DATE: December 19, 2022
TO: Potential Respondents
FROM: Carrie Stoeckert—Senior Construction Contract Coordinator
SUBJECT: Questions #2
RFCSP752-23-261209CS
UNT Advanced Air Mobility (UAAM) Test Center

This document is being issued to answer the following question that have been submitted as follows:

1. Please provide the site-specific geotechnical report.
   a. See attached document.
Geotechnical Engineering Report

UNT Outdoor Testing Facility
Denton, Texas

June 9, 2022
June 9, 2022

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GEOTECHNICAL INVESTIGATION
D&S ENGINEERING #G22-2040
UNT OUTDOOR TESTING FACILITY
DENTON, TEXAS

Ms. Nguyen,

As requested, D&S Engineering Labs, LLC has completed the Geotechnical Investigation for the above referenced project. This investigation was conducted in accordance with Proposal No. GP20-2040 dated April 6, 2022. Authorization to proceed was received on April 15, 2022.

We appreciate the opportunity to provide professional geotechnical engineering services to you. We are available to discuss any questions which may arise regarding this report. Please do not hesitate to call when we can provide any additional services.

Sincerely,

D&S Engineering Labs, LLC

[Signature]
Douglas Greenwood, P.E.
Senior Geotechnical Engineer

[Signature]
Ibrahim Baayeh, P.E.
Director of Geotechnical Engineering

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# TABLE OF CONTENTS

1.0 PROJECT DESCRIPTION ................................................................................................ 1

2.0 PURPOSE AND SCOPE................................................................................................... 1

3.0 FIELD AND LABORATORY INVESTIGATION .................................................................. 2

   3.1 General .......................................................................................................................  2

   3.2 Laboratory Testing....................................................................................................... 3

      3.2.1 Overburden Swell Tests...................................................................................... 4

      3.2.2 Soluble Sulfates.................................................................................................. 4

4.0 SITE CONDITIONS ........................................................................................................... 4

   4.1 Stratigraphy .................................................................................................................. 4

   4.2 Groundwater................................................................................................................ 5

5.0 ENGINEERING ANALYSIS ............................................................................................ 6

   5.1 Estimated Potential Vertical Movement (PVM) ............................................................ 6

6.0 FOUNDATION RECOMMENDATIONS .......................................................................... 6

   6.1 Straight-sided Drilled Shafts ..................................................................................... 6

      6.1.1 Drilled Shaft Construction Considerations......................................................... 8

      6.1.2 Lateral Loads ...................................................................................................... 9

   6.2 Shallow Foundations ................................................................................................. 9

7.0 EARTHWORK RECOMMENDATIONS .......................................................................... 10

   7.1 Subgrade Preparation for Soil Supported Elements.................................................... 11

   7.2 Additional Considerations ....................................................................................... 12

8.0 PAVEMENT RECOMMENDATIONS ............................................................................ 12

   8.1 General ...................................................................................................................... 12

   8.2 Behavior Characteristics of Expansive Soils Beneath Pavement ................................. 12

   8.3 Subgrade Strength Characteristics .......................................................................... 13

   8.4 Pavement Subgrade Preparation Recommendations ................................................. 13

      8.4.1 General.............................................................................................................. 13

      8.4.2 Aggregate Base ................................................................................................. 15

   8.5 Rigid Pavement ......................................................................................................... 15

      8.5.1 Pavement Joints and Cutting ............................................................................. 16

      8.5.2 Pavement Reinforcing Steel .............................................................................. 16

      8.5.3 Surface Drainage .............................................................................................. 17

   8.6 Flexible Pavement Design and Recommendations ..................................................... 17

      8.6.1 Full Depth HMAC ............................................................................................ 17

9.0 OTHER CONSTRUCTION ............................................................................................. 17
9.1 Utility and Service Lines .............................................................................................17
9.2 Exterior Flatwork ........................................................................................................18
9.3 Surface Drainage .......................................................................................................18
9.4 Landscaping ...............................................................................................................19
9.5 Excavations ................................................................................................................19
10.0 LIMITATIONS............................................................................................................20

APPENDIX A – BORING LOGS AND SUPPORTING DATA
APPENDIX B – GENERAL DESCRIPTION OF PROCEDURES
1.0 PROJECT DESCRIPTION

This report presents the results of the geotechnical investigation conducted for the proposed new outdoor UAS net test facility. The new facility will be located on the northern side of UNT Discovery Park campus, situated at 3940 N Elm Street in Denton, Texas.

Based on the provided information, we understand the proposed UAS facility will have an approximate footprint of about 36,000 square feet and will be constructed utilizing wire-supported 80-foot pole structures. Additionally, pavements are planned within the proposed facility. Considering the subsurface conditions encountered in the borings conducted for this project, and the type of structure, we anticipate that the pole structures will be supported utilizing drilled-shaft foundations, while the guy-wire anchor supports will be founded on either drill-shafts or shallow, ground supported footing foundations.

At the time of this investigation, the project site was occupied by short manicured grasses and few mature trees. Based on visual site observations and available topographic maps from the NCTCOG website (dfwmaps.com), the overall project site appears to be relatively level with an overall topographic relief of about 2 to 4 feet within the limits of proposed construction. Photographs showing recent site conditions are shown below.

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to:

- Identify the subsurface stratigraphy and groundwater conditions present at the project site.
- Evaluate the physical and engineering properties of the subsurface soil and bedrock strata for use in the geotechnical analyses.
• Provide geotechnical recommendations for use in design of foundations and site pavements for the proposed UAS test facility.

The scope of this investigation consisted of:

• Drilling and sampling a total of four (4) borings within the footprint of the proposed test facility project to depths of about 35 feet.

• Laboratory testing of selected soil and bedrock samples obtained during the field investigation.

• Preparation of a Geotechnical Report that includes the following:
  o Evaluation of Potential Vertical Movement (PVM).
  o Recommendations for the design and construction of foundations.
  o Recommendations for pavement sections and pavement subgrade stabilization.
  o Recommendations for earthwork.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 General

The borings were advanced utilizing truck-mounted drilling equipment outfitted with continuous hollow stem flight augers. Undisturbed samples of cohesive soils and weathered bedrock strata were obtained using 3-inch diameter tube samplers, which were advanced into the soils in 1-foot increments by the continuous thrust of a hydraulic ram located on the drilling equipment. After sample extrusion, a hand penetrometer measurement was performed on each cohesive soil to provide an estimate of soil stiffness.

Granular materials were sampled in general accordance with the Standard Penetration Test (ASTM D1586). During this test, a disturbed sample of subsurface material is recovered using a nominal 2-inch O.D. split-barrel sampler. The sampler is driven into the soil strata utilizing the energy equivalent of a 140-pound hammer falling freely from the height of 30 inches and striking an anvil located at the top of the drill string. The number of blows required to advance the sampler in three consecutive 6-inch increments is recorded, and the number of blows required for the final 12 inches is noted as the "N"-value. The test is terminated at the first occurrence of either of the following: 1) when sampler has advanced a total of 18 inches; 2) when the sampler has advanced less than one complete 6-inch increment after 50 blows of the hammer; 3) when the total number of blows reaches 100; 4) if there is no advancement of the sampler in any 10-blow intervals.

Soil and bedrock materials were also intermittently tested in-situ using cone penetration tests in order to determine their resistance to penetration. For this test, a
3-inch diameter steel cone is driven by the energy of a 170-pound hammer falling freely from a height of 24 inches and striking an anvil located at the top of the drill string. Depending on the resistance of the soil and bedrock materials, either the number of blows of the hammer required to provide 12 inches of penetration is recorded (as two increments of 6 inches each), or the inches of penetration of the cone resulting from 100 blows of the hammer are recorded (as two increments of 50 blows each).

All samples obtained were extruded in the field, placed in plastic bags to minimize changes in the natural moisture condition, labeled according to the appropriate boring number and depth, and placed in protective cardboard boxes for transportation to the laboratory. The approximate locations of the borings performed at the site are shown on the boring location map that is included in Appendix A. The surface elevations shown on the boring logs were interpolated from topographic maps available from the NCTCOG website (dfwmaps.com), which provides elevations in 2-foot intervals. The specific depths, thicknesses and descriptions of the strata encountered are presented on the individual Boring Log illustrations, which are also included in Appendix A. Strata boundaries shown on the boring logs are approximate.

### 3.2 Laboratory Testing

Laboratory tests were performed to identify the relevant engineering characteristics of the subsurface materials encountered and to provide data for developing engineering design parameters. The subsurface materials recovered during the field exploration were initially logged by the drill crew and were later described by a Staff Engineer in the laboratory. These descriptions were later refined by a Geotechnical Engineer based on results of the laboratory tests performed. All recovered soil samples were classified and described in part using the Unified Soil Classification System (USCS) and other accepted procedures. Bedrock strata were described using standard geologic nomenclature.

In order to determine soil characteristics and to aid in classifying the soils, index property and classification testing was performed on selected soil samples as requested by the Geotechnical Engineer. These index property and classification tests were performed in general accordance with the following ASTM testing standards:

- Moisture Content
- Atterberg Limits
- Percent of Particles Finer than No. 200 sieve

Additional tests were performed to aid in evaluating chemical and volume change characteristics which consisted of the following:

- Soluble Sulfates
• Overburden Swell Testing

The results of these tests are presented at the corresponding sample depths on the appropriate Boring Log illustrations. The index property and classification testing procedures are described in more detail in Appendix B.

3.2.1 Overburden Swell Tests

Selected samples of the near-surface soils were subjected to overburden swell testing. For this test, a sample is placed in a consolidometer and subjected to the estimated overburden pressure. The sample is then inundated with water and is allowed to swell. The moisture content of the sample is determined both before and after completion of the test. Test results are recorded, including the percent swell and the initial and final moisture contents.

3.2.2 Soluble Sulfates

Soluble sulfate tests were performed on representative samples obtained. These results are provided in Appendix A (Boring Logs and Supporting Data). Subgrade materials in some areas of Texas have experienced sulfate-induced heave after treatment with calcium-based additives such as lime. In general, a sulfate level less than 3,000 ppm is considered to have an acceptably low potential for sulfate induced heaving. The results of the sulfate tests performed on representative near-surface soil samples from test borings in this study indicate values of about 100 ppm or less, and thus should be considered to pose a minimal risk of sulfate-induced heaving after lime treatment.

4.0 SITE CONDITIONS

4.1 Stratigraphy

Based upon a review of the recovered samples, as well as the Geologic Atlas of Texas, Sherman Sheet, this site is characterized by soil and bedrock strata associated with the Grayson Marl and Main Street Limestone, undivided. This formation generally consists of clay soils overlying limestone and calcareous shale (locally referred to as Marl) bedrock.

Lean clay soils were encountered at the surface of all four borings conducted for this project. These lean clay soils were very stiff in consistency, various shades of brown in color, and contained varying concentrations of iron oxide stains, sand, rock fragments, and ferrous nodules.

The lean clay soils were underlain by clayey sand soils at a depth of about 1-foot below existing grades in Borings B1, B3, and B4. The clayey sand soils were medium dense to dense in condition, varying shades of orange, red, brown, and gray in color, were fine grained, and contained varying amounts of silt. The clayey sand soils continued to depths of about 3 to 5 feet below existing grades.
Sandy clay soils were encountered underlying the clayey sands at a depth of about 5 feet in Boring B4. The sandy clay soils were very stiff in consistency, gray and brown in color, and contained trace to few ferrous nodules, sand seams and laminations, and gravel. The sandy clay soils continued to a depth of about 9 feet below existing grades.

Weathered shale bedrock strata were encountered below the clayey sands and sandy lean clay soils in Borings B2 through B4 at depths of about 4.5 to 9 feet. The weathered shale was very soft to medium hard in rock hardness classification, tan and reddish-brown in color, fossiliferous in nature, and contained varying concentrations of interbedded sand seams. The weathered shale strata continued to depths of 13 to 15 feet below existing grades.

Weathered limestone bedrock was encountered below the overburden soils and weathered shale at depths of about 3 to 18 feet below existing grades. The weathered limestone was hard in rock hardness, tan to reddish-brown in appearance, and contained varying concentrations of sand seams. Fresh limestone bedrock was encountered underlyng the weathered limestone in Borings B1 and B4. The fresh limestone was gray in appearance, hard in rock hardness, and continued to depths of about 20 to 22 feet below existing grades.

Fresh shale bedrock was encountered underlying the limestone strata in all four borings at depths of about 18 to 23.5 feet below existing grades. The fresh shale rock strata were dark gray in color, soft to hard in rock hardness, and calcareous in nature. The fresh shale bedrock continued through the termination depth of about 35 feet in all four borings.

4.2 Groundwater

Groundwater seepage was observed during drilling operations at depths of 15 to 18 feet within Borings B2 and B3, and was measured within those same borings at depths of 20 to 30 feet upon completion of drilling activities. Groundwater seepage was not encountered within the remaining borings during or at the completion of drilling operations. Groundwater is often contained within the joints, fractures and other rock mass defects present in bedrock strata. When intercepted, these defects can produce appreciable amounts of water for a period of time, especially if those defects are extensive and well inter-connected. Groundwater levels may be anticipated to fluctuate with seasonal and annual variations in rainfall, and also may vary as a result of development and landscape irrigation.
5.0 ENGINEERING ANALYSIS

5.1 Estimated Potential Vertical Movement (PVM)

Potential Vertical Movement (PVM) was evaluated utilizing several different methods for predicting movement, as described in Appendix B, and based on our experience and professional opinion.

At the time of our field investigation, the near-surface soils were generally found to be dry to wet in moisture condition. Based upon the results of our analysis, the site is estimated to possess a PVM of about 1 to 1.5 inches at the soil moisture conditions existing at the time of the field investigation. If the near surface soils are allowed to dry appreciably to significant depth prior to or during construction, the potential for post-construction vertical movement may increase. Please note that dry, average, and wet are relative terms based on moisture content and plasticity.

6.0 FOUNDATION RECOMMENDATIONS

The near-surface soils present at the site have low potential for post-construction vertical movement with changes in soil moisture content. Based on the provided project information, it is our opinion that the pole structures and guy-wire anchor supports may be supported on drilled straight-shaft foundations founded in the fresh limestone or shale bedrock. Alternatively, the guywire supports may be anchored utilizing buried concrete deadman anchors. The deadman anchors may be designed as shallow, ground supported foundations.

Please note that a shallow foundation system may experience some vertical movement with changes in soil moisture content. However, with appropriate design, adherence to good construction practices and appropriate post-construction maintenance, these potential movements can be reduced.

Recommendations for deep and shallow foundation systems are presented in the following sections.

6.1 Straight-sided Drilled Shafts

Structural loads for the new tower structures and guy-wire supports may be supported on auger-excavated, straight-sided, reinforced concrete drilled shafts. Drilled shaft foundations may be founded in the moderately hard to hard weathered and/or fresh limestone bedrock, encountered at depths of about 3 to 18 feet below existing grades, or the fresh shale bedrock encountered at depths of about 18 to 23 feet below existing site grades. Straight-sided drilled shafts may be designed to transfer imposed loads into the bearing stratum using a combination of end-bearing and skin friction. Drilled shafts should be designed for an allowable end bearing and side friction as outlined in Table 1 below.
We recommend that straight-sided drilled piers for structural loads be a minimum of 18-inches in diameter. Piers should be at least 15 feet in length and penetrate a minimum of 1 foot into the limestone strata, or 3 feet into the fresh shale bedrock strata for full development of end bearing capacity. As there is appreciable strain-compatibility between the rock strata, the side friction for the weathered shale and limestone strata may be included in the shaft design for shafts extending into the fresh limestone and/or shale strata. Drilled shafts utilized for guy-wire supports may be designed as micro piles, with a minimum diameter of 12 inches, and a minimum penetration of 10 feet below existing grades.

The allowable side frictions noted in Table 1 may be taken from a minimum penetration of 3 feet into the bearing strata, from a depth of 8 feet below existing or finished grades, or from the bottom of any temporary casing used, whichever is deeper, to resist both axial loading and uplift. The strata depths provided in the table below represent approximate ranges of depths encountered across the site as a whole and should be used as general reference only; final design and construction of individual pier shafts should be based on the site-specific subsurface conditions encountered in the field during installation.

<table>
<thead>
<tr>
<th>Material</th>
<th>Depth Below Current Grades (ft)</th>
<th>Allowable End Bearing (psf)</th>
<th>Allowable Side Friction (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Shale</td>
<td>8-13</td>
<td>NA</td>
<td>1,800</td>
</tr>
<tr>
<td>Limestone/Shale</td>
<td>13-15</td>
<td>NA</td>
<td>4,500</td>
</tr>
<tr>
<td>Limestone/Shale</td>
<td>15+</td>
<td>35,000</td>
<td>4,500</td>
</tr>
</tbody>
</table>

The shafts should be provided with sufficient steel reinforcement throughout their length to resist potential uplift pressures that will be exerted. For the near surface soils, these pressures are estimated to be approximately 750 pounds per square foot (psf) of shaft area over an average depth of 8 feet within the overburden soils and weathered shale. Typically, one-half (½) of a percent of steel by cross-sectional area is sufficient for this purpose (ACI 318). However, the final amount of reinforcement required should be determined based on the information provided herein, and should be the greater of that determination, or ACI 318. Uplift forces acting on individual shafts will be resisted by the dead weight of the structure, plus the bearing stratum-to-concrete adhesion acting on that portion of the shaft from a minimum penetration of 3 feet into the weathered shale strata, from a depth of 8 feet below existing or finished grades, or from the bottom of any temporary casing used, whichever is deeper.

There is no reduction in allowable capacities for shafts in proximity to each other. However, for a two-shaft system, there is an 18 percent reduction in the available perimeter area for side friction capacity for shafts in contact (tangent). The area
reduction can be extrapolated linearly to zero at one shaft diameter clear spacing. Please contact this office if other close proximity geometries need to be considered.

We anticipate that a straight-side drilled pier foundation system designed and constructed in accordance with the information provided in this report will have a factor of safety in excess of 3 against shear failure and may experience settlements of small fractions of an inch. Where applicable, a lower factor of safety value of 2 may be applied when parts of the loads are temporary or transient such as wind, earthquake or ice loads.

### 6.1.1 Drilled Shaft Construction Considerations

Groundwater seepage was observed during drilling operations at depths of 15 to 18 feet within Borings B2 and B3 and observed in those same borings upon completion of drilling activities at depths of 20 to 30 feet. Groundwater seepage was not encountered within the remaining borings during or at the completion of drilling operations. The amount of water present in rock mass defects may fluctuate over time. In many cases, the rate of seepage would suggest that groundwater may be controlled using conventional pumps. In the event that excessive groundwater seepage is encountered that cannot be controlled with conventional pumps, sumps, or other means, or in the event that excessive sidewall sloughing occurs, temporary casing will be necessary. Concrete should be onsite during drilling operations, to facilitate placement immediately after drilling of each shaft is complete.

The installation of all drilled piers should be observed by experienced geotechnical personnel during construction to verify compliance with design assumptions including: 1) verticality of the shaft excavation, 2) identification of the bearing stratum, 3) minimum pier diameter and depth, 4) correct amount of reinforcement, 5) proper removal of loose material, and 6) that groundwater seepage, if present, is properly controlled. D&S would be pleased to provide these services in support of this project.

During construction of the drilled shafts, care should be taken to avoid creating an oversized cap ("mushroom") near the ground surface that is larger than the shaft diameter. These “mushrooms” provide a resistance surface that near-surface soils can heave against. If near-surface soils are prone to sloughing, a condition which can result in “mushrooming”, the tops of the shafts should be formed in the sloughing soils using cardboard or other circular forms equal to the diameter of the shaft.

Concrete used for the shafts should have a slump of 8 inches ± 1 inch. Individual shafts should be excavated in a continuous operation and concrete should be placed as soon as after completion of the drilling as is practical. All pier holes should be filled with concrete within 8 hours after completion of
drilling. In the event of equipment breakdown, any uncompleted open shaft should be backfilled with soil to be re-drilled at a later date. This office should be contacted when shafts have reached the target depth but cannot be completed.

6.1.2 Lateral Loads

We have the following general recommendations for subsurface resistance to imposed lateral loads suitable for use in LPILE® or other lateral load software. These values are based on the stratigraphy observed and should be modeled as “Stiff Clay w/o Free Water” and “Weak Rock” as described in the table below. The depth ranges are based on the borings drilled. This stratigraphic section was selected to conservatively approximate the subsurface conditions across the site for lateral analysis. In view of the nature and characteristics of the materials present, we recommend that the lateral resistance parameters be neglected for the uppermost 5 feet of soil materials to account for seasonal and annual cyclic variations in soil desiccation and contraction and due to a lack of confining pressure.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Approximate Depth Below Current Grades (ft)</th>
<th>Software Material Designation</th>
<th>Total Unit Weight (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Shale</td>
<td>8-13</td>
<td>Weak Rock</td>
<td>125</td>
</tr>
<tr>
<td>Limestone/shale</td>
<td>13+</td>
<td>Weak Rock</td>
<td>135</td>
</tr>
</tbody>
</table>

Table 3. Recommended Geotechnical Lateral Load Parameters

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Unconfined Compressive Strength – Rock (psi)</th>
<th>RQD (%)</th>
<th>Modulus (psi)</th>
<th>Strain Factor (ε₅₀)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Shale</td>
<td>100</td>
<td>50</td>
<td>7,000</td>
<td>0.0005</td>
</tr>
<tr>
<td>Limestone/Shale</td>
<td>200</td>
<td>80</td>
<td>25,000</td>
<td>0.0001</td>
</tr>
</tbody>
</table>

6.2 Shallow Foundations

Alternatively, structural loads for guy-wire anchors may be designed as reinforced concrete, monolithic shallow spread footings that are founded a minimum depth of 24 inches below the final exterior grade. Such footings may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf) when the foundations are based in properly prepared reworked soils. Footings should be a minimum of 36 inches in width.
The bottom of all footing excavations should be essentially free of any loose or soft material prior to the placement of concrete. All footings should be adequately reinforced to minimize cracking, as normal movements will occur in the foundation soils.

Tensile forces acting on the deadman anchors will be resisted by the weight of the anchor itself in addition to the weight of soil in-line with the guy wire and may be taken up to one foot below finished surface grades if no future excavations will take place above and to the front of the deadman anchors. A moist unit weight of 125pcf may be used to calculate the dead weight of the soil over the deadman anchor. This weight of soil should be taken as a wedge extending upward parallel to the guy wire plus 10 degrees (the wedge getting bigger as it approaches the ground surface) from the far edges of the top and front faces of the deadman anchor. We recommend that the front face of the deadman anchor excavation be neat cut to maximize the passive pressure and limit movement as a result of the tensile forces. Where forms are used for the remaining sides, the exterior sides of the anchors should be carefully backfilled with on-site lean (CL) clayey soils. The backfill soils should be compacted to at least 98 percent of the maximum dry density for the backfill material as determined by ASTM D698 (standard Proctor method). The backfill should also be placed at plus or minus two (+/-2%) percentage points of the optimum moisture content, as determined by that same test.

Periodically the tension in the guy wires should be adjusted as a loss in tension may occur as the deadman anchor adjusts to the tensile load.

All footings or footing segments should be constructed in a relatively seamless operation, with excavation activities and placement of the reinforcement steel and concrete occurring within 5 days. If concrete cannot be placed in newly excavated footings in a timely manner, the base of the excavated footing may be covered with a thin seal of lean concrete. We recommend that a qualified representative of a geotechnical engineer should observe all footing excavations prior to concrete placement to verify the competence of the bearing stratum. Any footing excavations that are left open overnight should be observed by the representative prior to concrete placement to determine the degree of stratum degradation and, if necessary, to recommend additional excavation where required. D&S would be pleased provide these services in support of this project.

7.0 EARTHWORK RECOMMENDATIONS

The near-surface materials present have a low to moderate potential for post-construction vertical movement with changes in subsurface soil moisture changes. We have the following earthwork recommendations to provide a uniform pad and limit the potential for post-construction movements to the order of 1-inch for all ground-supported foundation options provided in this report. Please note that more stringent tolerances limiting potential post-construction vertical movement will require more extensive effort.
7.1 Subgrade Preparation for Soil Supported Elements

- Strip the site of all vegetation and remove any remaining organic or deleterious material within the new structure areas. Typically, 6 to 12 inches are sufficient for this purpose.

- After stripping and performing any necessary cuts, scarify and rework the base of the excavation to a depth of 12 inches. The scarified and reworked soils should be compacted to 98 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and be placed at a moisture content plus or minus two (+/-2%) percentage points of the optimum moisture content, as determined by that same test. If competent, undisturbed limestone is present at the base of the excavation, the limestone should be left undisturbed and should not be scarified or recompacted.

- Within 24 hours of recompacting the reworked excavated subgrade, begin any necessary fill operations with debris-free on-site or imported soil to raise the pad to finished subgrade elevation. The fill soil should be placed in maximum 6-inch compacted lifts, be compacted at least 98 percent of the maximum dry density as determined by ASTM D698 (standard Proctor) and be placed at a moisture content plus or minus two (+/-2%) percentage points of the optimum moisture content, as determined by that same test.

- Grade raise fill within the pad areas may be onsite soils or imported materials having a Liquid Limit (LL) of 35 or less, a Plasticity Index (PI) between 6 and 20, a minimum of 30% of the material passing a No. 200 mesh sieve and be essentially free of particles in excess of 4 inches in their longest direction.

- The moisture content of the subgrade should be maintained up to the time of concrete placement. Depending on the speed of the earthwork layers, on hot or windy days, sprinkling with water atop the subgrade may be required, to maintain the compaction moisture content.

- Water should not be allowed to pond on the prepared subgrade either during fill placement, or after reaching final subgrade elevation. To that end, the subgrade surfaces should be shaped to shed water to the edges of the respective pads.

- Each lift of fill placed should be tested for moisture content and degree of compaction by a testing laboratory at a minimum frequency of one (1) test performed for every 3,000 square feet of fill, with a minimum of one (1) test performed per lift of fill placed within the footprint of each foundation pad and deadman anchor zone.
7.2 Additional Considerations

In order to minimize the potential for post-construction vertical movement, consideration should be given to the following:

- Trees or shrubbery with a mature height greater than 6 feet and/or that require excessive amounts of water should not be planted near structures or flatwork.
- Trees should not be planted closer than the mature tree’s height from structures or flatwork.
- Water should not be allowed to pond next to the foundations.

8.0 PAVEMENT RECOMMENDATIONS

8.1 General

The pavement design recommendations provided herein are derived from the subgrade information that was obtained from our geotechnical investigation, design assumptions based on project information, our experience with similar projects in this area, and on the guidelines and recommendations of the American Concrete Pavement Association (ACPA). It is ultimately the responsibility of the Civil Engineer of Record and/or other design professionals who are responsible for pavement design to seal the final pavement plans and associated specifications for this project. The pavements for this project are expected to service light duty passenger vehicles. If the pavements will be subjected to heavy traffic loads/volumes, the traffic loading information should be provided to this office for review prior to construction to confirm and/or revise the recommendations provided herein.

8.2 Behavior Characteristics of Expansive Soils Beneath Pavement

Near-surface soils at this site are considered to have a low potential for volume change with changes in soil moisture content. The moisture content can be “stabilized”, to some degree, in these soils by covering them with an impermeable surface, such as pavement. However, if moisture is introduced by surface or subsurface water, poor drainage or the addition of excessive irrigation after periods of no moisture, the soil strength can reduce causing distress to pavements as traffic travels over it.

The edges of pavements are particularly prone to moisture variations, therefore, these areas often experience the most distress (cracking, displacement, etc.). When cracks appear on the surface of the pavement, these openings can allow moisture to enter the pavement subgrade, which can lead to further weakening of the pavement section as well as accelerated failure of the pavement surface.
In order to minimize the potential impacts of excessive water entering pavement subgrades and to improve the long-term performance of the pavement, we have the following recommendations:

- Design a crowned or sloped pavement with edge slopes. A minimum slope of five percent within the first 5 feet from the edge of the pavement is considered ideal.

- Subgrade treatments should be extended to at least 24-inches beyond the back of curbs or edges of pavements.

8.3 Subgrade Strength Characteristics

The anticipated subgrade soils in the proposed paving areas will generally consist of clay soils. We recommend that a California Bearing Ratio (CBR) value of 3 be used for the on-site clay soils in the design, with a corresponding resilient modulus of 4,100 psi. For compacted aggregate base, we also recommend a resilient modulus of 20,000 psi.

8.4 Pavement Subgrade Preparation Recommendations

The anticipated subgrade soils will generally be a combination of sand and clay soils in the proposed paving areas which have poor subgrade characteristics and can become soft and pump with an increase in moisture content. A commonly used method to reduce the potential for pumping, improve the strength properties of the subgrade soils, provide a working platform, reduce PVM and provide a uniform subgrade is to providing an aggregate base course layer. We have included recommendations for the use of an aggregate base course. The following recommendations discuss the subgrade preparation.

8.4.1 General

- Strip the site of all vegetation and remove any remaining organic or deleterious material under the planned paved areas. Typically, 6 to 12 inches are sufficient for this purpose.

- Perform any cut operations as needed.

- After stripping and performing necessary cuts, the exposed subgrade should be proof rolled. Proof rolling should consist of rolling the entire pavement subgrade in mutually perpendicular directions with a heavily-loaded, tandem-axle dump truck weighing at least 25 tons or other approved equipment capable of applying similar loading conditions. Any soft, wet or weak soils that are observed to rut or pump excessively during proof rolling should be removed and replaced with well-compacted, on-site clayey material as outlined below. The proof rolling operation should be performed under the
observation of a qualified geotechnical engineer. D&S would welcome the opportunity to perform these services for this project.

- Following proof rolling, the upper one foot should be scarified and recompacted. The scarified fill should be recompacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and at a moisture content that is at or above the optimum moisture content, as determined by the same test.

- In areas to receive fill, fill may be derived from on-site or may be imported. The fill should be placed in maximum 6-inch compacted lifts, compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and placed at a moisture content that is at or above the optimum moisture content, as determined by the same test. Prior to compaction, each lift of fill should first be processed throughout its thickness to break up and reduce clod sizes and blended to achieve a material of uniform density and moisture content. Once blended, compaction should be performed with a heavy tamping foot roller. Once compacted, if the surface of the embankment is too smooth, it may not bond properly with the succeeding layer. If this occurs, the surface of the compacted lift should be roughened and loosened by dicing before the succeeding layer is placed.

- Water required to bring the fill material to the proper moisture content should be applied evenly through each layer. Any layers that become significantly altered by weather conditions should be reprocessed in order to meet recommended requirements. On hot or windy days, the use of water spraying methods may be required in order to keep each lift moist prior to placement of the subsequent lift. Furthermore, the subsurface soils should be kept moist prior to placing the pavement by water sprinkling or spraying methods.

- Fill materials should be placed on a properly prepared subgrade as outlined above. The combined excavation, placement, and spreading operation should be performed in such a manner as to obtain blending of the material, and to assure that, once compacted, the materials, will have the most practicable degree of compaction and stability. Materials obtained from on-site should be mixed and not segregated.

- Soil imported from off-site sources should be tested for compliance with the recommendations herein and approved by the project geotechnical engineer prior to being used as fill. Imported common fill materials should consist of lean clays (maximum Plasticity Index of
that are essentially free of organic materials and particles larger than 4 inches in their maximum dimension.

- Field density and moisture content testing should be performed at the rate of one (1) test per lift per 100 linear feet of drive aisle, and at the rate of one (1) test per lift per 5,000 square feet in parking and other open pavement areas. These tests are necessary to determine if the recommended moisture and compaction requirements have been attained.

8.4.2 Aggregate Base

Aggregate base may be placed over the prepared subgrade in accordance with the following recommendations prior to placing the pavement.

- Aggregate base should be TxDOT Type A or D and meeting the gradation, durability and plasticity requirements of TxDOT Item 247 Grade 1-2 or better (2014). Aggregate base material should be uniformly compacted in maximum 6-inch compacted lifts to a minimum of 95% of the maximum standard Proctor dry density (ASTM D698) and placed at a moisture content that is sufficient to achieve density.

- After proof rolling, and prior to the placement of aggregate base, the exposed subgrade beneath pavement areas should be scarified and reworked to a depth of 12 inches, moisture added or removed as required, and the subgrade soils recomposed to a minimum of 95 percent of the maximum dry density of the materials obtained in accordance with ASTM D698 (standard Proctor test) and that is at or above the material’s optimum moisture content, as determined by the same test. The rework and aggregate base should extend to at least 24-inches beyond the outside edges of curbs.

- Field density and moisture content testing should be performed at the rate of one (1) test per lift per 100 linear feet of drive aisle, and at the rate of one (1) test per lift per 5,000 square feet in parking and other open pavement areas.

8.5 Rigid Pavement

We recommend that Portland Cement Concrete Pavement for this site have a minimum thickness of 5 inches for light-duty automobile parking over 6 inches of aggregate base. Concrete thickness should be increased to 6 inches for fire lanes over 8 inches of aggregate base. For dumpster pads, the concrete thickness should be increased to 7 inches over 8 inches of aggregate base. Actual traffic loading,
frequency, and intensity may require an increase in these minimum recommendations. We have the following concrete mix design recommendations:

- Recommended minimum design compressive strength: 3,500 psi with nominal aggregate size no greater than 1 inch.
- 15 to 20 percent fly ash may be used with the approval of the Civil Engineer of record.
- Curing compound should be applied within one hour of finishing operations.

8.5.1 Pavement Joints and Cutting

The performance of concrete pavement depends to a large degree on the design, construction, and long-term maintenance of concrete joints. The following recommendations and observations are offered for consideration by the Civil Engineer and/or pavement Designer-of-Record.

The concrete pavements should have adequately spaced contraction joints to control shrinkage cracking. Experience indicates that reinforced concrete pavements with sealed contraction joints on a 12 to 15-foot spacing, cut to a depth of one-quarter to one-third of the pavement thickness, have generally exhibited less uncontrolled post-construction cracking than pavements with wider spacing. The contraction joint pattern should divide the pavement into panels that are approximately square where the panel length should not exceed 25 percent more than the panel width. Saw cut, post placement formed contraction joints should be saw cut as soon as the concrete can support the saw cutting equipment and personnel and before shrinkage cracks appear, on the order of 4 to 6 hours after concrete placement.

Isolation joints should be used wherever the pavement will abut a structural element subject to a different magnitude of movement, e.g., light poles, retaining walls, existing pavement, stairways, entryway piers, building walls, or manholes.

In order to minimize the potential differential movement across the pavement areas, all joints including contraction, isolation and construction joints should be sealed to minimize the potential for infiltration of surface water. Rubberized asphalt, silicone or another suitable flexible sealant may be used to seal the joints. Maintenance should include periodic inspection of these joints and the joints resealed as necessary.

8.5.2 Pavement Reinforcing Steel

We recommend that a minimum of 0.1 percent of steel be used for all concrete pavements. For a 6-inch-thick concrete pavement section, this reinforcement ratio is approximately equivalent to No. 3 bars spaced at 18-inches on center
each way. Reinforcement requirements may increase depending on specific traffic loading and design life parameters.

8.5.3 Surface Drainage

Proper drainage is critical to the performance of the paved areas. Positive surface drainage should be provided that directs surface water away from pavement edges. Where possible, we recommend that a slope of at least 5 percent be provided. The slopes should direct water away from the structure and should be maintained throughout construction and the life of the structure.

8.6 Flexible Pavement Design and Recommendations

If utilized for this project, hot mix asphaltic concrete (HMAC) pavement should conform to current TxDOT standards. The following subparagraph provides recommendations for HMAC. Aggregate base should meet the material requirements and be prepared as outlined in Section 8.4 of this report.

8.6.1 Full Depth HMAC

Full-depth HMAC should consist of at least 2 inches of Type C or D surface course over 4 inches of Type B base course as specified by TxDOT. The full depth of asphalt should be placed over 6-inches of aggregate base course. The total thickness of HMAC and treated subgrade/base should be increased by 1 and 2 inches, respectively, for fire lanes. We do not recommend that HMAC pavement be used for dumpster pad areas.

9.0 OTHER CONSTRUCTION

9.1 Utility and Service Lines

Backfill for utility lines should consist of on-site material and should be placed in accordance with the following recommendations. The on-site fill soil should be placed in maximum 6-inch compacted lifts, compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and placed at a moisture content that is at least the optimum moisture content, as determined by that same test. It is not uncommon to realize some settlement along the trench backfill. We also recommend that the utility trenches be visually inspected during the excavation process to ensure that undesirable fill that was not detected by the test borings does not exist at the site. This office should be notified immediately if any such fill is detected.

Utility lines connected to the structure may experience differential movement in response to changing moisture conditions in expansive soil. These movements may result in damage to the lines, especially at connections. Flexible connections may be considered to account for potential differential movement between the building and utilities.
Utility excavations should be sloped so that water within excavations will flow to a low point away from the active construction where it can be removed before backfilling. Compaction of bedding material should not be water-jetted. Compacted backfill above the utilities should be on-site clays to limit the percolation of surface water. Utility trenches extending under structures should include fat clay or concrete cut-off collars at the perimeter/edge to prevent the transmission of water along trench lines.

9.2 Exterior Flatwork

Concrete flatwork should include high tensile steel reinforcement to reduce the formation and size of cracks. Flatwork should also include frequent and regularly spaced expansion/control joints and dowels to limit vertical offsets between neighboring flatwork slabs. Structure entrances should either be part of the structure or designed to tolerate vertical movement without inhibiting access. The moisture content of the subgrade should be maintained up to the time of concrete placement. If subgrade soils are allowed to dry below the levels recommended herein, additional moisture conditioning of the soils may be required. These recommendations are intended to reduce possible distress to exterior flatwork but will not prevent movement and/or vertical offsets between slabs.

The concrete flatwork should have adequately spaced contraction joints to control shrinkage cracking. Past experience indicates that reinforced concrete flatwork with sealed contraction joints on a 5 to 10-foot spacing, cut to a depth of one-quarter to one-third of the pavement thickness, have generally exhibited less uncontrolled post-construction cracking than pavements with wider spacing. The contraction joint pattern should divide the pavement into panels that are approximately square where the panel length should not exceed 25 percent more than the panel width. Saw cut, post placement formed contraction joints should be saw cut as soon as the concrete can support the saw cutting equipment and personnel and before shrinkage cracks appear, on the order of 4 to 6 hours after concrete placement. Rubberized asphalt, silicone or other suitable flexible sealant could be used to seal the joints. Isolation joints should be used wherever the pavement will abut a structural element subject to a different magnitude of movement, e.g., light poles, retaining walls, existing pavement, stairways, entryway piers, building walls, or manholes.

9.3 Surface Drainage

Proper drainage is critical to the performance and condition of the building foundation, pavement and flatwork. Positive surface drainage should be provided that directs surface water away from buildings and flatwork. Where possible, we recommend that the exterior grades slope away from foundations at the rate of five (5) percent in the first five (5), and preferably ten (10) feet away. The slopes should direct water away from the structure, and these grades should be maintained throughout construction and the life of the structure.
9.4 Landscaping

Landscaping against and around the exterior of the structure can adversely affect subgrade moisture resulting in localized differential movements if not properly maintained. If used, landscaping should be kept as far away from the foundation as possible, and positive drainage away from the structure should be designed, constructed, and maintained. Landscaping elements (such as edging) should not prohibit or slow the drainage of water that could result in water ponding next to foundations or edges of flatwork. When feasible, irrigation lines and heads should not be placed in close proximity to the foundation to prevent the collection of water near the foundation or flatwork, particularly in the event of leaking lines or sprinkler heads.

Trees (if planned) should not be placed in proximity to the structure or movement sensitive flatwork, as trees are known to cause in localized soil shrinkage due to desiccation of the soil by the root system, possibly leading to differential movements of the structure. The desiccation zone varies by tree, but trees should not be planted closer to structures than the mature tree height, and in no case, should the drip-line of the mature tree extend closer than 10-feet of rooflines. A moist but not overly wet soil condition should be maintained at all times in all landscaped areas near buildings after construction to minimize soil volume changes caused by changing soil moisture conditions.

9.5 Excavations

Excavations greater than 5 feet in height/depth should be in accordance with OSHA 29CFR 1926, Subpart P. Temporary construction slopes should incorporate excavation protection systems or should be sloped back. Where the excavation does not extend close to building lines, these areas may be laid back. Where space allows, temporary slopes should be sloped at 1.5 horizontal to 1 vertical (1.5H: 1V) or flatter.

Where excavation slopes greater than five (5) feet in height cannot be laid back, these areas will require installation of a temporary retention system or shoring to protect the existing construction, restrain the subsurface soils and maintain the integrity of the excavation. We recommend that monitoring points be established around the retention system and that these locations be monitored during and after the excavation activities to confirm the integrity of the retention system.

The slopes and temporary retention system should be verified by and designed by the contractor's engineer and should not be surcharged by traffic, construction equipment, or permanent structures. The slopes and temporary retention system should be adequately maintained and periodically inspected to insure the safety of the excavation and surrounding property.
10.0 LIMITATIONS

The professional geotechnical engineering services performed for this project, the findings obtained, and the recommendations prepared were accomplished in accordance with currently accepted geotechnical engineering principles and practices.

Variations in the subsurface conditions are noted at the specific boring locations for this study. As such, all users of this report should be aware that differences in depths and thicknesses of strata encountered can vary between the boring locations. Statements in the report as to subsurface conditions across the site are extrapolated from the data obtained at the specific boring locations. The number and spacing of the exploration borings were chosen to obtain geotechnical information for the design and construction of moderately loaded pole structures, guy-wire anchors, and associated pavements. If there are any conditions differing significantly from those described herein, D&S should be notified to re-evaluate the recommendations contained in this report.

Recommendations contained herein are not considered applicable for an indefinite period of time. Our office must be contacted to re-evaluate the contents of this report if construction does not begin within a one-year period after completion of this report.

The scope of services provided herein does not include an environmental assessment of the site or investigation for the presence or absence of hazardous materials in the soil, surface water, or groundwater.

All contractors referring to this geotechnical report should draw their own conclusions regarding excavations, construction, etc. for bidding purposes. D&S is not responsible for conclusions, opinions or recommendations made by others based on these data. The report is intended to guide preparation of project specifications and should not be used as a substitute for the project specifications.

Recommendations provided in this report are based on our understanding of information provided by the Client to us regarding the scope of work for this project. If the Client notes any differences, our office should be contacted immediately since this may materially alter the recommendations.

This report has been prepared for the exclusive use of our client for specific applications to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless D&S reviews the changes and either verifies or modifies the conclusions of this report in writing.
APPENDIX A - BORING LOGS AND SUPPORTING DATA
**BORING LOCATIONS ARE INTENDED FOR GRAPHICAL REFERENCE ONLY**

N.T.S.
DENTON TEXAS

DATE TO DRILL
May 19, 2022

BORING SITE MAP
UNT OUTDOOR TESTING FACILITY

SHEET NO.
G1
TEXAS
### KEY TO SYMBOLS AND TERMS

#### LITHOLOGIC SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td></td>
</tr>
<tr>
<td>Aggregate Base</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Fill</td>
<td></td>
</tr>
</tbody>
</table>

#### ARTIFICIAL

- Asphalt
- Aggregate Base
- Concrete
- Fill

#### SOIL

- CH: High Plasticity Clay
- CL: Low Plasticity Clay
- GP: Poorly-graded Gravel
- GW: Well-graded Gravel
- SC: Clayey Sand
- SP: Poorly-graded Sand
- SW: Well-graded Sand

#### ROCK

- Limestone
- Mudstone
- Shale
- Sandstone
- Weathered Limestone
- Weathered Shale
- Weathered Sandstone

#### CONSISTENCY OF SOILS

**CONSISTENCY: FINE GRAINED SOILS**

<table>
<thead>
<tr>
<th>Consistency</th>
<th>SPT (# blows/ft)</th>
<th>UCS (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>0 - 2</td>
<td>&lt; 0.25</td>
</tr>
<tr>
<td>Soft</td>
<td>3 - 4</td>
<td>0.25 - 0.5</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>5 - 8</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
<td>1.0 - 2.0</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
<td>2.0 - 4.0</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 4.0</td>
</tr>
</tbody>
</table>

**CONSISTENCY: COARSE GRAINED SOILS**

<table>
<thead>
<tr>
<th>Condition</th>
<th>SPT (# blows/ft)</th>
<th>TCP (# blows/ft)</th>
<th>Relative Density (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0 - 4</td>
<td>&lt; 8</td>
<td>0 - 15</td>
</tr>
<tr>
<td>Loose</td>
<td>5 - 10</td>
<td>8 - 20</td>
<td>15 - 35</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>11 - 30</td>
<td>20 - 60</td>
<td>35 - 65</td>
</tr>
<tr>
<td>Dense</td>
<td>31 - 50</td>
<td>60 - 100</td>
<td>65 - 85</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
<td>&gt; 100</td>
<td>85 - 100</td>
</tr>
</tbody>
</table>

#### SECONDARY COMPONENTS

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity Descriptors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt; 5% of sample</td>
</tr>
<tr>
<td>Few</td>
<td>5% to 10%</td>
</tr>
<tr>
<td>Little</td>
<td>10% to 25%</td>
</tr>
<tr>
<td>Some</td>
<td>25% to 35%</td>
</tr>
<tr>
<td>With</td>
<td>&gt; 35%</td>
</tr>
</tbody>
</table>

#### RELATIVE HARDNESS OF ROCK MASS

<table>
<thead>
<tr>
<th>Designation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>Can be carved with a knife. Can be excavated readily with point of pick. Pieces 1&quot; or more in thickness can be broken by finger pressure. Readily scratched with fingernail.</td>
</tr>
<tr>
<td>Soft</td>
<td>Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows with the pick point. Small, thin pieces can be broken by finger pressure.</td>
</tr>
<tr>
<td>Medium Hard</td>
<td>Can be grooved or gouged 1/4&quot; deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1&quot; maximum size by hard blows with the point of a pick.</td>
</tr>
<tr>
<td>Moderately Hard</td>
<td>Can be scratched with knife or pick. Gouges or grooves 1/4&quot; deep can be excavated by hard blow of the point of a pick. Hand specimens can be detached by a moderate blow.</td>
</tr>
<tr>
<td>Hard</td>
<td>Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach a hand specimen.</td>
</tr>
<tr>
<td>Very Hard</td>
<td>Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows from a hammer or pick.</td>
</tr>
</tbody>
</table>

#### WEATHERING OF ROCK MASS

<table>
<thead>
<tr>
<th>Designation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh</td>
<td>No visible sign of weathering</td>
</tr>
<tr>
<td>Slightly weathered</td>
<td>Penetrative weathering on open discontinuity surfaces, but only slight weathering of rock material</td>
</tr>
<tr>
<td>Moderately weathered</td>
<td>Weathering extends throughout rock mass, but the rock material is not friable</td>
</tr>
<tr>
<td>Highly weathered</td>
<td>Weathering extends throughout rock mass, and the rock material is partly friable</td>
</tr>
<tr>
<td>Completely weathered</td>
<td>Rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved</td>
</tr>
<tr>
<td>Residual Soil</td>
<td>A soil material with the original texture, structure, and mineralogy of the rock completely destroyed</td>
</tr>
</tbody>
</table>
# Unified Soil Classification System

## Soil Classification Chart

### Major Divisions

<table>
<thead>
<tr>
<th>Gravels</th>
<th>CLEAN GRAVELS (Less than 5% Finer)</th>
<th>GW</th>
<th>Well-Graded Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cu ≥ 4 and 1 ≤ Cc ≤ 3</td>
<td>GP</td>
<td>Poorly-Graded Gravel</td>
</tr>
<tr>
<td></td>
<td>Cu &lt; 4 or (Cc &lt; 1 or Cc &gt; 3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravels With Finer (More than 12% Finer)</td>
<td>Finer classify as ML or MH</td>
<td>GM</td>
<td>Silty Gravel</td>
</tr>
<tr>
<td>Sands</td>
<td>CLEAN SANDS (Less than 5% Finer)</td>
<td>SW</td>
<td>Well-Graded Sand</td>
</tr>
<tr>
<td></td>
<td>Cc ≥ 6 and 1 ≤ Cc ≤ 3</td>
<td>SP</td>
<td>Poorly-Graded Sand</td>
</tr>
<tr>
<td></td>
<td>Cu &lt; 6 and (Cc &lt; 1 or Cc &gt; 3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sands With Finer (More than 12% Finer)</td>
<td>Finer classify as ML or MH</td>
<td>SM</td>
<td>Silty Sand</td>
</tr>
<tr>
<td></td>
<td>Finer classify as CL or CH</td>
<td>SC</td>
<td>Clamy Sand</td>
</tr>
</tbody>
</table>

### Fine-Grained Soils

<table>
<thead>
<tr>
<th>Silts and Clays</th>
<th>INORGANIC</th>
<th>CL</th>
<th>Lean Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit Less Than 50</td>
<td>Pi &gt; 7 and plots on or above &quot;A&quot; line</td>
<td>ML</td>
<td>Clay</td>
</tr>
<tr>
<td></td>
<td>Pi &lt; 4 or plots below &quot;A&quot; line</td>
<td>OL</td>
<td>Organic Clay</td>
</tr>
<tr>
<td></td>
<td>Liquid limit – oven dried</td>
<td>CH</td>
<td>Fat Clay</td>
</tr>
<tr>
<td></td>
<td>Liquid limit – not dried &lt; 0.6%</td>
<td>MH</td>
<td>Elastic Silt</td>
</tr>
<tr>
<td>Organic</td>
<td>Pi plots on or above &quot;A&quot; line</td>
<td>OH</td>
<td>Organic Clays Silt</td>
</tr>
<tr>
<td></td>
<td>Pi plots below &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Liquid limit – oven dried</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Liquid limit – not dried &lt; 0.7%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Highly Organic Soils

| Primarily Organic Matter, Dark in Color, and Organic Odor | PT | Peat |

## Plasticity Chart

For classification of fine-grained soils and fine-grained fraction of coarse-grained soils:

- Equation of "A"-Line
  Horizontal at Pi = 4 to LL = 25.5, then Pi = 0.73(LL-20)
- Equation of "U"-Line
  Vertical at LL = 16 to Pi = 7 then Pi = 0.9(LL-6)

- "U" Line
- "A" Line
- CL/ML
- ML or OL
- MH or OH
- OL or OL
- CH or OH
**BORING LOG**

**CLIENT:** University of North Texas  
**LOCATION:** Denton, Texas

**PROJECT NUMBER:** G22-2040  
**GPS COORDINATES:** N33.255546, W97.150806

**START DATE:** 5/19/2022  
**FINISH DATE:** 5/19/2022

**PROJECT:** UNT Outdoor Testing Facility  
**DRILL METHOD:** Hollow Stem Flight Auger

**DRILLED BY:** Octavio Herrera (D&S)  
**LOGGED BY:** Ismael Hernandez (D&S)

---

**Ground Elevation:** Approx. 715 feet  
**GPS Coordinates:** N33.255546, W97.150806

---

**Depth (ft)** | **Sample Type** | **Hand Pen. (tsf) or SPT or TCP** | **Legend** | **REC (%)** | **ROD (%)** | **Atterberg Limits** | **Passing #200 Sieve (%)** | **Total Suction (pF)** | **Clay (%)** | **Swell (%)** | **DUW (pcf)** | **Unconf. Compr. Str (ksf)**
---|---|---|---|---|---|---|---|---|---|---|---|---
0 | S | 4.0 | LEAN CLAY (CL); very stiff, brown; trace to few ferrous nodules; trace sand | 1.0 ft | 1.0 ft | 11.8 | 30 | 15 | 15
0 | S | 1.0 | | | | | | | |
0 | S | 2.0 | CLAYEY SAND (SC); dense; orange, red, brown, gray; fine grained | 3.0 ft | 3.0 ft | 9.8 | | | |
5 | T | 50=1.5" | LIMESTONE; moderately weathered; hard; red, tan; frequent sand seams | 5.0 ft | 5.0 ft | 11.8 | 30 | 15 | 15
5 | B | 50=1.0" | | | | | | | |
10 | T | 50=0.5" | LIMESTONE; fresh; hard; gray | 710.0 ft | 710.0 ft | 9.2 | | | |
10 | T | 50=0.5" | | | | | | | |
15 | T | 50=1.5", 50=0.75" | SHALE; fresh; hard; dark gray; calcareous | 693.0 ft | 693.0 ft | 11.8 | 30 | 15 | 15
20 | T | 50-0.25", 50=0.25" | | | | | | | |
25 | T | 50=1.5", 50=0.75" | | | | | | | |
30 | T | 50=1.0", 50=1.0" | | | | | | | |
35 | T | 50=1.0", 50=0.25" | End of boring at 35.1' | 35.1 ft | 35.1 ft | 679.9 ft | | | |
40 | | | | | | | | |

**Notes:**
- dry during drilling
- dry upon completion
LEAN CLAY (CL); very stiff; brown; with sand; trace ferrous nodules
CLAYEY SAND (SC); dense; brown, orange; fine grained
SHALE; highly to completely weathered; soft to medium soft; brown, red, orange; with silt and sand seams
LIMESTONE; moderately weathered; hard; tan; reddish-brown; with sand seams
SHALE; fresh; hard; dark gray; calcareous

Notes:
- Seepage at 18 feet during drilling
- Water at 20 feet upon completion
**BORING LOG**

**CLIENT:** University of North Texas  
**LOCATION:** Denton, Texas  
**PROJECT NUMBER:** G22-2040  
**GROUND ELEVATION:** Approx. 714 feet  
**START DATE:** 5/19/2022  
**FINISH DATE:** 5/19/2022  
**DRILL METHOD:** Hollow Stem Flight Auger  

**LOGGED BY:** Ismael Hernandez (D&S)  
**DRILLED BY:** Octavio Herrera (D&S)

---

**Hand Pen. (tsf) or SPT or TCP**  
**Depth (ft)**  
**Sample Type**  
**Legend:**  
- S - Shelby Tube  
- N - Standard Penetration  
- T - Texas Cone Penetration  
- C - Core  
- B - Bag Sample  
- W - Water Encountered

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample Type</th>
<th>Hand Pen. (tsf) or SPT or TCP</th>
<th>Graphic Log</th>
<th>REC (%)</th>
<th>RQD (%)</th>
<th>Atterberg Limits (%)</th>
<th>Passing #200 Sieve (%)</th>
<th>Total Suction (pF)</th>
<th>Clay (%)</th>
<th>Swell (%)</th>
<th>DUW (pcf)</th>
<th>Unconf. Compr. Str (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>S</td>
<td>4.5+</td>
<td>LEAN CLAY (CL); very stiff; brown; with sand; trace iron oxide stains and rock fragments</td>
<td>1.0 ft</td>
<td>713.0 ft</td>
<td>11.4</td>
<td>56</td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>5</td>
<td>N</td>
<td>1.0</td>
<td>CLAYEY SAND (SC); dense; brown, orange; fine grained</td>
<td>5.0 ft</td>
<td>709.0 ft</td>
<td>10.6</td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>10</td>
<td>B</td>
<td>2.5&quot;, 48</td>
<td>SHALE; highly to completely weathered; soft to medium hard; tan, reddish-brown; with interbedded sand seams</td>
<td>5.0 ft</td>
<td>709.0 ft</td>
<td>13.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>T</td>
<td>3.0&quot;, 50=1.75</td>
<td>LIMESTONE; moderately weathered; hard; tan, reddish-brown</td>
<td>13.5 ft</td>
<td>700.5 ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>B</td>
<td>4.5&quot;, 50=4.5&quot;</td>
<td>SHALE; fresh; soft to moderately hard; dark gray; calcareous</td>
<td>18.0 ft</td>
<td>696.0 ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>B</td>
<td>1.75&quot;, 50=1.5&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>B</td>
<td>2.25&quot;, 50=2.0&quot;</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>B</td>
<td>1.5&quot;, 50=1.0&quot;</td>
<td>End of boring at 35.2&quot;</td>
<td>35.2 ft</td>
<td>678.8 ft</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

**Notes:**  
- seepage at 15 feet during drilling  
- water at 30 feet upon completion
LEAN CLAY (CL); very stiff, brown; with sand; trace iron oxide stains and rock fragments

CLAYEY SAND (SC); medium dense to dense; orange, brown; trace silt; fine grained

SANDY LEAN CLAY (CL); very stiff; gray, brown; trace to few ferrous nodules and sand seams; trace sand laminations and gravel

SHALE; highly to completely weathered; very soft; reddish-brown, tan; with interbedded sand seams; fossiliferous - push refusal at 9 feet

LIMESTONE; moderately weathered; moderately hard; tan

LIMESTONE; fresh; gray

SHALE; fresh; soft to hard; dark gray

Notes:
- dry during drilling
- dry upon completion
### SWELL TEST RESULTS

**PROJECT:** UNT Outdoor Testing Facility  
**CLIENT:** University of North Texas  
**PROJECT NUMBER:** G22-2040  
**LOCATION:** Denton, Texas

<table>
<thead>
<tr>
<th>Boring Number</th>
<th>Depth feet</th>
<th>Initial Moisture Content, %</th>
<th>Final Moisture Content, %</th>
<th>Applied Pressure, psf</th>
<th>Vertical Swell, %</th>
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<tbody>
<tr>
<td>B4</td>
<td>2-3</td>
<td>9.7</td>
<td>19.4</td>
<td>257</td>
<td>0.0</td>
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<tr>
<td>B4</td>
<td>7-8</td>
<td>14.0</td>
<td>19.2</td>
<td>907</td>
<td>1.7</td>
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<tr>
<td>Boring Number</td>
<td>Depth (feet)</td>
<td>Soil Description</td>
<td>Soluble Sulfate Content (ppm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>-------------</td>
<td>-------------------------------------</td>
<td>------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td>0-1</td>
<td>LEAN CLAY (CL); very stiff; brown</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B3</td>
<td>2-3</td>
<td>CLAYEY SAND (SC); dense; brown, orange</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>6-7</td>
<td>SANDY LEAN CLAY (CL); very stiff; gray, brown</td>
<td>100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX B - GENERAL DESCRIPTION OF PROCEDURES
ANALYTICAL METHODS TO PREDICT MOVEMENT

INDEX PROPERTY AND CLASSIFICATION TESTS

Index property and classification testing is perhaps the most basic, yet fundamental tool available for predicting potential movements of clay soils. Index property testing typically consists of moisture content, Atterberg Limits, and Grain-size distribution determinations. From these results a general assessment of a soil’s propensity for volume change with changes in soil moisture content can be made.

Moisture Content

By studying the moisture content of the soils at varying depths and comparing them with the results of Atterberg Limits, one can estimate a rough order of magnitude of potential soil movement at various moisture contents, as well as movements with moisture changes. These tests are typically performed in accordance with ASTM D2216.

Atterberg Limits

Atterberg limits determine the liquid limit (LL), plastic limit (PL), and plasticity index (PI) of a soil. The liquid limit is the moisture content at which a soil begins to behave as a viscous fluid. The plastic limit is the moisture content at which a soil becomes workable like putty, and at which a clay soil begins to crumble when rolled into a thin thread (1/8” diameter). The PI is the numerical difference between the moisture constants at the liquid limit and the plastic limit. This test is typically performed in accordance with ASTM D4318.

Clay mineralogy and the particle size influence the Atterberg Limits values, with certain minerals (e.g., montmorillonite) and smaller particle sizes having higher PI values, and therefore higher movement potential.

A soil with a PI below about 15 to 18 is considered to be generally stable and should not experience significant movement with changes in moisture content. Soils with a PI above about 30 to 35 are considered to be highly active and may exhibit considerable movement with changes in moisture content.

Fat clays with very high liquid limits, weakly cemented sandy clays, or silty clays are examples of soils in which it can be difficult to predict movement from index property testing alone.

Grain-size Distribution

The simplest grain-size distribution test involves washing a soil specimen over the No. 200 mesh sieve with an opening size of 0.075 mm (ASTM D1140). This particle size has been defined by the engineering community as the demarcation between coarse-grained and fine-grained soils. Particles smaller than this size can be further distinguished between silt-size and clay-size particles by use of a Hydrometer test (ASTM D422). A more complete grain-size distribution test that uses sieves to relative amount of particles according is the Sieve Gradation Analysis of Soils (ASTM D6913). Once the characteristics of the soil are determined through classification testing, a number of movement prediction techniques are available to predict the potential movement of the soils. Some of these are discussed in general below.
TEXAS DEPARTMENT OF TRANSPORTATION METHOD 124-E

The Texas Department of Transportation (TxDOT) has developed a generally simplistic method to predict movements for highways based on the plasticity index of the soil. The TxDOT method is empirical and is based on the Atterberg limits and moisture content of the subsurface soil. This method generally assumes three different initial moisture conditions: dry, “as-is”, and wet. Computation of each over an assumed depth of seasonal moisture variation (usually about 15 feet or less) provides an estimate of potential movement at each initial condition. This method requires a number of additional assumptions to develop a potential movement estimate. As such, the predicted movements generally possess large uncertainties when applied to the analysis of conditions under building slabs and foundations. In our opinion, estimates derived by this method should not be used alone in determination of potential movement.

SWELL TESTS

Swell tests can lead to more accurate site-specific predictions of potential vertical movement by measuring actual swell volumes at in situ initial moisture contents. One-dimensional swell tests are almost always performed for this measurement. Though swell is a three-dimensional process, the one-dimensional test provides greatly improved potential vertical movement estimates than other methods alone, particularly when the results are “weighted” with respect to depth, putting more emphasis on the swell characteristics closer to the surface and less on values at depth.
**POTENTIAL VERTICAL MOVEMENT**

A general index for movement is known as the Potential Vertical Rise (PVR). The actual term PVR refers to the TxDOT Method 124-E mentioned above. For the purpose of this report the term Potential Vertical Movement (PVM) will be used since PVM estimates are derived using multiple analytical techniques, not just TxDOT methods.

It should be noted that all slabs and foundations constructed on clay or clayey soils have at least some risk of potential vertical movement due to changes in soil moisture contents. To eliminate that risk, slabs and foundation elements (e.g., grade beams) should be designed as structural elements physically separated by some distance from the subgrade soils (usually 6 to 12 inches).

In some cases, a floor slab with movements as little as 1/4 of an inch may result in damage to interior walls, such as cracking in sheet rock or masonry walls, or separation of floor tiles. However, these cracks are often minor and most people consider them 'livable'. In other cases, movement of one inch may cause significant damage, inconvenience, or even create a hazard (trip hazard or others).

Vertical movement of clay soils under slab on grade foundations due to soil moisture changes can result from a variety of causes, including poor site grading and drainage, improperly prepared subgrade, trees and large shrubbery located too close to structures, utility leaks or breaks, poor subgrade maintenance such as inadequate or excessive irrigation, or other causes.

**PVM** is generally considered to be a measurement of the change in height of a foundation from the elevation it was originally placed. Experience and generally accepted practice suggests that if the PVM of a site is less than one inch, the associated differential movement will be minor and acceptable to most people.

**SETTLEMENT**

Settlement is a measure of a downward movement due to consolidation of soil. This can occur from improperly placed fill (uncompacted or under-compacted), loose native soil, or from large amounts of unconfined sandy material. Properly compacted fill may settle approximately one percent of its depth, particularly when fill depths exceed 10 feet.
SPECIAL COMMENTARY ON CONCRETE AND EARTHWORK

RERAINT TO SHRINKAGE CRACKS

One of the characteristics of concrete is that during the curing process shrinkage occurs and if there are any restraints to prevent the concrete from shrinking, cracks can form. In a typical slab on grade or structurally suspended foundation there will be cracks due to interior beams and piers that restrict shrinkage. This restriction is called Restraint to Shrinkage (RTS). In post tensioned slabs, the post tensioning strands are slack when installed and must be stressed at a later time. The best procedure is to stress the cables approximately 30% within one to two days of placing the concrete. Then the cables are stressed fully when the concrete reaches greater strength, usually in 7 days. During this time before the cables are stressed fully, the concrete may crack more than conventionally reinforced slabs. When the cables are stressed, some of the cracks will pull together. These RTS cracks do not normally adversely affect the overall performance of the foundation. It should be noted that for exposed floors, especially those that will be painted, stained or stamped, these cracks may be aesthetically unacceptable. Any tile which is applied directly to concrete or over a mortar bed over concrete has a high probability of minor cracks occurring in the tile due to RTS. It is recommended if tile is used to install expansion joints in appropriate locations to minimize these cracks.

UTILITY TRENCH EXCAVATION

Trench excavation for utilities should be sloped or braced in the interest of safety. Attention is drawn to OSHA Safety and Health Standards (29 CFR 1926/1910), Subpart P, regarding trench excavations greater than 5 feet in depth.

FIELD SUPERVISION AND DENSITY TESTING

Field density and moisture content determinations should be made on each lift of fill with a minimum of one (1) test per lift per 3,000 square feet of fill, with a minimum of one (1) test performed per lift of fill placed within the footprint of each foundation pad and deadman anchor, one (1) test per lift per 50 feet of deadman anchor perimeter backfill, one (1) test per lift per 100 linear feet of utility trench backfill, one (1) test per lift per 100 linear feet of drive aisle, and one (1) test per lift per 5,000 square feet of parking and other open pavement areas. 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.

It is recommended that all site and subgrade preparation, proof rolling, and pavement construction be monitored by a qualified engineering firm. Density tests should be performed to verify proper compaction and moisture content of any earthwork. Inspection should be performed prior to and during concrete placement operations. D&S would be pleased to perform these services in support of this project.