Purchasing Department
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ADDENDUM #1

RFP # 6899

CONSTRUCTION OF FIRE STATION #3

Issue Date: January 22, 2019
Response due Date and Time (Central Time):
Tuesday, February 12, 2019, 11:00 A.M. C.S.T

RFP # 6899

ADDENDUM #1

The following are changes to specifications:

- Add D&S Engineering Labs, Report GP 17-2115, dated June 16, 2017 to Specification 003132-Geotechnical Data

- Section 233516 Vehicle Exhaust Removal – Filtration, 1.1, A, replace "…in NJ Contact Air Technology Solutions 800-743-3323." with "by MagneGrip Group. Local representative is Bruce Sears (214) 551-7042."

NO OTHER CHANGES AT THIS TIME.

<u>Please acknowledge addendum on Attachment F of the main solicitation document when submitting a proposal.</u>

Geotechnical Engineering Report

Denton Fire Station No. 3

Denton, TX

June 16, 2017

D&S ENGINEERING LABS

D&S ENGINEERING LABS

June 16, 2017

Mr. Herman Lawson
City of Denton
869 South Woodrow
Denton, TX 76205

GEOTECHNICAL INVESTIGATION
D&S ENGINEERING #17-2115
FIRE STATION NO. 3
DENTON, TEXAS

Mr. Lawson,

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. GP17-2115, dated May 2, 2017.

We appreciate the opportunity to provide professional geotechnical engineering services for this project. At your request, we are available to discuss any questions which may arise regarding this report. Please do not hesitate to call if we can provide any additional services.

Sincerely,

D&S Engineering Labs, LLC

STATE OF TEXAS
MICHAEL T. TAYLOR
87344
LICENSED
PROFESSIONAL ENGINEER

6-16-2017

Jennifer Shields, P.G.
Senior Engineering Geologist

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TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
FIRE STATION NO. 3
DENTON, TEXAS

1.0 PROJECT DESCRIPTION

This report presents the results of the geotechnical investigation for the proposed new City of Denton Fire Station No. 3, which will be located at the intersection of McCormick Street and Underwood Street in Denton, Texas. As proposed, this project will consist of a new fire station building with an approximate footprint of 13,800 square feet, including four truck bays and sleeping quarters. A detention pond located southwest of the Fire Station will be constructed, as well as new concrete pavements and drives.

The majority of the site is currently on an undeveloped, largely unvegetated property, with mature medium sized trees present on the perimeter of the site. Recent aerial photographs depict a building located on the site that was recently razed. Portions of the new pavements will be constructed at locations currently covered with the pavements of the exsiting McCormick Street and existing Fire Station. The overall site slopes gently down to the southeast with about 4 feet of overall topographic relief. Photographs showing the recent condition of the site are provided below.

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to:

- Identify the subsurface soil strata 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 for the proposed structures and related site work, including pavements and flatwork..

The scope of this investigation consisted of:

- Drilling and sampling a total of ten (10) borings.
    - Five (5) borings were drilled within the footprint of the structure to depths of about 35 feet. One (1) boring was drilled within the detention pond to a depth of about 15 feet. Four (4) borings were advanced in the parking lot areas to depths of 5 feet.
- Laboratory testing of selected soil samples obtained during the investigation.
- Preparation of a Geotechnical Report that includes:
    - Evaluation of potential soil heave through Potential Vertical Movement (PVM) estimates.
    - Recommendations for foundation design.
    - Recommendations for earthwork.
    - Recommendations for the design of pavements and other flatwork.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 General

The borings were advanced using a truck-mounted drilling rig, outfitted with hollow stem flight augers and wet rotary coring equipment. Undisturbed samples of cohesive soil and weathered bedrock strata were obtained using 3-inch diameter tube samplers that were advanced into the soils in 1-foot increments by the continuous thrust of a hydraulic ram located on the drilling equipment. A field estimate of the unconfined compressive strength of each sample was then obtained using a calibrated hand penetrometer.

Soils and bedrock were also sampled in general accordance with the Standard Penetration Test (ASTM D 1586). 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 a 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; or 4) if there is no advancement of the sampler in any 10-blow interval.

The bedrock strata present in Boring B1 was drilled and sampled using a double-tube core barrel fitted with a tungsten-carbide, sawtooth bit. The length of core recovered (REC), expressed as a percentage of the coring interval, along with the Rock Quality Designation (RQD), is tabulated at the appropriate depths on the Log of Boring illustrations. The RQD is the sum of all core pieces longer than four inches divided by the total length of the cored interval. Pieces shorter than four inches which were determined to be broken by drilling or by handling were fitted together and considered as one piece.

Soil and bedrock materials were periodically tested in situ in all borings using the Texas Cone penetration tests in order to examine the resistance of the bedrock materials to penetration. For this test, a 3-inch diameter steel cone is driven utilizing the energy equivalent 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 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 are shown on the boring location map that is included in Appendix A. 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 soil samples recovered during the field exploration were described by either an engineering geologist or geotechnical engineer in the field. 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 ASTM and Unified Soil Classification System (USCS) procedures. Bedrock strata were described using standard geologic nomenclature.

In order to determine soil characteristics and to aid in classifying the soils, classification testing was performed on selected samples as requested by the Geotechnical Engineer. The tests were performed in general accordance with the following test procedures. The classification tests are described in more detail in Appendix B (General Description of Procedures).

- Moisture Content ASTM D 2216
- Atterberg Limits ASTM D 4318
- Percent Passing No. 200 Sieve ASTM D 1140

Additional tests were performed to aid in evaluating soil strength and volume change including:

- Unconfined Compression ASTM D 2166
- Overburden Swell Tests ASTM D 4546

The results of these tests are presented at the corresponding sample depths on the appropriate Boring Log illustrations. The classification tests are described in more detail in Appendix B (General Description of Procedures).

3.2.1 Unconfined Compression Tests

Unconfined compression strength testing was performed on selected samples of the cohesive soils and weathered bedrock. These tests were performed in general accordance with ASTM D 2166. During each test, a cylindrical specimen is subjected to an axial load that is applied at a constant rate of strain until either failure or a large strain (i.e., greater than 15 percent) occurs. Once the test is completed, the unit weight of the sample is determined based on the moisture content.

3.2.2 Overburden Swell Tests

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

4.0 SITE CONDITIONS

4.1 Stratigraphy

Based upon our observation of the recovered samples and a review of the Geologic Atlas of Texas, Sherman Sheet, this site is associated with the Woodbine Formation. Although not encountered during the field investigation, this geological formation can contain very hard sandstone boulders or “erratics” which often require coring to penetrate during pier installation.

The near surface soils comprise low to medium plasticity lean clays with varying amounts of sand and silt, clayey sands, silty sand and silt. The clays encountered are generally stiff to very stiff in consistency, reddish brown in color and contain trace roots, rock fragments, and thin sand seams. The sands are generally loose to very dense in condition, are reddish brown, orange brown, light brown and light gray in color, and contain trace roots, rock fragments, and iron oxide staining.

Below the surficial soils within the structure borings, weathered shale bedrock strata was encountered at depths of 9 to 20 feet below grade. The slightly to highly weathered shale was generally very soft to moderately hard in rock hardness and light brown, orange brown, reddish brown and gray in color, with trace silty laminations and sand seams.

Below the weathered shale layer, a slightly weathered to fresh shale layer was encountered at depths of 24 to 35 feet below grade. This shale layer was very soft to hard in rock hardness and gray to dark gray in color. This layer was encountered in all of the structure borings and extended to the maximum depth explored of 35 feet. The shale bedrock strata encountered are differentially weathered, having been leached by percolating waters over time.

Within the detention pond Boring D1, soils were encountered to the maximum depth explored of 16 feet. Weathered shale bedrock was not encountered. Near surface soils comprised clayey sand over clays with varying amounts of sand. We anticipate that excavation into these soils can be accomplished with conventional earthwork equipment and methods. These soils have a low permeability and will drain very slowly following a flood event.

Subsurface conditions at each boring location are described on the individual boring logs in Appendix A.

4.2 Groundwater

Groundwater seepage was observed in all of the structure borings at depths ranging to 8 to 30 feet. Groundwater was measured upon completion of drilling at depths of 9 to 32 feet. Borings P1 through P4, and Boring D1, were found to be dry during and upon completion of drilling. Groundwater levels should 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 a variety of 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 in a dry to wet moisture condition. Based upon the results of our analysis, the site is estimated to possess a PVM of 1-inch or less at the soil moisture conditions existing at the time of the field investigation. Please note that dry, average and wet are relative terms based on moisture content and plasticity.

6.0 FOUNDATION RECOMMENDATIONS

Considering the type of structure, the most positive foundation system for the proposed building is a straight-sided drilled shaft foundation system bearing in the slightly weathered to fresh shale. We anticipate that a straight-sided drilled pier foundation system should limit potential foundation movements to small fractions of an inch. The soils present at the site have a low potential for post-construction vertical movement with changes in soil moisture content and if post-construction movements on the order of approximately 1-inch can be tolerated a soil supported floor slab may be provided.

Recommendations for subgrade preparation to reduce potential post-construction movement are described in the Earthwork Section of this report. Please note that a soil-supported floor system may experience some vertical movement with changes in soil moisture content. Non-load bearing walls, partitions, and other elements bearing on the floor slab will reflect these movements should they occur. However, with appropriate design, adherence to good construction practices, and appropriate post-construction maintenance, these potential movements can be reduced.

6.1 Straight-sided Drilled Shafts

We recommend that structural loads for the new building be supported on auger-excavated, straight-sided, reinforced concrete drilled shafts founded in the slightly weathered to fresh shale encountered below depths of about 24 to 35 feet. We recommend those shafts penetrate a minimum of 3 feet into the slightly weathered to fresh shale to utilize the full amount of allowable end bearing. As there is appreciable strain-compatibility between the weathered and the slightly weathered to fresh shales, the side friction for both may be included in the shaft design for shafts extending into the slightly weathered to fresh shale.

Drilled shafts may be designed to transfer imposed loads into the bearing stratum using a combination of end-bearing and skin friction. Drilled shafts for structural loads should be a minimum of 18-inches in diameter and should be designed for an allowable end bearing and side friction as outlined in Table 1 below. The allowable side friction noted may be taken from below the top of the slightly to highly weathered shale or from the bottom of any temporary casing used, whichever is deeper, to resist both axial loading and uplift.

Table 1. Drilled Shaft Allowable Bearing

Material	Depth Below Current Grades (ft)	Allowable Side Friction (psf)	Allowable End Bearing (psf)
Light Brown, Orange Brown, Reddish Brown and Gray Slightly to Highly Weathered Shale	9 to 20	750	NA
Gray, Dark Gray, Slightly Weathered to Fresh Shale	24 to 35	1,200	6,000

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 approximated to be on the order of 500 pounds psf of shaft area over an average depth of 10 feet. Often, 1/2 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 that is in contact with either the slightly to highly weathered and slightly weathered to fresh shale below depths as shown in Table1.

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-sided 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 2.5 against shear failure and should limit potential movement to small fractions of an inch.

6.1.1 Drilled Shaft Construction Considerations

Groundwater seepage was encountered during drilling operations in all of the structure borings. In addition, sands with varying degrees of clay and silt were encountered in each of the structure borings. We anticipate that temporary casing will be required at many of the shaft locations for the shaft installations. Temporary casing should be available on-site in the event that excessive sidewall sloughing occurs, or if excessive groundwater seepage is encountered that cannot be controlled with conventional pumps, sumps, or other means.

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 redrilled 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

The following soils and rock geo-parameters for lateral analysis of drilled shafts for use in LPILE® or other lateral load software. These values are based on the stratigraphy, laboratory data and experience. The recommended model layers are “Stiff Clay w/o Free Water” and “Weak Rock.” The depth ranges are based on the borings drilled.

Table 2. Lateral Load Parameters – Stiff Clay w/o Free Water

Material	Depth Range (ft)	Effective Unit Wt. (pcf)	Undrained Cohesion (psf)	Strain Factor (ε_{50})
Clay, Clayey Sand	0 - 5	120	500	0.02
Clay, Clayey Sand	5 - 20	120	1,000	0.01
Slightly to Highly Weathered Shale	20 – 35	125	1,500	0.007

Table 2. Lateral Load Parameters – Weak Rock

Material	Depth Range (ft)	Effective Unit Wt. (pcf)	Initial Modulus (psi)	Uniaxial Compressive Strength (psi)	RQD	Strain Factor (K_{rm})
Slightly Weathered to Fresh Shale	35+	130	4,000	25	75	0.0005

6.1.3 Pier-Supported Grade Beams

For pier-supported grade beams, a minimum void space of 4 inches should be provided beneath all grade beams (and the floor slab if structurally suspended).

Cardboard carton forms (void boxes) may be used to provide the required voids beneath the grade beams; however, trapezoidal void boxes should not be used. Care should be taken to assure that the void boxes are not allowed to become wet or are crushed prior to or during concrete placement and finishing operations. We recommend that masonite (1/4 of an inch thick) or other protective material be placed on top of the carton forms to reduce the risk of crushing the cardboard forms during concrete placement and finishing operations. We strongly recommend the use of side retainers placed along the grade beams to prevent soil from infiltrating the void space after the carton forms deteriorate.

Grade beams may be earth-formed, but only if the sides can be cut and maintained vertically. If sloughing occurs, or if the sides cannot be maintained vertically, the grade beams should then be formed on both sides. The bottoms of all grade beam excavations should be essentially free of any loose or soft material prior to the placement of concrete. All grade beams and floor slabs should be adequately reinforced to minimize cracking as normal movements occur in the foundation soils.

If grade beams are formed, the exterior side of the grade beams around the structure should be carefully backfilled with on-site clayey soils. The backfill soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 698 (standard Proctor), and should be placed at a moisture content that is either at or above the optimum moisture, as determined by the same test. This fill should extend the full depth of the grade beam plus void space and should extend a minimum distance of 2 feet away from the exterior grade beam perimeter.

6.1.4 Soil-Supported Floor System

A soil-supported floor system that is placed directly on a modified subgrade may be used for this structure. We recommend the floor slab be at least 7 inches thick atop a 6-inch layer of aggregate base similar to the truck area pavements outside the building. A ground supported system has an increased risk of potential vertical movement resulting from subgrade soil volume changes, which may occur as a result of changes in soil moisture content. The majority of such movement typically occurs in the perimeter 10 feet of buildings with slabs constructed with Finished Floor Elevations near those of the final exterior grades and any walls bearing on the slab in the areas of movement may exhibit distress. We recommend that the subgrade be prepared according to the Earthwork section of this report in order to reduce the potential for post-construction movement. The floor slab should be doweled to the beams at the locations of the doors in order to prevent vertical steps from forming at these high-traffic areas.

In order to reduce the effects of seasonal moisture fluctuations and subsequent possible soil movement beneath soil-supported floor slabs as described above, consideration is often given to the installation of a horizontal barrier around the perimeter of the structure. This barrier may be in the form of an independent barrier, such as a minimum 5-foot wide sidewalk. The joints between the building and any sidewalks and pavements should be sealed and the seals inspected periodically and re-sealed as necessary through the life of the structure.

We understand that sidewalks are not always practical or desired around the full perimeters of some facilities. Where landscaping will be present adjacent to building perimeters, diligent post-construction maintenance should be employed to prevent excessive wetting or drying of those adjacent soils.

7.0 EARTHWORK RECOMMENDATIONS

The near-surface soils present generally have a low potential for post-construction vertical movement with changes in subsurface soil moisture changes. For a soil supported slab in conjunction with a pier and beam foundation system, we have the following recommendations for earthwork. Please note that more stringent tolerances limiting potential post-construction vertical movement will require more extensive effort.

7.1 Soil Rework

- Strip the site of all, vegetation, organic soil, and deleterious material within the new building area. Typically, 6 inches is sufficient for this purpose.
- After stripping and after performing any necessary grade cuts, excavate an additional 2 feet of material and stockpile for re-use. The excavation should extend at least 5 feet beyond the perimeter of the new structure.
- After excavating, scarify, rework, and recompact the exposed stripped subgrade to a depth of 12 inches. The scarified and reworked soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 698 (standard Proctor), and placed at a moisture content that is in the range of zero to three percentage points (0 to +3) above the optimum moisture content, as determined by the same test. This procedure should extend at least 5 feet beyond the perimeter of the new structure.
- Within 24 hours of recompacting the reworked excavated exposed subgrade, begin any fill operations necessary to within 6 inches of the bottom of the floor slab. The fill soil should be placed in maximum 8-inch compacted lifts, be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 698 (standard Proctor), and be placed at a moisture content that is in the range of zero to three percentage points (0 to +3) above the optimum moisture content, as determined by that same test.

- Fill soils within the building pad area maybe on-site natural, blended, or imported soils with a maximum PI of 20.
- Provide a minimum of 6 inches of aggregate base on top of the subgrade surface as outlined in Section 8.4.1.
- 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.
- Place a minimum 10-mil thick vapor retarder or 15-mil thick vapor barrier beneath all floor slabs.
- 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, and with a minimum of three (3) tests performed per lift of fill placed within the footprint of the building pad and one test per 100 linear feet of utility trenches. D&S would be pleased to provide these services in support of this project.

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. Rainfall roof runoff should be collected and conveyed to downspouts. Downspouts should be directed to discharge at least 5 feet away from the foundations
- The moisture content of subgrade soils that are in proximity to the structures should be maintained as close as possible to a consistent level throughout the year. However, we strongly recommend that excessive watering near foundations be avoided.

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 provide the final pavement design and associated specifications for this project.

8.2 Behavior Characteristics of Expansive Soils beneath Pavement

Near-surface soils at this site are considered to generally 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 as a result of surface water percolation or poor drainage, the soils can heave and/or soften, causing distress to pavements in contact with the soil in the form of cracks.

The edges of pavement are particularly prone to moisture variations, and so these areas therefore often experience the most distress. 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 expansive soil on paved areas and to improve the long-term performance of the pavement, we have the following recommendations:

- Design a crowned pavement. A minimum slope of five percent within the first 5 feet from the edge of the pavement is considered ideal.
- Subgrade treatments should to extend to at least 2 feet beyond the back of curbs or edges of pavements
- Avoid long areas of low-sloping roadway and adjust slopes to provide maximum drainage away from pavement edges.

8.3 Subgrade Strength Characteristics

We recommend that a California Bearing Ratio (CBR) value of 3 be used in the design with a corresponding resilient modulus of 4,500 psi. We recommend using a Modulus of Subgrade Reaction (k) of 100 pci for the subgrade soils prepared in accordance with the recommendations in this report.

8.4 Pavement Subgrade Preparation Recommendations

Due to the variable nature of the soils within the pavement areas we recommend that the subgrade beneath concrete pavements include an aggregate base course to improve stability and provide an adequate and uniform surface for the pavement. We have the following additional recommendations for pavement subgrade preparation.

- Remove all surface vegetation, including tree root balls and root mats, and similar unsuitable materials from within the limits of the project. We anticipate a typical stripping depth of about 6 inches.

- Perform any cut operations as-needed. We anticipate that excavation of overburden soils can be accomplished with conventional earthwork equipment and methods.
- After stripping and performing necessary cuts, the exposed subgrade should be proof rolled. Proof rolling should consist of rolling the entire pavement subgrade with a heavily-loaded, tandem-axle dump truck weighing at least 20 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 must be performed under the observation of a qualified geotechnical engineer. D&S would welcome the opportunity to perform these services for this project.
- After proof rolling, all exposed surfaces should be scarified and reworked to a depth of 12 inches. The soils should then be recompacted to a minimum of 95 percent of the maximum dry density obtained in accordance with ASTM D 698 (standard Proctor), and to a moisture content that is at or above the material's 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 8-inch compacted lifts, compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 698 (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.
- Field density tests should be performed at a minimum rate of one test per each 5,000 square feet, per lift, for all compacted fills in drive areas, fire lanes and truck areas. The frequency of testing can be reduced to one test per lift every 10,000 square feet in parking areas (minimum 3 tests per lift). Earthwork operations, including proof rolling, should be observed and tested on a continual basis by an experienced technician working under the supervision of a licensed geotechnical

engineer. D&S would be pleased to provide those services in support of this project.

- 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 must 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 materials should consist of lean clays (maximum Plasticity Index of 30) that are essentially free of organic materials and particles larger than 4 inches in their maximum dimension.

8.4.1 Aggregate Base

- After proof rolling is completed and any grade raise fill has been placed, install a 5-inch thick layer of aggregate base for parking areas and a 6-inch thick layer in drive areas and areas that will be subjected to truck traffic. The area of aggregate base should extend a minimum of 2-feet beyond the back of roadway curbs or edges of 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 to a minimum of 95% of the maximum standard Proctor dry density (ASTM D 698) and placed at a moisture content that is sufficient to achieve density.
- Field density and moisture content testing should be performed at the rate of one test per 10,000 square feet in pavement areas.

8.5 Rigid Pavement

We recommend that reinforced Rigid Portland Cement Concrete for this site have a minimum thickness of 7 inches for all fire lanes and truck areas. A minimum thickness of 5 inches is recommended for automobile parking. The reinforced concrete paving should be placed over an aggregate base course.

We have the following concrete mix design recommendations:

- Recommended minimum design compressive strength: 3,500 psi.

- 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.

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 other suitable flexible sealant may be used to seal the joints. Maintenance should include periodic inspection of these joints and 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.

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 D 698 (standard Proctor), and placed at a moisture content that is at least or above the optimum moisture content, as determined by that same test. Field density and moisture content testing should be performed at the rate of one test per 100 linear feet in utility trenches. 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 excavations should be sloped so that water within excavations will flow to a low point away from the building where it can be removed from 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.

9.3 Surface Drainage

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

The location of gutter downspouts, and other features, should be designed such that these items will not create moisture concentrations at or beneath the structure or flatwork. Downspouts should discharge well away from the structure, and should not be allowed to erode surface soil.

Moisture related issues can be positively addressed by constructing continuous exterior flatwork that extends to the building line. Where this occurs, the joints created at the interface of the flatwork and building line should be sealed with a flexible joint sealer to prevent the infiltration of water. Open cracks that may develop in the flatwork should also be sealed. The joint and any cracks that develop should be resealed as they become apparent, and should be part of a periodic inspection and maintenance program.

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 cases should the drip-line of the mature tree extend closer than 10-feet of rooflines. To the extent practical, it is recommended that trees scheduled for removal (where required) in the vicinity of the proposed structure and pavements be removed as far in advance of slab construction as possible, ideally by several months or longer. This will tend to restore a more favorable soil moisture equilibrium which will, in turn, tend to minimize the potential for greater than anticipated post-construction ground movements. A moist but not overly wet soil condition should be maintained at all times in all landscaped areas near the building after construction to minimize soil volume changes caused by changing soil moisture conditions.

10.0 SEISMIC CONSIDERATION

Based on the boring log data and general geologic information gathered, we recommend that Soil Site Class “C” be used at this site.

11.0 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 largely chosen by others to obtain geotechnical information for the design and construction of lightly to moderately loaded municipal structure foundations and pavements. The information and recommendation contained herein should not be used as a basis for final design. 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, or if the Scope or configuration of the project changes from has is described herein, our office should be contacted immediately since this may require significant modifications to the recommendations provided in this report.

APPENDIX A - BORING LOGS AND SUPPORTING DATA

BORING LOCATIONS ARE INTENDED FOR GRAPHICAL REFERENCE ONLY

D&S ENGINEERING LABS

N.T.S.

DENTON

PLAN OF BORINGS

DENTON FIRE STATION NO. 3

TEXAS

SHEET NO.

G1

DATE DRILLED

May 26, 2017

D&S ENGINEERING LABS

KEY TO SYMBOLS AND TERMS

LITHOLOGIC SYMBOLS

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		
Consistency	SPT (# blows/ft)	UCS (tsf)
Very Soft	0 - 2	< 0.25
Soft	3 - 4	0.25 - 0.5
Medium Stiff	5 - 8	0.5 - 1.0
Stiff	9 - 15	1.0 - 2.0
Very Stiff	16 - 30	2.0 - 4.0
Hard	> 30	> 4.0

CONDITION OF SOILS

CONDITION: COARSE GRAINED SOILS			
Condition	SPT (# blows/ft)	TCP (#blows/ft)	Relative Density (%)
Very Loose	0 - 4	< 8	0 - 15
Loose	5 - 10	8 - 20	15 - 35
Medium Dense	11 - 30	20 - 60	35 - 65
Dense	31 - 50	60 - 100	65 - 85
Very Dense	> 50	> 100	85 - 100

SECONDARY COMPONENTS

QUANTITY DESCRIPTORS	
Trace	< 5% of sample
Few	5% to 10%
Little	10% to 25%
Some	25% to 35%
With	> 35%

RELATIVE HARDNESS OF ROCK MASS

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

WEATHERING OF ROCK MASS

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

D&S ENGINEERING LABS

UNIFIED SOIL CLASSIFICATION SYSTEM

ADAPTED FROM ASTM D 2487

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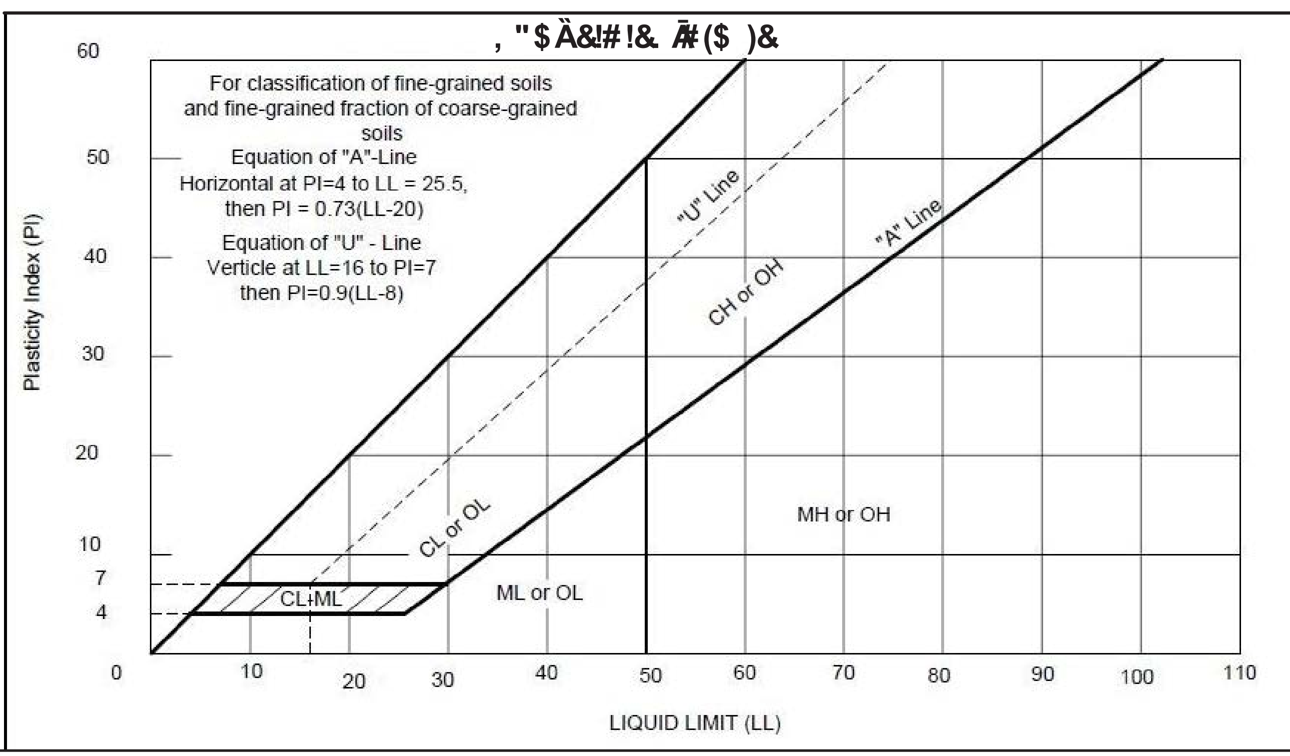


BORING LOG

B1

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.202958, W97.147682
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Kevin Kavadas (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Description	Strata Depth / Elevation	REC (%) RQD (%)	MC (%)	LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+	**LEAN CLAY (CL);** very stiff, dark brown, reddish brown, trace rock fragments, thin sand seams			12.6									
	S	4.5+													
	S	4.5+				14.6	29	12	17				0.1	119.0	
	T	8,11													
5	S	4.5+				12.2									
	T	14,22		6.0 ft / 672.0 ft											
	S	2.5	**CLAYEY SAND (SC);** medium dense, light gray			15.0				20					
	T	22,21		8.0 ft / 670.0 ft											
	S	4.5+	**LEAN CLAY (CL);** very stiff, light brown to reddish brown and gray, sand seams, trace rock fragments	9.0 ft / 669.0 ft		15.7	23	14	9						
10	S	4.5+	**SHALE;** highly weathered, very soft, light brown, orange brown and gray												
	T	9,11													
15	S	4.5+													
	T	21,19													
20	S	4.5+													
	T	18,26		24.0 ft / 654.0 ft											
25	S	4.0	**SHALE;** slightly weathered to fresh, medium hard, dark gray			12.8								115.3	3.6
	T	50=1.5" 50=0.5"													
30	S	4.5+													
	T	50=1.0" 50=0.5"													
	C				0 / 0										
35	T	50=3.0" 50=3.0"		35.5 ft / 642.5 ft											
40			End of boring at 35.5'												

Notes:
-seepage at 28.5 feet during drilling
-water at 11 feet for coring purposes

D&S ENGINEERING LABS

BORING LOG

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203156, W97.147378
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+	**SANDY LEAN CLAY (CL);** very stiff, reddish brown, trace rock fragments											
	S	4.5+			8.0	23	14	9				1.0	122.3	
	S	4.5+												
	T	8,7												
5	S	4.5+			12.9								114.2	3.8
	T	14,16	6.0 ft											
	S	0.5	672.0 ft **CLAYEY SAND (SC);** dense, orange brown, silty deposits		11.4	20	14	6	22					
	T	29,50=3.0"												
	S	0.5												
10	S	0.5	10.0 ft		12.1									
	T	38,38	668.0 ft **SHALE;** moderately weathered, very soft to moderately hard, light brown, trace silty laminations											
15	S	4.5+												
	T	26,42												
20	S	4.5+			15.6								113.2	11.1
	T	50=4.0" 50=1.0"												
25	S	4.5+	24.5 ft											
	T	50=1.25"	653.5 ft **SHALE;** slightly weathered to fresh, very soft to moderately hard, dark gray, trace silty laminations											
30	S	4.5+			16.4								111.9	1.6
	T	36,42												
35	S	4.5+	35.3 ft											
	T	50=3.0" 50=1.0"	642.7 ft End of boring at 35.3'											
40														

Notes:
-seepage at 30 feet during drilling
-water at 32 feet upon completion

BORING LOG

B3

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017   FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203142, W97.147606
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ▽ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.0	**SANDY LEAN CLAY (CL);** stiff to very stiff, reddish brown to dark brown, trace roots, trace rock fragments											
	S	2.0			8.4									
	S	3.25			10.9									
	T	5,7												
5	S	3.0			14.2	29	14	15				0.1	118.7	
	T	14,23	6.0 ft											
	S	1.0	**CLAYEY SAND;** medium dense, reddish brown, trace silty laminations (672.0 ft)		15.0				32					
	T	25,16												
	S	0.5												
10	S	0.5			15.7									
	T	30,30	14.5 ft											
15	S	4.25	**SHALE;** moderately weathered, medium hard to moderately hard, light brown to gray, trace silty laminations (663.5 ft)											
	T	50=2.0" 50=1.0"												
20	S	4.25												
	T	50=2.5" 50=3.0"	24.0 ft											
25	S	4.5+	**SHALE;** slightly weathered to fresh, very soft to soft, gray to dark gray (654.0 ft)		17.5								112.5	6.8
	T	21,36												
30	S	4.5+												
	T	50=3.5",46												
35	T	50=5.25" 50=1.0"	35.5 ft											
40			End of boring at 35.5' (642.5 ft)											

Notes:
-seepage at 8 feet during drilling
-water at 9 feet at completion

BORING LOG

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek
LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203299, W97.147483
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Kevin Kavadas (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ▽ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Material Description	REC (%) RQD (%)	MC (%)	LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+	**CLAYEY SAND (SC);** loose to medium dense, reddish brown, trace roots, trace rock fragments											
	S	4.5+			10.3				44					
	S	4.5+			13.7	26	12	14				0.0	120.2	
	T	8,7												
5	S	4.5+			13.8									
	T	10,13												
	S	4.5+			10.4									
	T	50=4.0" 50=1.5"												
	S								18					
10	N	13,50=3.5"												
	T	50=2.75" 50=1.0"												
15	N	18,26 50=3.0"				NP	NP	NP	12					
20	N	29,28,48	20.0 ft / 658.0 ft — **SHALE;** highly weathered, very soft to soft, reddish brown to light brown, gray sand seams											
25	N	19,43, 50=3.5"												
30	N	10,31,22												
35	T	50=1.5" 50=0.5"	35.0 ft / 643.0 ft — **SHALE;** slightly weathered to fresh, hard to very hard, reddish brown to light brown, gray sand seams — 35.2 ft / 642.8 ft											

End of boring at 35.2'

Notes:
-seepage at 25 feet during drilling
-water at 12 feet at completion

BORING LOG

B5

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203104, W97.147899
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+		**SANDY LEAN CLAY (CL);** very stiff, reddish brown, trace rock fragments											
	S	4.5+													
	S	4.5+				11.3	32	13	19						
	T	40,50=4.0"		4.0 ft											
5	S	2.0		**CLAYEY SAND (SC);** very dense, light brown to reddish brown — 674.0 ft		5.7				23					
	T	50=3.5" 50=0.5"													
	S	3.0				13.0	26	11	15						
	T	18,50=3.25"		8.0 ft											
	S	4.5+		**SILTY SAND (SM);** dense, light brown — 670.0 ft		16.5	18	16	2						
10	S	4.5+				12.4	19	12	7				0.0	120.5	
	T	28,37													
				14.0 ft											
15	S	3.5		**SHALE;** moderately weathered, soft to medium hard, brown — 664.0 ft		17.4								112.3	4.2
	T	50=1.0" 50=2.0"													
20	S	4.5+													
	T	50=1.0" 50=2.0"													
25	T	50=3.0" 50=3.0"		26.0 ft											
				SHALE; slightly weathered to fresh, medium hard, dark gray — 652.0 ft											
30	T	50=1.0" 50=2.5"													
35	T	50=3.5" 50=2.0"		35.5 ft											
				End of boring at 35.5' — 642.5 ft											
40															

Notes:
-seepage at 25 feet during drilling
-water at 30 feet upon completion

BORING LOG

D1

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.202804, W97.148117
GROUND ELEVATION: Approx. 676 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ▽ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	3.0		**CLAYEY SAND (SC);** loose, light brown, trace roots and trace iron oxide staining											
	S	3.5		2.0 ft		9.5	29	12	17	43					
	S	3.5		**LEAN CLAY WITH SAND(CL);** very stiff, orange brown, with sand — 674.0 ft											
	T	4,3													
5	S	4.0													
	T	14,32													
	S	4.5+													
	T	16,10													
	S	4.5+				13.4	25	12	13	76					
10	S	4.0													
	T	40,45													
				14.0 ft											
15	S	4.0		**SANDY LEAN CLAY (CL);** very stiff, brown, few iron oxide stains — 662.0 ft						63					
	T	23,26		16.0 ft											
				End of boring at 16.0' — 660.0 ft											

Notes:
-dry upon completion

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

BORING LOG

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203009, W97.147396
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.0		**SILT (ML);** dense, reddish brown, trace rock fragments		12.0	18	15	3						
	S	3.0		2.0 ft											
	S	4.5+		**LEAN CLAY (CL);** very stiff, light brown (676.0 ft)											
	S	4.5+													
5	S	4.5+		5.0 ft											
				End of boring at 5.0' (673.0 ft)											

Notes:
-dry upon completion

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

BORING LOG

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203223, W97.147007
GROUND ELEVATION: Approx. 676 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- ■ S-Shelby Tube
- ☒ N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ▽ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+		**SANDY LEAN CLAY (CL);** very stiff, brown, trace roots, few rock fragments											
	S	4.5+				11.6	35	12	23						
	S	4.5+													
	S	4.5+													
5	S	4.5+		5.0 ft											
				End of boring at 5.0' 671.0 ft											
10				Notes: -dry upon completion											
15															
20															
25															
30															
35															
40															

D&S ENGINEERING LABS

BORING LOG

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203332, W97.147641
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+		**SILTY CLAY (CL-ML);** very stiff, reddish brown, trace roots, trace iron oxides		8.1	20	13	7						
	N	15,18,14													
	N	14,19,33													
5	S	4.5+		5.0 ft											
				End of boring at 5.0' 673.0 ft											

Notes:
-dry upon completion

BORING LOG

D&S ENGINEERING LABS

PROJECT: Denton Fire Station #3
CLIENT: City of Denton
PROJECT NUMBER: G17-2115
START DATE: 5/26/2017 FINISH DATE: 5/26/2017
LOGGED BY: Mike Mielcarek

LOCATION: 1204 McCormick Street, Denton, Texas
GPS COORDINATES: N33.203239, W97.148106
GROUND ELEVATION: Approx. 678 feet
DRILL METHOD: Cont. Flight Auger
DRILLED BY: Octavio Herrera (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ∇ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5		**SILTY CLAY (CL-ML);** very stiff, reddish brown, trace iron concretions, thin sand seams											
	S	1.5				13.6	21	14	7						
	N	22,11,32													
5	N	27,16,11		5.0 ft											
				End of boring at 5.0' 673.0 ft											

Notes:
-dry at completion

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

D&S ENGINEERING LABS

SWELL TEST RESULTS

PROJECT: Denton Fire Station #3

CLIENT: City of Denton

PROJECT NUMBER: G17-2115

LOCATION: 1204 McCormick Street, Denton, Texas

Boring Number	Depth feet	Initial Moisture Content, %	Final Moisture Content, %	Applied Pressure, psf	Vertical Swell, %
B1	2-3	14.6	15.1	391	0.1
B2	1-2	8.0	16.2	260	1.0
B3	4-5	14.2	15.5	658	0.1
B4	2-3	13.7	14.2	390	0.0
B5	9-10	12.4	16.7	1303	0.0

APPENDIX B - GENERAL DESCRIPTION OF PROCEDURES

ANALYTICAL METHODS TO PREDICT MOVEMENT

CLASSIFICATION TESTS

Classification testing is perhaps the most basic, yet fundamental tool available for predicting potential movements of clay soils. Classification 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 D 2216.

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 D 4318.

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 classification 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 D 1140)). 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 D 422). A more complete grain-size distribution test that uses sieves to relative amount of particles according is the Sieve Gradation Analysis of Soils (ASTM D 6913). 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.

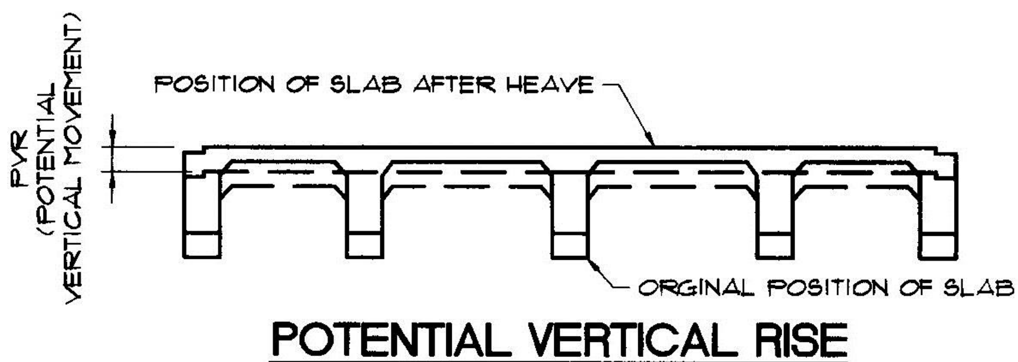
It should be noted that slabs and foundations constructed on clay or clayey soils may have at least some risk of potential vertical movement due to changes in soil moisture contents. To eliminate that risk, slabs and foundation elements may be designed as structural elements physically separated by some distance from the subgrade soils (usually 4 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 'liveable'. 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 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. The potential for post-construction vertical movement can be minimized through adequate design, proper construction, and adherence to the recommendations contained herein for post-construction maintenance.

POTENTIAL VERTICAL MOVEMENT (PVM)

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.

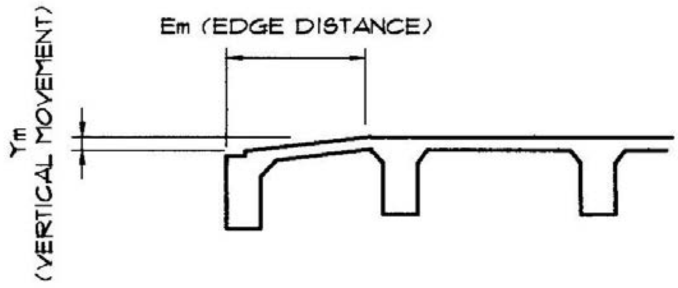


SETTLEMENT

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 1 percent of its depth, particularly when fill depths exceed 10 feet.

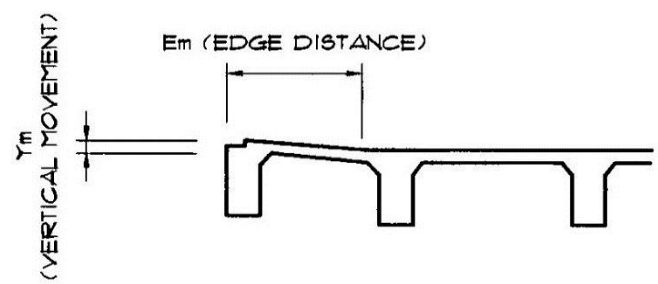
EDGE AND CENTER LIFT MOVEMENT (y_m)

The Post-Tensioning Institute (PTI) has developed a parameter of movement defined as the differential movement (y_m) estimated using the change in soil surface elevation in two locations separated by a distance e_m within which the differential movement will occur; e_m being measured from the exterior of a building to some distance toward the interior. All calculations for this report are based on the modified PTI procedure in addition to our judgment as necessary for specific site conditions. The minimum movements given in the PTI are for climatic conditions only and have been modified somewhat to account for site conditions which may increase the actual parameters.



CENTER LIFT PARAMETERS

"Center lift" occurs when the center, or some portion of the center of the building, is higher than the exterior. This can occur when the soil around the exterior shrinks, or the soil under the center of the building swells, or a combination of both occurs.



EDGE LIFT PARAMETERS

"Edge lift" occurs when the edge, or some portion of the exterior of the building, is higher than the center. This can occur when the soil around the exterior swells. It is not uncommon to have both the center lift and the edge lift phenomena occurring on the same building, in different areas.

SPECIAL COMMENTARY ON CONCRETE AND EARTHWORK

RESTRAINT 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 percent 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

Construction observation and testing by a field technician under the direction of a licensed geotechnical engineer should be provided. Some adjustments in the test frequencies may be required based upon the general fill types and soil conditions at the time of fill placement.

We recommend that all site and subgrade preparation, proof rolling, and pavement construction be monitored by a qualified engineering firm. D&S would be pleased to provide these services in support of this project. 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.

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