differential deflections should be reduced. If differential settlements due to shrinkage caused by tree roots become objectionable, these areas could be mudjacked in the future as needed to level the pavement.

7.0 UTILITY LINE CONSTRUCTION

We understand that new water and wastewater lines will be constructed along the roadway during the pavement reconstruction. We understand that the invert depths of these lines will be 15 feet or less below final pavement grades. We understand that the new utility lines will be installed using open cut construction techniques during the pavement reconstruction operations.

7.1 TRENCH EXCAVATIONS

It is understood that open cut trench excavations will be performed for utility line construction along the proposed alignment. It is understood that the invert depths will generally be 15 feet or less below the existing ground surface. As indicated on the boring logs, soil conditions will vary significantly at the invert depths along the alignments. Subsurface conditions will vary from stiff to hard clay, sandy clay, clayey sand, fine sand, shaley clay and water bearing granular soils. The clay soils are typically jointed and fissured. The shaley clays are jointed and blocky. Therefore, sloughing should be anticipated during excavation and installation operation.

For trench excavations to <u>any depth</u> in unstable soil at this site, (sand, gravel, soft clay, existing fill and submerged soil) or where sloughing is observed, it will be necessary 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.

7.2 OPEN CUTS

Recommended slope ratios for the respective soil conditions are presented graphically on Figure 15. 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 equipment, should be placed no closer than five feet from the crest of the slope, or in accordance with OSHA regulations. Vehicle traffic should be maintained at least five feet from the edge of the crest.

Excavations may encounter non-compact alluvial granular soils and fill soils placed during previous construction of underground utilities. Where encountered, these fill soils should be sheeted, shored, and braced, or laid back on slopes no steeper than 1.5 (H):1(V) on short term basis (under 8 hours) and no steeper than 2(H):1(V) on long term basis. The contractor will need to take measures to avoid undermining and damaging existing underground utilities, sidewalks, driveways, etc.

Note1: Due to the potential for pressurized aquifer conditions (See Boring B-6) dewatering must be performed prior to the start of excavation to prevent disturbance and softening of the supporting soils below the pipe. The dewatering should be performed to a depth of at least 3 three (3) feet below the proposed bottom of the excavations in order to minimize softening of the supporting soils.

Note 2: Bottom “heave” (invert blowout) and/or softening of the foundation subgrade is likely in some areas where proper dewatering is not performed prior to the start of excavation due to groundwater levels that are (or will likely be) several feet above the proposed invert depths at the time of construction.

7.3 TRENCH BRACING / SHORING

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 16. 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.

7.4 DEWATERING

Groundwater seepage was generally not encountered during drilling the drilling operation. However, ground water was encountered at depths ranging from 13 to 18.4 feet below existing grade within two of the test borings (Borings B-6 and B-7). Shallower groundwater levels should be anticipated in all areas after periods of rainfall.

In areas where groundwater is encountered, a system of ditches, sumps, deep wells and/or dual staged 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 and/or existing adjacent utility trenches. Prior to excavation below groundwater levels, the utility alignment should be dewatered whereby the groundwater level is lowered to at least 3 feet below the required pipe bedding.

7.5 CONSTRUCTION CONSIDERATIONS

The following guidelines are presented to aid in 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 personnel.
- 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)

7.6 TRENCH BACKFILL PLACEMENT

The excavated soils can be used for trench backfill. The backfill should be placed in thin compacted lifts as specified below. The fill materials should be free of rock fragments greater than six (6) inches in any dimension and surficial vegetation or debris.

Clay fill soils should be placed in 8 inch horizontal loose lifts and compacted to a minimum of 95% ASTM D 698. The clay soils should be compacted at optimum to +3% above the optimum moisture content.

The clay, sandy clay and shaley clay soils should be placed in 8 inch horizontal loose lifts and compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698 test method. These materials should be compacted at optimum to +4% of optimum moisture. Very sandy clay and clayey sand should be compacted at -1% to +2% of optimum moisture.

Note 1: The upper eight (8) inches of pavement subgrade directly below the bottom of the proposed lime stabilized subgrade, flexbase or pavement should be compacted to a minimum of 98% ASTM D 698 at -1% to +2% of optimum moisture.

Note 2: For backfill soils placed below 10 feet, we recommend that the compaction be increased to a minimum of 98% ASTM D 698 at -1% to +2% of optimum moisture content.

Note 3: If the fill soils are placed in accordance with the recommendations contained within this report on compact natural subgrade soils, settlement / consolidation is estimated to be on the order of 1.0 percent of the fill height. This assumes that all fill is placed in accordance with recommendations contained in this report.

8.0 FIELD SUPERVISION AND CONSTRUCTION TESTING

Field density and moisture content determinations should be made on each lift of fill with a minimum of 1 test per lift per 100 linear feet for the roadway and a minimum of 1 test per lift per 150 linear feet for trench backfill. Supervision by the field technician and the project engineer is required. Some adjustments in the test frequencies may be required based upon the general fill types and soil conditions at the time of fill placement.

Many problems can be avoided or solved in the field if proper inspection and testing services are provided. It is recommended that site preparation, concrete placement, and fill compaction 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. AGG employs a group of experienced well-trained and certified engineering technicians to perform inspection and construction materials testing who would be pleased to assist you on this project.

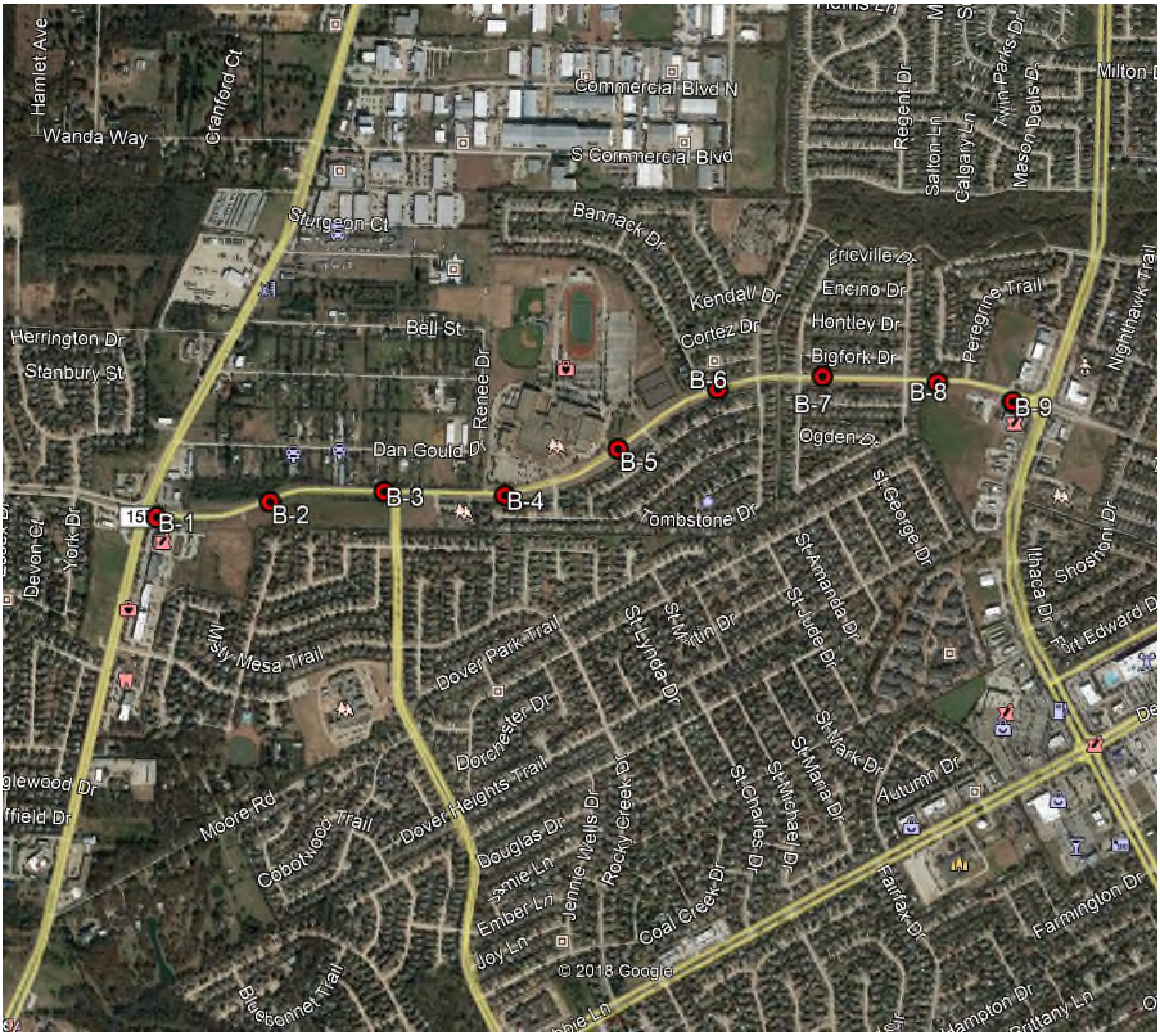
9.0 LIMITATIONS

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 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 client and their client and 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 final report. After this period, the samples will be discarded unless otherwise notified by the owner in writing.

FIGURES



ALLIANCE GEOTECHNICAL GROUP	Project No: DE19-335	**PLAN OF BORINGS**
	PAVEMENT RECONSTRUCTION TURNER WARNELL ROAD ARLINGTON, TEXAS	Figure No: **1**

LOG OF BORING B-1

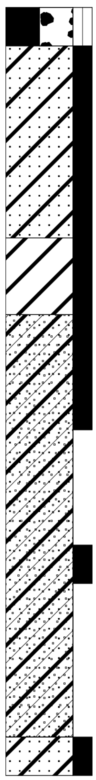
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: 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)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	2" ASPHALT over 10" Crushed STONE BASE									
	Dark brown sandy CLAY w/ iron nodules	20	44	14	30			2.25		
								3.5		
								3.25		
5								3.25		
	Brown and tan CLAY, jointed, fissured, w/ calcareous nodules	18						4.5		
								3.5		
	Tan and light gray very sandy CLAY to clayey SAND							4.5+		
		12				52	119	4.5++		
10								4.5++		
15		10	23	12	11	42				
	Brown and tan very sandy CLAY w/ calcareous nodules	12	32	14	18			4.5++		
20	Boring terminated at 20'									
25										
30										
35										

Notes:

FIGURE:2

Alliance Geotechnical Group, Inc.

LOG OF BORING B-2

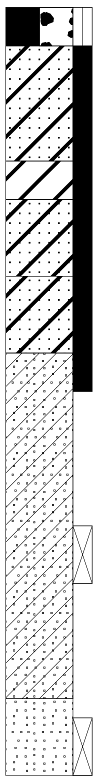
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: 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 %
0		2" ASPHALT over 10" Crushed STONE BASE									
		Dark gray sandy CLAY w/ calcareous nodules and iron nodules	21	46	15	31			3.25		
									3.25		
									3.0		
		Brown CLAY, jointed, w/ calcareous nodules	20					109	4.25		
5		Tan and light gray sandy CLAY w/ calcareous deposits							3.0		
									3.75		
		Tan and light gray very sandy CLAY							3.25		
			19					86	2.5	1.8	7.1
10		Reddish brown clayey SAND w/ clay layers									
15	6/6" 9/6" 15/6"		15								
	8/6" 10/6" 16/6"	Tan fine SAND w/ clay seams									
20		Boring terminated at 20'									
25											
30											
35											

Notes: **FIGURE:3**

LOG OF BORING B-3

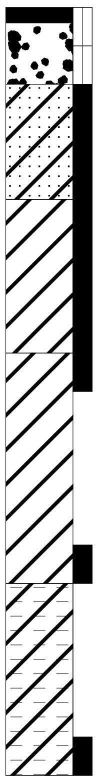
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: 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)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	3.5" ASPHALT									
	Crushed STONE (FILL)									
	Dark gray sandy CLAY w/ calcareous nodules and iron nodules	17	44	16	28			2.75		
								2.0		
								2.5		
5	Light gray and tan CLAY, jointed, fissured, w/ calcareous nodules							2.5		
								4.5+		
		21					108	4.5+		
								4.5+		
10	Tan and light gray CLAY, jointed, fissured							4.5+		
								4.5+		
15	Tan and brown shaley CLAY, jointed, blocky									
		22						4.5+		
20	Boring terminated at 20'									
25										
30										
35										

Notes:

FIGURE:4

Alliance Geotechnical Group, Inc.

LOG OF BORING B-4

Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: 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)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	4" ASPHALT over Crushed STONE BASE									
	Dark gray sandy CLAY w/ calcareous nodules and iron nodules	20	41	14	27		108	4.0 4.5+		
								3.75		
								3.5		
5								3.25		
	Dark gray and tan CLAY, jointed, fissured, w/ calcareous nodules	22				84		3.0		
								3.0		
	Tan and light gray CLAY, jointed, w/ gypsum crystals							3.75		
								3.0		
10										
		24						4.0		
15										
		29						3.5		
20	Boring terminated at 20'									
25										
30										
35										

Notes: **FIGURE:5**

LOG OF BORING B-5

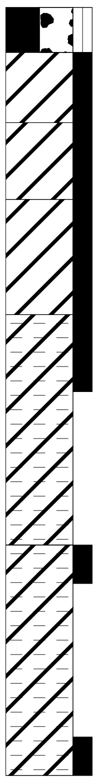
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/06/2019** Elev.: 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 %
0		3" ASPHALT over Crushed STONE BASE									
		Dark brown CLAY, jointed, fissured, w/ calcareous nodules	25	62	19	43			3.0 2.75		
		Brown and tan CLAY, jointed, fissured, w/ calcareous nodules							2.5 2.5		
5		Tan and light gray CLAY, jointed, fissured, w/ gypsum crystals	31	79	27	52			2.5 2.5 2.75		
		Tan and light gray shaley CLAY, jointed, blocky							3.0 3.75		
10											
15		Gray and tan shaley CLAY, jointed, blocky w/ gypsum crystals	24					101	4.5+ 4.5+		
20		Boring terminated at 20'									
25											
30											
35											

Notes:

FIGURE:6

LOG OF BORING B-6

Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: Location: **See Figure 1**

Depth to water at completion of boring: **18.2'**

Depth to water when checked: **during drilling** was: **13'**

Depth to caving when checked: was:

DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
3" ASPHALT over Crushed STONE BASE									
Dark gray CLAY, jointed, fissured, w/ calcareous nodules and iron nodules	24	54	18	36			4.0		
							3.0		
							3.0		
Brown and tan CLAY, jointed, fissured, w/ calcareous nodules							3.0		
							3.5		
	24						3.5		
Tan and light gray CLAY, jointed, fissured, w/ calcareous nodules and gypsum crystals							2.0		
	26				95	102	3.0	3.3	10.1
Light gray and tan very sandy CLAY to clayey SAND w/ iron fragments and layers	27						1.0		
-seepage at 13' during drilling									
							3.5		
Boring terminated at 20'									

4/6"
8/6"
8/6"

Notes:

FIGURE:7

LOG OF BORING B-7

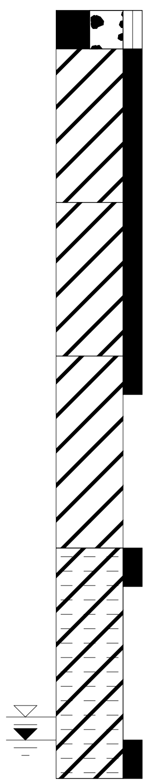
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/05/2019** Elev.: Location: **See Figure 1**

Depth to water at completion of boring: **18.4'**

Depth to water when checked: **during drilling** was: **19'**

Depth to caving when checked: was:



ELEVATION/ DEPTH (feet)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	3" ASPHALT over Crushed STONE BASE									
	Dark gray CLAY, jointed, fissured, w/ calcareous nodules							3.0		
		35	73	25	48			2.75		
								3.25		
		34	74	25	49		88	2.5		
5	Dark gray and tan CLAY, jointed, fissured, w/ calcareous nodules							2.25		
		32				97		2.75		
								2.5		
								2.5		
	Tan and light gray CLAY, jointed, fissured, w/ gypsum crystals							3.25		
10										
	Tan and light gray shaley CLAY, jointed, blocky, w/ gypsum crystals							3.25		
15										
	-seepage at 19' during drilling	16						4.5+		
20	Boring terminated at 20'									
25										
30										
35										

Notes:

FIGURE:8

LOG OF BORING B-8

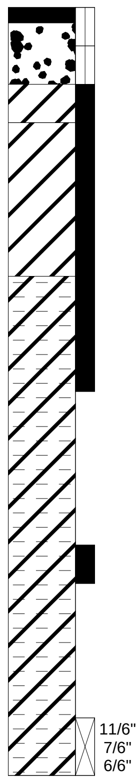
Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/06/2019** Elev.: 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)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	2" ASPHALT									
	Crushed STONE (FILL)							3.5		
	Dark gray CLAY, jointed, fissured, w/ calcareous nodules	30	69	24	45			4.5+		
	Brown and tan CLAY, jointed, fissured, w/ calcareous nodules	27					98	3.0		
								2.75		
5								2.75		
								3.5		
	Tan and light gray shaley CLAY, jointed, blocky							4.25		
								4.5		
		30	77	28	49			4.5+		
10										
15		29					97	4.5+		
		30						3.75		
20	Boring terminated at 20'									
25										
30										
35										

Notes:

FIGURE:9

LOG OF BORING B-9

Project: **Turner Warnell Road Pavement Reconstruction - Arlington, TX** Project No.: **DE19-335**

Date: **09/06/2019** Elev.: 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)	DESCRIPTION	MC %	LL %	PL %	PI %	-200 %	DD pcf	P.PEN tsf	UNCON ksf	Strain %
0	3" ASPHALT over Crushed STONE BASE									
	Dark brown CLAY, jointed, fissured, w/ calcareous nodules	30	69	24	45			3.25		
								3.5		
	Brown CLAY, jointed, fissured, w/ calcareous nodules							3.5		
								3.5		
5								3.0		
		31	82	30	52			3.0		
	Tan and light gray CLAY, jointed, fissured, w/ calcareous nodules							3.25		
		25					97	4.5+		
								4.25		
10										
	Tan and light gray shaley CLAY, jointed, blocky	32						4.5+		
15										
								4.5+		
20	Boring terminated at 20'									
25										
30										
35										

Notes:

FIGURE:10

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KEY TO LOG TERMS & SYMBOLS

Symbol	Description
Strata symbols	
	Asphaltic Paving
	Crushed STONE BASE
	CLAY, sandy
	CLAY
	Sandy CLAY, Clayey SAND
	SAND, clayey
	SAND
	CLAY, shaley
Misc. Symbols	
	Water table when checked

Symbol	Description
	Water table at boring completion
Soil Samplers	
	Rock Core
	Thin Wall Shelby Tube
	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)	LL = liquid limit (%)
MC = natural moisture content (%)	PL = plastic limit (%)
Uncon.= unconfined compression (tsf)	PI = plasticity index
P.Pen.= hand penetrometer (tsf)	-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

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	9-10	119.0	-	-	-	12.4	12.8	1,188	0.2
B-2	4-5	109.1	-	-	-	19.6	22.7	563	3.0
B-3	7-8	107.6	-	-	-	20.9	23.5	938	3.1
B-4	2-3	107.7	41	14	27	20.4	21.5	313	1.0
B-5	14-15	100.7	-	-	-	24.3	25.5	1,813	1.3
B-7	4-5	88.0	74	25	49	34.1	35.2	563	1.0
* B-7	4-5	98.4*	74	25	49	26.6*	31.1	563	5.6*
B-8	3-4	97.7	-	-	-	26.9	28.4	438	1.5
* B-8	3-4	106.4*	-	-	-	21.8*	26.4	438	5.8*
B-9	8-9	97.1	-	-	-	25.3	27.5	1,063	2.3

* The sample was allowed to air dry prior to performing the swell test in order to simulate dry soil conditions.

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.

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SWELL TEST RESULTS		
TURNER WARNELL ROAD PAVEMENT RECONSTRUCTION		
ARLINGTON, TEXAS		
ALLIANCE GEOTECHNICAL GROUP		
DE19-335	Date: 09/19/2019	FIGURE: 12

SOLUBLE SULFATES TEST RESULTS (PPM)

BORING NO.	DEPTH (FT.)	SOLUBLE SULFATES (PPM)
B-1	2-3	160
B-2	1-2	120
B-3	2-3	120
B-4	2-3	200
B-4	14-15	860
B-5	2-3	240
B-6	2-3	140
B-6	9-10	8,600
B-7	2-3	440
B-8	2-3	140
B-9	14”-3’	140

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SOLUBLE SULFATES TEST RESULTS		
TURNER WARNELL ROAD PAVEMENT RECONSTRUCTION		
ARLINGTON, TEXAS		
ALLIANCE GEOTECHNICAL GROUP		
DE19-335	DATE: 09/19/2019	FIGURE: 13

LIME SERIES RESULTS

BORING NO.	DEPTH (FEET)	LIME ADDED (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (PI)
B-5	2-3	0%	62	43
		5%	52	23
		7%	51	19
		9%	49	16
B-7	2-3	0%	73	48
		6%	55	21
		8%	53	18
		10%	50	14

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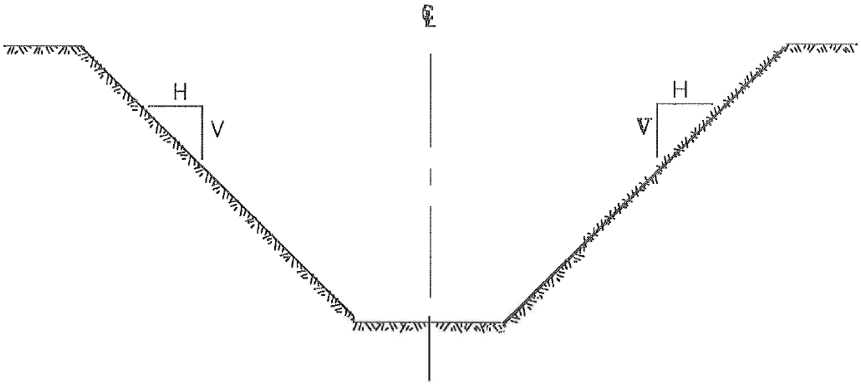
LIME SERIES TEST RESULTS		
TURNER WARNELL ROAD PAVEMENT RECONSTRUCTION		
ARLINGTON, TEXAS		
ALLIANCE GEOTECHNICAL GROUP		
DE19-335	Date: 09/19/2019	FIGURE 14

RECOMMENDED SLOPE RATIOS*

SOIL / ROCK	Short Term (under 8 hours) H	Short Term (under 8 hours) V	Long Term (over 8 hours) H	Long Term (over 8 hours) V
Fill soils, sand, gravel, clayey sand, very sandy clay, and/or soft clayey soils (hand penetrometer less than 0.9 tsf)	1.5	1	2	1
Submerged soils from which water is seeping **	1.5	1	2	1
Stiff to hard sandy clay, clay and shaley clay <u>above</u> existing groundwater level	1	1	1	1



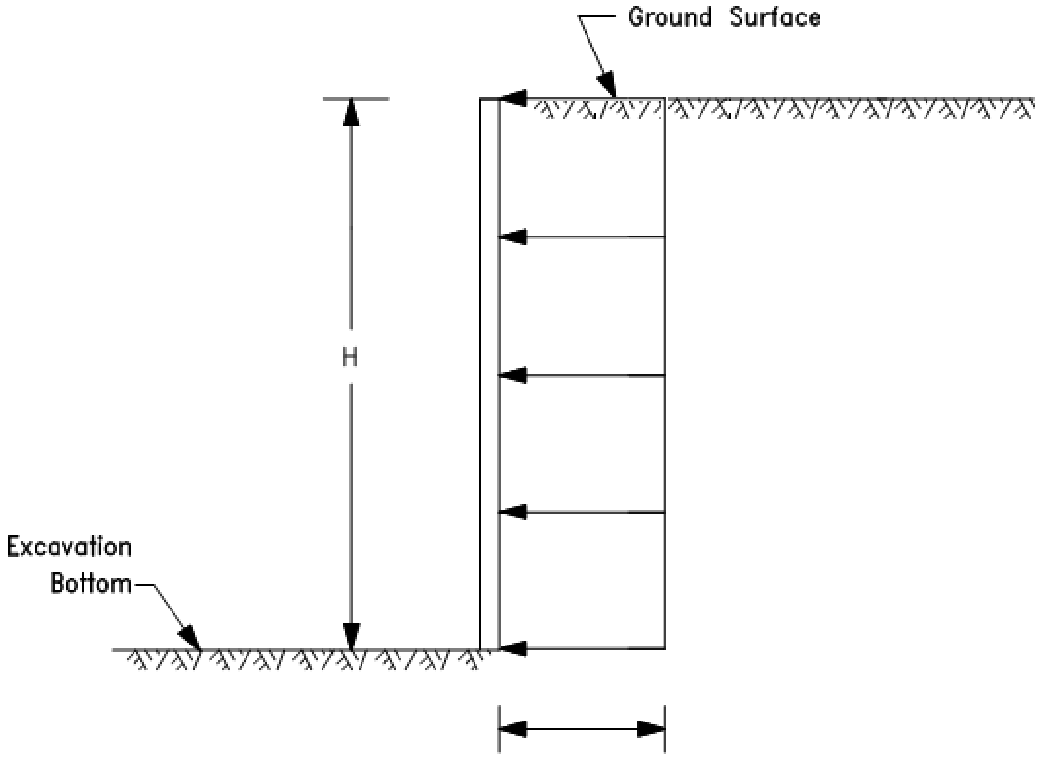
* For excavations deeper than 20', additional studies should be performed per OSHA to determine specific requirements and slope ratios for excavations deeper than 20 feet.

** In accordance with the best interpretation of OSHA regulations, submerged soil is defined as water bearing granular soils, fissured clay soils, or fractured rock (jointed weathered clay-shale, 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	PAVEMENT RECONSTRUCTION TURNER WARNELL ROAD	RECOMMENDED SLOPE RATIOS	FIGURE 15
	DATE: 9-19-19	PROJECT NO: DE19-335	

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
- H = Height of Excavation, ft.
- k = Earth Pressure Coefficient;
  Use 0.40 for existing fill soils, clayey sand, sand and very sandy clay.
  Use 0.35 for Sandy Clay, Clay and Shaley 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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PAVEMENT RECONSTRUCTION TURNER WARNELL ROAD	LATERAL EARTH PRESSURES	Figure 16
DATE: 9-19-19	PROJECT NO: DE19-335	

APPENDIX A

MEASURES TO MINIMIZE DEEP-SEATED SWELL

MEASURES TO MINIMIZE DEEP SEATED SWELL

In order to reduce the risk of excessive upward ground movements caused by soil swelling associated with free water sources, the following measures should be taken during design and construction:

- The use of superior contractors and utility line materials accompanied with Quality Control inspection and testing of all utility line installations.

- Utility under-drains with impervious barriers along the trench bottom may be used as an additional safeguard to minimize post-construction upward movement caused by water percolation into the deeper clay soils.

- Positive drainage should be provided. Surface drainage gradients within 10 feet of the pavement should be constructed with maximum slopes allowed by local codes.

- Rapid repair of any utility leak including water lines, sewer lines, and storm drains.

- Trees and deep rooted shrubs should be located no closer to the pavement than their ultimate mature height (and to greater distances were multiple trees are present and/or when the ground beneath the drip lines are paved) to reduce foundation settlement effects caused by moisture absorption of the root systems.

- It is imperative that all cracks and joints in the pavement be sealed and maintained by routine sealing in order to minimize differential pavement deflections caused by soil swelling.

- It is important that porous fill soils (sandy soil) <u>not</u> be used as backfill behind the curbs or as leveling sand below pavements to prevent ponding beneath the pavement or near the curb line.