FUGRO CONSULTANTS, INC.

GEOTECHNICAL INVESTIGATION

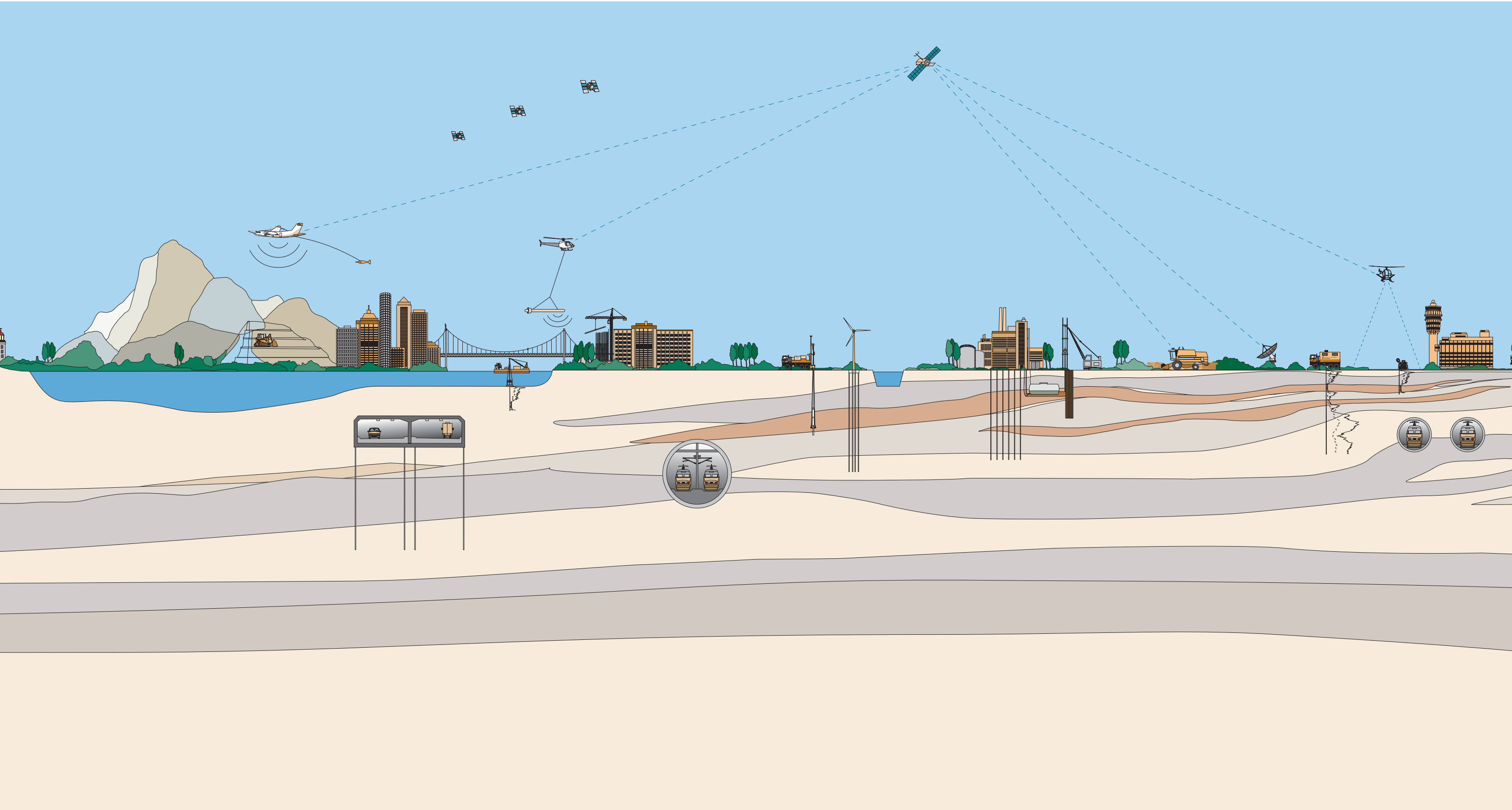
TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS

PROJECT NO. 04.4010-1069

Report to:

TARRANT COUNTY FACILITY MANAGEMENT
FORT WORTH, TEXAS

NOVEMBER 2010



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FUGRO CONSULTANTS, INC.

Report No. 04-4010-1069
November 9, 2010

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Tarrant County Facility Management
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Fort Worth, Texas 76196

Attention: Ms. Tracy Pelle

GEOTECHNICAL INVESTIGATION
TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS

Dear Ms. Pelle:

This report presents the results of a geotechnical investigation performed for the referenced project in Fort Worth, Texas. This study was performed in accordance with our Proposal No. P04-4010-1069 dated September 2, 2010 and revised September 10, 2010.

Our engineering analyses as well as the results of the field and laboratory investigations are included in this report. Our firm is interested in providing the professional material testing that will be required during the construction phase of the project.

We appreciate the opportunity to be of assistance on this project. Please feel free to contact us if you have any questions or if we can be of further service.

Very truly yours,

FUGRO CONSULTANTS, INC.
TBPE Firm Registration No. F-299

ORI□INAL SI□NED

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INTRODUCTION

Project Description

We understand the proposed project will consist of the design and construction of a new Civil Courts building in Fort Worth, Texas. The new Civil Courts building will be bounded by Belknap, Jones, Weatherford and Calhoun streets in downtown Fort Worth. We understand that the building will consist of six levels above grade and one level below grade. The basement depth is anticipated to be about 15 feet below existing grade. The proposed foundation support for concentrated column loads will be about 20 to 2,500 kips.

Based on our observation at the site, the ground surface is generally level. The general location of the site is shown on the Site Vicinity Map as Plate A. The general site orientation and the envisioned layout of the proposed facility are shown on the Site and Boring Plan as Plate B.

Scope of Work

The purpose of this study was: 1) to evaluate the subsurface and depth-to-water conditions at selected boring locations at the site, and 2) to provide geotechnical recommendations for the design and construction of foundations for the proposed structures. More specifically, the scope of work included:

1. Exploration and evaluation of the soil and rock strata at the boring locations;
2. Evaluation of soil swell potential;
3. Recommendations for suitable foundation types and design parameters; and
4. Recommendations for site preparation, drainage and landscaping.

Report Format

The first sections of the report describe the field and laboratory phases of the study. The remaining sections present our geotechnical recommendations to guide design and preparation of plans and specifications. Boring logs and laboratory test results are presented in the report **Illustrations**.

FIELD INVESTIGATION

The field portion of this project was accomplished by drilling 6 borings to depths of 5 to 58 feet below existing grade. Borings B-4 and B-6 were originally planned to be drilled at least 30 feet into the bedrock. During drilling it was noticed that the soil in these boring locations exhibited a hydrocarbon odor. Due to the contamination encountered, the drilling was haltered at depths of about 5 to 7 feet. The approximate locations of the borings are shown on Plate B. Logs of the borings drilled for this study with descriptions of the subsurface materials are presented on Plates 1 through 6. A key to the terms and symbols used on the boring logs is presented on Plates 7 and 8.

The borings were drilled with a truck-mounted drilling rig. The rig used for this project was equipped with: 1) continuous flight augers for advancing the boreholes without drilling fluids; 2) thin-walled tube samplers for obtaining undisturbed samples of cohesive soils; 3) a split-barrel sampler for obtaining disturbed samples of cohesionless soils or hard materials in conjunction with the Standard Penetration Test, with the penetration resistance to driving the sampler, or “N-values,” recorded in the field; and 4) Texas Department of Transportation Cone Penetrometer to obtain an indication of the general strength of harder formations or rock. The resistance or penetration for 100 blows is recorded on the logs (i.e., 100/6.5”, etc.).

Soil samples were obtained at selected intervals. The bedrock was cored using N-size coring equipment. For the cored bedrock samples, the percentage of core recovered was recorded in the “Percent Recovery” column on the boring logs. Along with the recovery, the Rock Quality Designation (RQD) for each core run is also included. The RQD is defined as the ratio (percent) of cumulative length of rock core segments 4 inches long or longer to the total core run.

Upon recovery from the borehole, each soil sample was extruded from the sampler and visually classified. To aid in field classification, the undrained shear strength of cohesive soil samples was estimated using a calibrated hand penetrometer. The hand penetrometer values, or “P-values,” in tons per square foot (tsf), are shown on the boring logs. All samples were sealed in plastic bags to preserve the moisture content and transported to our laboratory for further observation and testing. After completion of the field investigation, the borings were backfilled with cuttings and plugged at the surface.

LABORATORY TESTING

Samples obtained during the field exploration were observed and classified by a geotechnical engineer. Laboratory tests were assigned and performed to help evaluate the engineering properties of the soils encountered. These tests included visual classification, moisture content, dry unit weight, Atterberg limits, unconfined compressive strength and overburden swell. The tests were performed in general accordance with applicable ASTM test procedures. ASTM test designations used for these tests include visual classification (ASTM D 2488), moisture content (ASTM D 2216), unconfined compression for soil (ASTM D 2166), unconfined compression for rock (ASTM D 7012), Atterberg limits (ASTM D 4318) and overburden swell (ASTM D 4546).

The results of index, classification, and strength tests are presented on the boring logs, Plates 1 through 6. The soils were classified according to the Unified Soil Classification System based on visual observation of the samples and laboratory test results. Selected samples of cohesive soil were subjected to overburden swell tests. Test results are recorded as percent swell, with associated initial and final moisture contents. The results of the overburden swell tests are shown in Plate 9.

GENERAL SUBSURFACE CONDITIONS

Geology and Soil / Rock Stratigraphy

Based on available geological maps, the site is located within the Fort Worth Limestone/ Duck Creek formation (undivided) as shown on Plate C. The subsurface materials of this formation, generally, consist of clays underlain by very pale brown weathered limestone and gray limestone. The clays of this formation can exhibit high shrink or swell potential with variations in moisture content.

The subsurface materials encountered in each of the borings are described in the attached boring logs. The stratification boundaries shown on the boring logs represent the approximate locations of the changes in the soil and rock types; in situ, the transition between material types may be gradual and indistinct. The following table summarizes the rock stratigraphy encountered at the boring locations:

Boring No.	Depth to Top of Weathered Limestone, ft.	Depth to Top of Gray Limestone, ft.
B-1	10.0	17.0
B-2	2.5	12.0
B-3	7.0	14.5
B-4	3.5	NE*
B-5	2.0	12.0
B-6	4.0	NE*

** NE = Not encountered.*

Based on experience with these materials, the gray limestone bedrock should be considered as hard to very hard (sedimentary rock basis) and some difficult drilling should be anticipated through the gray limestone.

Groundwater

The borings were advanced using auger-drilling methods that allowed observations of groundwater seepage levels as the borings were advanced. The boring locations were observed to be dry during and at completion of drilling. The groundwater flow quantities depend on local permeability, seasonal rainfall conditions, and other factors. Future construction activities may also alter the surface and subsurface drainage characteristics of this site. In our experience, groundwater is often observed to fluctuate at shallow depths in the overlying soils during and after periods of wet weather. If a noticeable change from the conditions is observed prior to construction, then we should be notified immediately to review its effect on the design recommendations.

Site Seismic Classification

North Central Texas is generally regarded as an area of low seismic activity. Due to the depth to bedrock and based on the results of the field and laboratory tests conducted for this investigation, review of the available geologic mapping, and site class definitions shown in Tables 1613.5.2 and 1613.5.5 of the 2006 International Building Code (IBC), it is our opinion that the subject site be classified as Site Class C with a soil profile name of “Very Dense Soil and Soft Rock” category.

ENGINEERING ANALYSIS AND RECOMMENDATIONS

Minimum Foundation Depth

A maximum frost penetration depth on the order of one foot is generally assumed for foundation design in the north central and northeast Texas area. However, due to the possibility of localized changes in soil moisture content in the expansive clays around the foundation perimeter, it is recommended that all foundation elements bear at a depth of at least 1.5 feet below final exterior grade.

Expansive Soils

The near surface soils encountered in the borings drilled exhibited plasticity indices (PI) ranging from 15 to 43. These soils are considered as slightly to moderate expansive and will experience some vertical movement with changes in moisture conditions. The results of the tests performed during this investigation indicate that the soils were at a relatively dry to average moisture state at the time the borings were drilled. The magnitude of the moisture-induced potential vertical movement (PVM) beneath covered areas calculated using TxDOT Method Tex-124-E and in conjunction with free swell tests, is estimated to be up to 2 inches at dry conditions.

It should be noted that the TxDOT method of calculating PVM is empirical and is based on the results of the Atterberg limits and moisture content of the soils. Swell tests will provide swell data based on the existing moisture profile of the subsurface soils at the time the borings were advanced. The total swell potential depends on the moisture content of the expansive soils within the zone of moisture changes. The drier the expansive soil is, the higher swell potential will be. Considerably more movement will occur in areas where ponding of free water is allowed to occur at the ground surface. For this reason, care should be taken adjacent to the building, so that water ponding is not allowed to occur during or after construction.

Drilled Pier Foundations

Due to the type of structure and the subsurface soil conditions encountered at the site, the most positive foundation system would consist of auger excavated, cast-in-place reinforced concrete straight-shaft drilled piers. The drilled piers should bear in the gray limestone encountered at a depth of about 12 to 17 feet below existing ground surface. The drilled piers can be designed for a maximum allowable end bearing capacity of up to 40,000 pounds per square foot (psf) and should penetrate a minimum depth of 3 feet into the gray limestone or one shaft diameter, whichever is deeper. The length of the piers should be at least 10 feet below the final grade. In addition to end bearing, a maximum allowable skin friction of 6,000 psf can be assumed for compressive loads. The skin friction should be applied to that portion of the drilled pier in direct contact with the gray limestone below the minimum penetration depth or below any temporary casing (if used).

The drilled piers should also be designed to resist potential uplift loads due to swelling of the expansive clays in contact with the pier shafts. This uplift load can be approximated by assuming an uplift skin friction of 800 psf acting over a depth of 10 feet. To resist the uplift loads, an allowable resistance to uplift of 5,000 psf can be assumed in the gray limestone (below the minimum penetration of 3 feet). The drilled shafts should be reinforced with sufficient, full-depth, vertical reinforcing steel to resist uplift forces.

The above allowable bearing capacity and skin friction values include a factor of safety of approximately 3 against bearing or bond failure in the rock sockets. Settlement will primarily be within the elastic range with a portion of settlement occurring during construction. Settlements of properly constructed drilled piers should be limited and not exceed about ½ inch. A reduction of the allowable skin friction will be required if the drilled piers are placed closer than 3 pier diameters apart, measured center to center. A reduction factor of 75 percent should be used if the drilled piers are placed at a spacing between 2 and 3 shaft diameters apart, measured center to center. The factor should be reduced to 50 percent if the piers are placed less than 2 shaft diameters apart, measured center to center.

Resistance Against Lateral Loads. Foundations are subjected to lateral loads due to a variety of forces. As such, these forces must be considered as part of the overall foundation design. Forces transmitted to drilled shafts will be resisted by the lateral resistance developed by the drilled shaft interacting with the surrounding subsurface soils and bearing materials. The upper

5 feet of soil below the finished grade should be neglected in passive resistance to allow for soil shrinkage.

Based on the subsurface conditions encountered in the borings, the following parameters may be used for the analysis using a computer program with the p-y curve method such as LPILE.

LPILE Design Parameters

[1]Stratum	[2]γ_d (pcf)	[3]RQD (%)	[4]C (ksf)	[5]Φ (degree)	[6]E_{50}	[7]E_s (psi)	[8]k (pci)
Very Pale Brown Limestone	120	70	5	0	N/A	N/A	N/A
Gray Limestone	130	90	15	0	0.005	$2x10^4$	N/A

Notes: (1) γ_d is dry unit weight.
(2) RQD is the rock quality designation index.
(3) C is undrained cohesion.
(4) Φ is the internal Friction Angle.
(5) E_{50} is the strain at 50% of the soil strength (equivalent to K_{rm} for rock).
(6) E_s is the Young's Modulus.
(7) k is soil modulus used with p-y curve model.

Construction of Drilled Piers

Allowable bearing capacity recommendations provided in this report are based on proper construction procedures, including maintaining a dry shaft excavation and proper cleaning of bearing surfaces prior to placing reinforcing steel and concrete for drilled pier foundations. The construction of drilled piers should be observed by experienced geotechnical personnel during construction to help assure compliance with design assumptions. Observations should include: 1) identification of the bearing stratum, 2) minimum penetration depth, 3) removal of all smear zones and cuttings, 4) correct handling of groundwater seepage, if encountered, 5) piers are within acceptable vertical tolerance, and 6) related items.

We recommend that the pier-drilling equipment be equipped with suitable rock drilling teeth and the rig should have sufficient torque and weight to drill through the rock strata. Excavations for the piers must be maintained in a dry condition.

Groundwater was not encountered during drilling, but it may be encountered during installation of the piers, particularly if construction proceeds during a wet period of the year. Generally, rapid placement of steel and concrete may permit pier installation to proceed; however, at some locations, the seepage rates could require the use of temporary steel casing for proper installation. Water and loose materials in the cased pier excavations should be removed prior to the concrete placement.

A completed shaft excavation should not be allowed to remain open for more than 6 hours. Concrete placed in an excavation in excess of 10 feet should be placed in such a manner (using a tremie, centralizing chute, or by similar means) to prevent segregation of aggregates or to prevent concrete from striking the reinforcing steel.

After the satisfactory installation of the temporary casing, the required penetration into the bearing material may be excavated through the casing. Reinforcing steel and concrete should then be placed immediately after the excavation has been completed, dewatered, cleaned and observed. Dewatering could consist of using a bailing bucket, pumping, mixing the water with dry soil, etc. In the event groundwater levels observed during construction are above the base of the casing, extreme care should be exercised at all times to insure that the head of the plastic concrete is higher than the groundwater level outside of the casing. In actual practice, it is desirable that the head of the concrete in the casing be well above the static groundwater level prior to breaking the seal between the casing and the bearing stratum. If the outside groundwater level is higher than the plastic concrete level, water and some of the surrounding soils can contaminate the plastic concrete or cause necking (a reduction of the shaft diameter), reducing its strength. Once the seal is broken, the casing may be slowly removed in a vertical direction (no rotation permitted) while additional concrete is introduced into the top of the casing and placed through a tremie in order to provide for a continuous placement of the drilled pier concrete.

During construction of the drilled shafts, care should be taken to avoid creating an oversized cap ("mushroom"), particularly near the ground surface. A "mushroom" at the top of the drilled shaft could be lifted by heave of the expansive soils. Pier caps extending outside the nominal pier diameter (if used) should be constructed over void forms to minimize the potential for additional uplift forces.

Grade Beams and Pier Caps

Grade beams and pier caps (if required) should be physically isolated from the underlying soil surface by a void space and be structurally supported by the drilled pier foundations. A minimum void space of 6 inches should be provided beneath all grade beams. The purpose of the void is to provide space for swelling of expansive subsurface materials without resulting in structural distress to the grade beam. Structural cardboard carton forms are often used to provide this void beneath grade beams. Soil retainers (void form skirt) are further recommended to minimize the potential for infilling of the void space over time after carton forms deteriorate.

Cardboard void forms must have sufficient strength to support the weight of the grade beam during construction. Our experience indicates that major distress in grade beams will occur if the integrity of the void box is not maintained during construction. The excavation in which the void box lays must remain dry. Care must be exercised during construction to prevent collapse of these cartons. Backfill material must not be allowed to enter the void carton area below the grade beams, since this reduces the void space in which the underlying soils need to swell.

The exterior grade beams or foundation walls should be backfilled with a well-compacted, on-site clay or clay cover with a minimum thickness of at least 2 feet to retard migration of surface water into any drainage layer or into the void space. The backfill soils should be placed in maximum 8-inch loose lifts at a moisture content of +2 percentage points greater than their optimum moisture content (or wetter), determined in accordance with the standard Proctor procedure, ASTM D 698. The backfill should be compacted between 95 and 100 percent of its maximum dry density as determined by ASTM D 698 (standard Proctor). Interior backfill of grade beams should be compacted to at least 90 percent of its maximum dry density at a moisture content of +2 percentage points above optimum, or wetter.

Floor Slab

Two methods can be used to construct the floor system, the structural floor or the slab-on-grade floor system. In general, a structural (suspended) floor system is regarded as the most positive approach to limit the potential for post-construction movements of the slab due to soil swell to less than 1 inch. It should be noted that floor slab movements of 1 inch can result in some distress to interior finishes. This distress could include cracks in sheetrock and floor tiles, racking of doors and windows, and differential movement between the floor and exterior walls.

Structural Floor System. Two methods are available for constructing a suspended floor slab system. These include using 1) pan and joist type construction and raising the floor slab well above the underlying expansive soils, or 2) using cardboard carton forms to create a void.

1) The most effective suspended slab system is to use pan-joist type construction utilizing either concrete or steel beams. If this system is used, we recommend that the floor slab be suspended at least 6 inches, and preferably more, above final subgrade elevations. If utility lines are suspended beneath the slab, the void space clearance should be increased to a minimum of 2 feet to provide for access to these lines. Future movements of soil supported utility lines must be considered when designing connections, especially where these lines approach or enter the stationary structure. Provisions should be made for positive drainage of the under-floor space. Construction with metal beams and joists must also contain sufficient ventilation to limit corrosion of the metal components and deterioration of the finished floor and/or musty odors within the building. Furthermore, ponded water can cause mold to grow within the dry walls of the building. Precast concrete segments spanning between grade beams may also be considered.

2) Cardboard carton forms may also be used to create the void beneath the slab. These void forms should be at least 6 inches thick. If these forms are used, care must be taken to preserve their structural integrity and ability to create a consistent void. A rigid material layer (such as masonite or plywood) should be placed directly on the forms to prevent puncture by personnel during placement of concrete. This rigid layer would also help reduce the potential for concrete to leak down between the cardboard forms. A qualified inspector should be present during floor-slab concrete placement to help confirm that the void is maintained.

Soil-Supported Floor Slab on Improved Subgrade. If some post-construction movement of the floor slab can be tolerated, consideration may be given to soil-supported floor slab on an improved subgrade. Soil-supported floor slab may be constructed, provided the following subgrade preparations are implemented and maintained during construction. It should be understood by all parties that a soil-supported foundation floor system will likely experience some movement with time. Care exercised during construction will tend to minimize this risk. We recommend that the soil-supported floor slab be designed to minimize differential movement between floor slabs and pier-supported foundations. The soils should not be allowed to dry after required site preparation and moisture conditioning has been accomplished.

To limit post-construction movement to 1 inch, we recommend replacing the top 5 feet of soil with 5 feet of select fill material. Flexible base material (TxDOT, Item 247, Grade 1 or 2) or crushed concrete may be used instead of select fill, if desired. If rock is encountered during the excavation of the 5 feet of soil, the excavation process may be terminated at top of rock. For such a case, a geotechnical engineer, or his representative should observe the top of the rock surface prior to backfilling with select fill or flexible base.

Select fill material should not extend beyond the inside of the perimeter grade beams. We also recommend that the select fill be topped with 6 inches of flexible base material.

The material used as select fill should be clayey sand or sandy clay that are free of organics and other deleterious materials including clay balls or rock fragments larger than 4 inches. It should have a Plasticity Index (PI) between 5 and 15, a liquid limit of 40 or less, and between 25 and 55 percent passing the No. 200 sieve. Select fill material should be spread in six to eight-inch loose horizontal lifts, and uniformly compacted to a minimum of 95 percent of the material's maximum (standard Proctor) dry density (ASTM D 698) and at minimum moisture content of +3 percent above its optimum moisture content.

Select fill should not extend beyond the outside edge of the floor slab. Clayey soils should be used to backfill around and beyond the building's perimeter, thus limiting the potential for infiltration of surface water. The select fill should be kept in a moist condition until the floor slab is constructed. This could be achieved by regularly sprinkling the select fill with water during dry and windy days. A vapor barrier of polyethylene sheeting or similar material should be placed between the building slab and the subgrade soils to retard moisture or vapor migration through the slab.

The flexible base layer may consist of crushed limestone or crushed concrete meeting the requirements of Texas DOT Item 247, Grade 1 or 2. The material should be compacted to a minimum of 100 percent of standard Proctor (ASTM D 698) density, at or slightly above optimum moisture.

Basement Floor Slab

We understand that one level below grade a partial basement is planned for the civil courts building. The basement floor will be about 15 feet below the existing grade. Based on the subsurface conditions encountered in the borings, limestone (weathered or unweathered) will be exposed at the bottom of the basement excavation. Therefore, a slab-on-grade floor system is recommended for the floor slab construction.

The bottom of the excavation should be scarified to a depth of 6 inches and compacted with non-expansive select fill or flexible base materials. The placement and compaction of the select fill or flexible base materials should be performed as outlined in the section above entitled **"Soil-Supported Floor Slab on Improved Subgrade."** If rock is encountered at the bottom of the excavation, the floor slab should be placed on a minimum of 6 inches of compacted select fill or flexible base.

***Floor Slab Underdrains*.** Groundwater was not observed in the borings during our field investigation. However, we anticipate that granular materials of unknown lateral and vertical extent and orientation may be present in the final floor of the excavation. As such, we recommend that floor slab underdrains be provided beneath all below-grade floor slabs in areas where the potential for at least some seepage inflow is not acceptable, since groundwater levels can fluctuate both long-term and seasonally. These drains would be in addition to the perimeter drains recommended behind the below-grade walls, as described below in the section entitled **"Wall Drainage."**

The underdrain system should consist of a layer of free-draining sand or gravel (such as ASTM C 33 clean concrete sand), at least 4 to 6 inches in thickness, with a grid of perforated pipe underdrain trenches spaced at up to 15- to 20-foot centers. Recommended details for the pipe underdrain trenches are presented on Plate D. The pipe underdrains should be sloped to flow by gravity to the perimeter foundation drains and then to a sump pit. We strongly recommend that a geotextile filter fabric be placed between any below-grade subgrade soils and drainage layer materials to minimize migration of fine soil particles into the drainage materials, which could clog the system.

Construction Dewatering

We understand that a partial basement is planned for the civil courts building. Even though groundwater was not encountered during drilling, the use of a temporary dewatering system or implementation of dewatering procedures may be required during excavation of the basement. Also, for the long term, a permanent dewatering system will be required for the basement.

Below-grade Walls

Below-grade walls will need to be constructed for basement and any underground portions of the structure. We assume that below-grade walls for the structure will be supported on drilled straight-shaft pier foundations and grade beams as previously recommended in this report, and will require waterproofing to prevent groundwater migration through the walls.

Lateral Earth Pressures. The magnitude of lateral earth pressure against the proposed walls is dependent on the method of backfill placement, the type of backfill soils, drainage provisions, and whether the wall is permitted to yield after placement of the backfill. Typically, retaining walls are designed to allow some translational and rotational movements to occur, sufficient to allow the active lateral earth pressures to develop. Lateral movements on the order of 0.01 to 0.002 times the height of the wall are required to mobilize the active pressures of the backfill soils. If the retaining walls are restrained (such as for basement walls), the wall will be subjected to the at-rest lateral earth pressures, which are greater than the active lateral pressures. Recommended design lateral equivalent fluid pressures for the below grade walls are presented in the following table.

Equivalent Fluid Earth Pressures for Walls

Backfill Type	Equivalent Fluid Pressure Without Hydrostatic Pressures (Drained Condition) (psf/ft)		Equivalent Fluid Pressure With Hydrostatic Pressures (Undrained Condition) (psf/ft)	
	Active	At-Rest	Active	At-Rest
Free draining granular materials	40	55	80	90
Select fill	50	68	85	100
On-site soils	70	90	100	110

Note: The above values assume level back slope conditions behind the wall and wall drainage.

A surcharge load, q (in pounds per square foot), will typically result in a lateral pressure equal to about 0.4q (uniformly distributed with depth). This should be added to the above equivalent fluid pressure to obtain the total lateral earth pressure for design purposes. If the surcharge loading is located a distance away from the back of the wall greater than the wall height, the lateral pressure to the wall from the source will be minimal.

Backfill Material. Three types of backfill material that could be considered for proposed walls include:

> <u>*Free Draining Gravel or Sand*</u> – Granular, free draining materials that include sand, crushed stone, sand / gravel mixtures, or sand / crushed stone mixtures. The material should have less than 5 percent material passing the No. 200 sieve and less than 40 percent passing the No. 40 sieve. The minus 40 sieve material should be non-plastic. ASTM C33, size 57 or 67 coarse concrete aggregate will meet these requirements.

Select Fill – Select fill material used as backfill should be a clayey sand with a liquid limit (LL) of less than 40 and between 20 to 45 percent passing a No. 200 sieve. The plasticity index (PI) of this material should be 15 or less.

On-Site Soils – The on-site soils include lean/fat clays, free of organic matter and other deleterious material, and rock fragments less than 4 inches.

The wall backfill limits should extend outward at least 3 feet from the base of the wall and then upward on a 1H:2V slope. An impervious cover of well-compacted clay should be placed over the free-draining and select backfill types.

Wall backfill materials should be placed in loose lifts less than 9 inches thick and uniformly compacted to a minimum of 95 percent of the material's maximum dry density as determined by ASTM D 698 for cohesive soils. The moisture contents for cohesive backfill soils, if used, should be in the range of 0 to +5 percent above the optimum moisture content as determined in test method ASTM D 698. The moisture content for granular materials or select fill should be in the range of -1 to +3 percent above the optimum moisture content. Care should be exercised to avoid overstressing the wall by operating heavy compaction equipment too close to the back of the wall. In general, only light compaction equipment, less than 2,000 pounds, should be allowed to operate within 5 feet of the back of the wall.

***Wall Drainage*.** We recommend that positive drainage be provided for the backfill materials so that hydrostatic pressures are not allowed to develop behind the below-grade walls, or that the walls be designed to withstand the equivalent fluid pressures with hydrostatic surcharge. The upper surface of the wall backfill should be sloped to provide for positive drainage away from the structure and minimize the potential for infiltration of surface water into the backfill. If select fill is chosen as backfill, then a vertical wall drain consisting of a 6-inch thickness of free-draining coarse aggregate (protected by a suitable geotextile filter) or a composite geosynthetic drainage medium with similar transmissivity is recommended for walls with a height greater than 4 feet. The wall drain should be located immediately behind the wall stem and extend from the level of longitudinal drains, upward to not higher than 2 feet below the top of the wall. We recommend that the elevation of the perimeter foundation drains be established at a depth of at least 2 feet below the finish floor elevation. The wall drains should transport water to the perimeter foundation drains and then to a sump pump.

If free-draining aggregate backfill is used, a vertical wall drain would not be necessary. However, in this case, we recommend that a 2-foot thickness of well-compacted, impervious clay cover be placed over the backfill surface to minimize infiltration in areas that are not covered by pavement or flatwork. A geotextile filter fabric should be placed between the aggregate backfill and the clay cover materials to minimize infiltration of fines into the backfill, and between the aggregate backfill and the backslope of the native material.

Freestanding walls less than 5 feet tall can be drained using weep holes. Taller retaining walls should have a drainage system similar to what is recommended for the interior walls.

Backfill Settlements. Backfill and associated new fill placed behind the walls should be constructed in well-compacted lifts. Special care must be exercised to “tie in” the backfill with adjacent undisturbed, firm, natural soils by providing deep benches into the firm natural soil during placement of each fill lift. All loose materials and “slope wash” that may accumulate in the wall excavation during construction should be completely removed prior to placement of the backfill materials.

Some post-construction settlement of the backfill surface should be anticipated. This is typically on the order of 1 to 2 percent of the backfill height, even if satisfactory compaction of the backfill materials is achieved. Therefore, it is recommended that special consideration be given to the design of any foundation elements, floor slabs, and pavements that may extend over this backfill as a result of the potential for differential settlements introduced by this condition.

Excavations

An excavation is expected for the construction of the partial basement. When employed, temporary construction slopes should utilize excavation protection systems or be sloped back at an appropriate angle. For any unprotected excavations, the following slopes are required by OSHA standards:

Maximum Allowable Excavation Slopes

Soil Type	Allowable Slope	Maximum Depth
Type ‘A’	3/4 H:1V	20 feet
Type ‘B’	1 H:1V	20 feet
Type ‘C’	1-1/2 H:1V	20 feet

Soil type needs to be identified by the contractors “competent person” as defined by OSHA at the time of excavation. Excavations deeper than 20 feet will need to be engineered on a case-by-case basis according to OSHA standards.

Flat Work Considerations

Differential upward movement of all ground-supported slabs should be anticipated and considered during the design of the grading plan. We recommend that all access and entryway slabs and areas of flatwork be constructed on a subgrade prepared in accordance with the recommendations for the building pads of this report. Sidewalks should not be structurally tied to the building. To prevent potential tripping hazards, the slabs should be elevated noticeably above the adjacent, relatively non-modified, ground-supported sidewalks and pavement slabs.

Site Preparation and Fill Construction

Prior to placing any new fill, all existing surface vegetation, loose fill, debris, and similar unsuitable materials should be removed from within the limits of the fill areas. After stripping and any cutting operations, the exposed subgrade should be proofrolled with a loaded, tandem-axle dump truck weighing a minimum of 25 tons or other heavy, rubber-tired construction vehicle to locate any zones that are soft or unstable. The proofrolling should consist of several overlapping passes in mutually perpendicular directions over a given area. The subgrade in areas where rutting or pumping occurs during proofrolling should be removed and replaced with suitable fill, as described below, if it cannot be compacted in place. Proofrolling is not required where structurally suspended floor slabs will be used.

The site may then be filled to grade using a suitable fill according to the same criteria, free from deleterious matter and rock fragments larger than 4 inches. Fill materials placed outside building area should be placed in about 6 to 8-inch loose lifts with each lift compacted between 95 and 100 percent of the maximum dry density (ASTM D 698), at moisture contents between optimum and +5 percentage points above optimum.

Field density tests should be taken at the rate of one test per each 5,000 square feet, per lift, for all compacted fills. For areas where hand tamping is required, the testing frequency should be increased to approximately one test per lift, per 100 linear feet of area.

PAVEMENTS

Pavement Subgrade Preparation

If some cut and fill is required to achieve the final pavement subgrade elevations, then the work should be performed in accordance with recommendations outlined in the above report section entitled "**Site Preparation and Fill Construction**." It is recommended the exposed subgrade soils at this site be proofrolled with observations by qualified geotechnical personnel. The proofrolling should be performed with a loaded tandem-axle dump truck, scraper, or other heavy, rubber-tired vehicle weighing at least 25 tons, to locate any zones that are soft or unstable. The proofrolling should consist of several overlapping passes in mutually perpendicular directions over a given area. The subgrade in areas where rutting or pumping occurs during this operation should be removed and replaced with on-site soils.

The subgrade soils beneath the pavement sections should be lime stabilized to a depth of 6 inches and compacted to a minimum of 95 percent of the material's maximum standard Proctor dry density (ASTM D 698) at a moisture content between optimum and +5 percentage points above optimum. Seven percent-hydrated lime by dry soil weight (32 pounds per square yard) is estimated for a 6-inch thick stabilized subgrade based on our experience. The actual percentage of lime should be verified during construction. In general, the pH of the lime/soil mixture should be about 12.4 or greater. Lime treatment may be omitted if the concrete thickness is increased by one inch and the top 6 inches of the subgrade soil is compacted between 95 and 100 percent of its maximum dry density (standard Proctor) and a moisture content between optimum and 5 percentage points above optimum.

Concrete Pavements

Specific axle loading and traffic volume characteristics have not been provided at this time, but we assume that automobile traffic will be predominant in the parking areas, and some relatively heavy truck traffic could occur in drive areas around and behind the structure and in fire lanes. We have assumed that Portland Cement concrete pavement sections will be used for light and heavy traffic areas.

For heavy traffic areas, we have assumed 5 equivalent 18-kip axle loadings per day for a design period of 20 years. For light traffic areas, we have assumed 2 equivalent 18-kip axle loadings per day for a design period of 20 years.

The following Portland Cement concrete pavement sections are recommended for consideration at this site:

Traffic / Layer Material	Thickness, (in.) (inches)
Light Traffic (Parking Areas):	
Portland Cement Concrete	5
Lime Treated Subgrade	6
Heavy Traffic (Fire Lanes):	
Portland Cement Concrete	6
Lime Treated Subgrade	6

Note: Concrete thickness at dumpster area should be a minimum of 7 inches on a lime-treated subgrade. Lime treatment may be omitted if the concrete thickness is increased by one inch for each pavement section.

Some differential movements in the pavements are anticipated due to swelling of the subgrade clays. Pavement recommendations are based on assumed loading conditions and commonly accepted design procedures for an estimated design life for the assumed traffic loadings. Additional evaluation of the pavement design can be performed upon request based on more specific estimates of the design traffic and design period.

Design of the concrete pavements should specify a minimum 28-day concrete compressive strength of 3,600 psi. Hand-placed concrete should have a maximum slump of 6 inches. A sand-leveling course should not be placed beneath pavements. Use of superplasticizer should be considered to improve the concrete workability without increasing water cement ratio.

Past experience indicates that pavements with sealed contraction joints on 15 to 20-foot spacings, cut to a depth of at least one-quarter of the pavement thickness, have generally exhibited less uncontrolled, post-construction cracking than pavements with wider joint spacings. As a minimum, isolation joints should be used wherever the pavement will abut a structural element subject to different movement levels, e.g., light poles, retaining walls, existing pavement, stairways, entryway piers, building walls, or manholes. After construction, the construction, isolation and contraction joints should be inspected periodically and resealed, as necessary. The pavement should be nominally reinforced using at least No. 3 bars, 24 inches on center, each way.

It is our opinion that minimizing subgrade saturation is an important factor in maintaining subgrade strength. Water allowed to pond on or adjacent to the pavement could saturate the pavement and cause premature pavement deterioration. We recommend sloping all pavement surfaces to provide rapid surface drainage. Typically, two percent slopes are used to facilitate rapid surface drainage.

SITE GRADING, DRAINAGE, AND LANDSCAPING

In our experience with local expansive clays and residual soils similar to those found at this site, permanent cut and fill slopes should be gentle and preferably should not exceed about 4 horizontal to 1 vertical (4H:1V). It is anticipated that excavation of overburden soils can be accomplished with ordinary earthwork equipment and operations.

Excess water ponding on and beside roadways and sidewalks can cause unacceptable heave of these slabs. To reduce this potential heave, good surface drainage should be established in all building and pavement areas. Sprinkler systems, if provided, should be designed and operated to minimize saturation of soil adjacent to these structures. Sprinkler mains should not be placed next to the building.

Trees will remove water from the soil and, as a result, can cause the soil to shrink; therefore, trees should either a) not be planted closer than the mature tree height from the building, b) have a controlled irrigation system, or c) be planted in containers. Where it is desired to leave existing trees closer than the mature height of the tree to either the building or paved areas a root barrier should be provided to redirect the tree’s growth from impacting these structures. Additionally, where trees have been removed it is also important to excavate the entire root ball and replace with well compacted on-site clay in lifts.

Bedding soils for plants may collect and direct water underneath the building and pavements; therefore, care should be taken to insure that water entering the bedding soils drains away from these structures. If positive drainage away from these structures cannot be achieved, an impermeable synthetic membrane should be considered to reduce the risk of water migrating beneath the building and pavements. An 18-inch deep vertical water barrier along the pavement edge fronting landscaped areas may be desirable to help prevent irrigation water from having ready access to the soils beneath the pavement. Special attention should be given to provide good drainage from plantings inside building courtyards and planter boxes.

The completed landscaping should be carefully inspected to verify that plantings properly drain. Soil in plantings may settle, which will tend to pond water, or plantings may block entrances to surface drains. Therefore, maintaining positive drainage from landscape irrigation will be an ongoing concern.

LIMITATIONS

Since some variation was found in subsurface conditions at the specific boring locations for this study, all readers should be aware that a greater variation could occur between the boring locations. Statements in the report as to subsurface variations across the site are intended only as estimations from the data obtained at specific boring locations.

Additionally, Fugro's scope of work does not include the investigation, detection, or recommendations related to the presence of any biological pollutants. The term "biological pollutants" includes, but is not limited to, mold, fungi, spores, bacteria, and viruses, and the by-products of any such biological organisms.

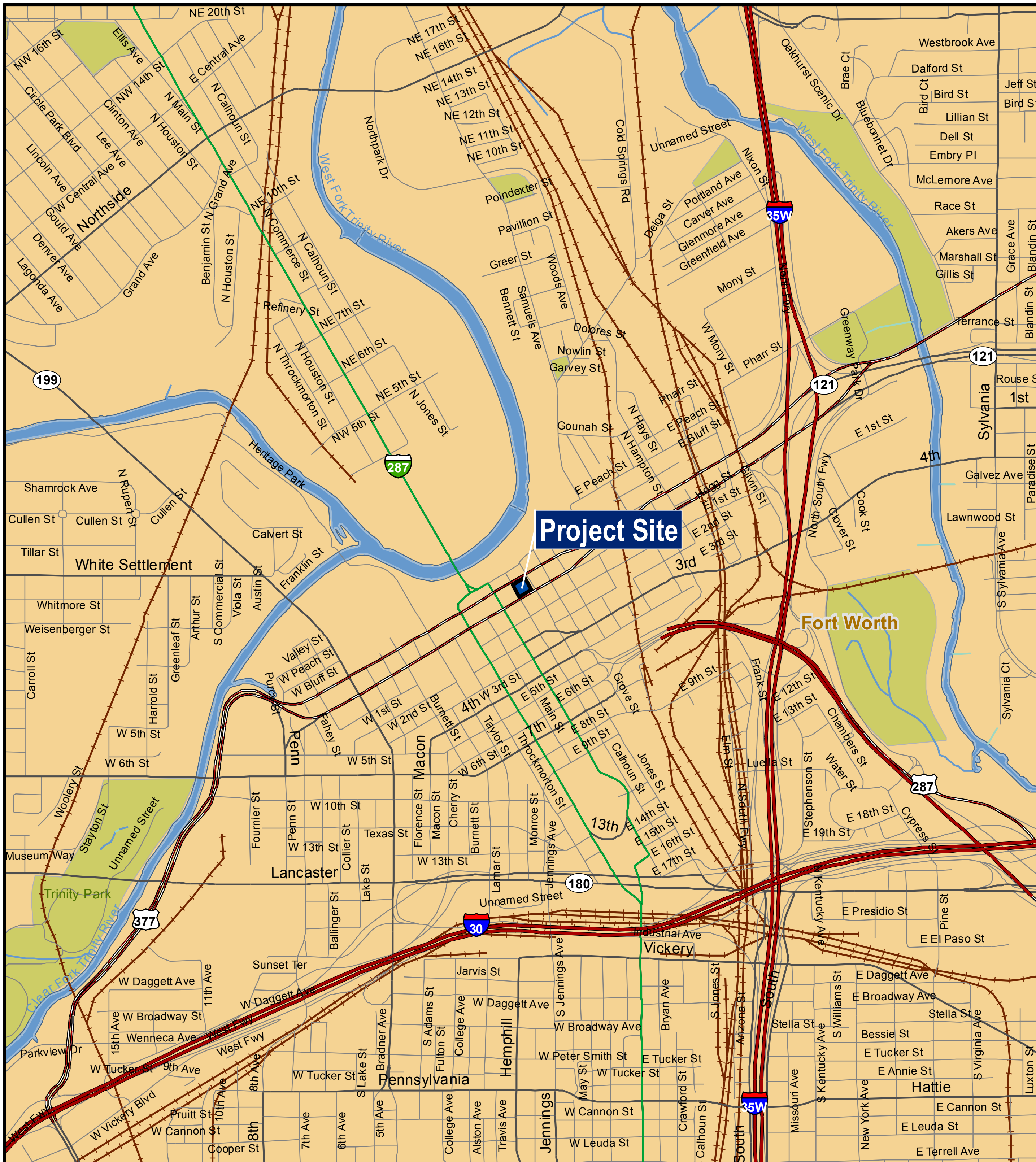
In preparation of this report, we have strived to perform our services in a manner consistent with that level of care and skill ordinarily exercised by other members of our profession currently practicing in the same locality under similar conditions. No other representation, expressed or implied, and no warranty or guarantee is included or intended in this report, any addendum report, opinion, document, or other instrument of service.

The results, conclusions, and recommendations contained in this report are directed at, and intended to be utilized within, the scope of work contained in the agreement executed by Fugro and client. This report is not intended for any other purposes. Fugro makes no claim or representation concerning any activity or condition falling outside the specified purposes to which this report is directed, said purposes being specifically limited to the scope of work as defined in our agreement. Inquiries as to our scope of work or concerning any activity or condition not specifically contained therein should be directed to Fugro for evaluation and, if necessary, further investigation.

ILLUSTRATIONS

Boring and laboratory data presented were developed solely for the preparation of this report. We are not responsible for interpretation or use of these data for purposes beyond the stated scope of this report.

Subsurface conditions different than those found at our boring location may be present as a result of, among other factors, soil moisture variations, fill placement, and naturally occurring variations in soil properties and elevation of the top of the rock.



Scale: 1 inch = 2,000 feet

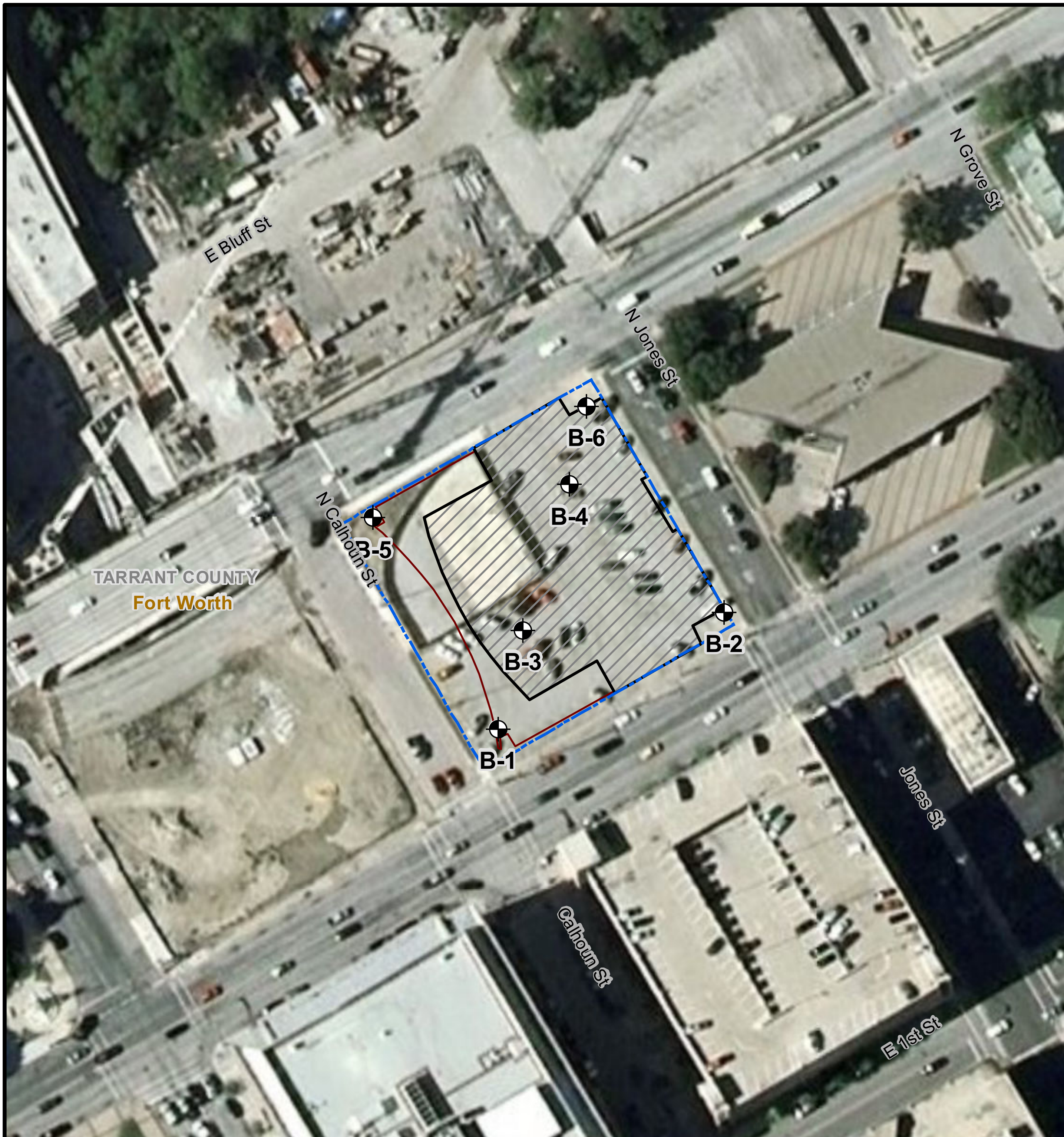
0 1,000 2,000 4,000 Feet

N

Coordinate System: State Plane Texas North Central FIPS 4202 Ft
Datum: D North American 1983

FUGRO
Fugro Consultants, Inc.

Tarrant County Civil Courts				**SITE VICINITY MAP**
Belknap Street				**Fort Worth, Texas**
Source: Street map: TNRIS Stratmap, 2006	Drawn By: SE	Date: November 3, 2010	Project No.: 04-4010-1069	**PLATE A**



Legend

- Approximate Boring Locations
- Project Site
- Foundation
- Building Above

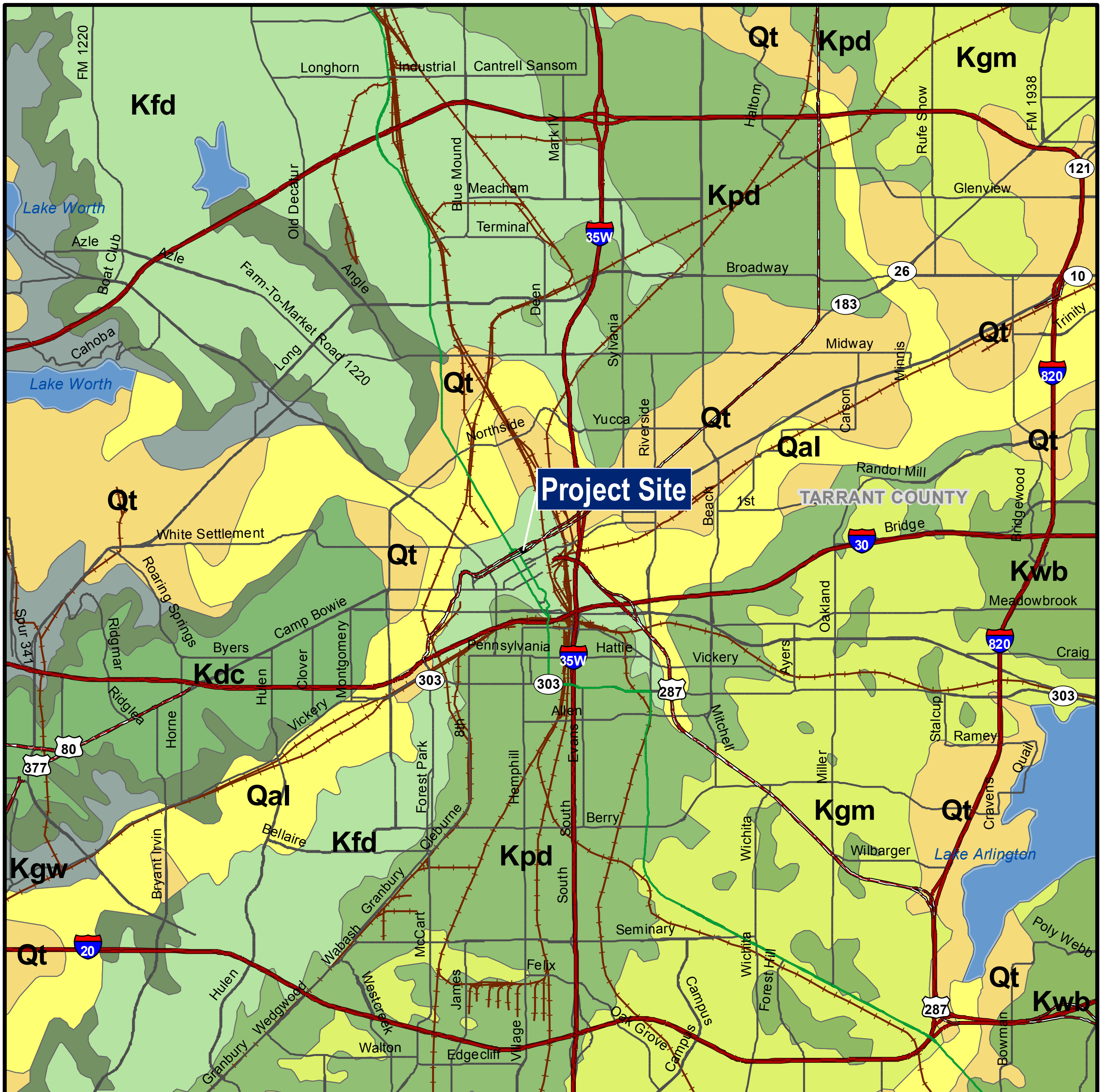
Scale: 1 inch = 100 feet

0 50 100 200 Feet

Coordinate System: State Plane Texas North Central FIPS 4202 Ft
Datum: D North American 1983

N

FUGRO Fugro Consultants, Inc.	**Tarrant County Civil Courts**			**SITE AND BORING PLAN**
	Belknap Street			**Fort Worth, Texas**
	Source: Orthophotography: i-cubed via ArcGIS Online, 2007	Drawn By: SE	Date: November 4, 2010	Project No.: 04-4010-1069 — **PLATE B**



Legend

Kdc	*Duck Creek Limestone*
Kfd	*Fort Worth Limestone and Duck Creek Formation, undivided*
Kgm	*Grayson Marl and Main Street Limestone, undivided*
Kgw	*Goodland Limestone and Walnut Clay, undivided*
Kki	*Kiamichi Formation*
Kpd	*Pawpaw Formation, Weno Limestone, and Denton Clay, undivided*
Kwb	*Woodbine Formation*
Kwu	*undivided part of Washita Group*
Qal	*Alluvium*
Qt	*Terrace deposits*
	Water

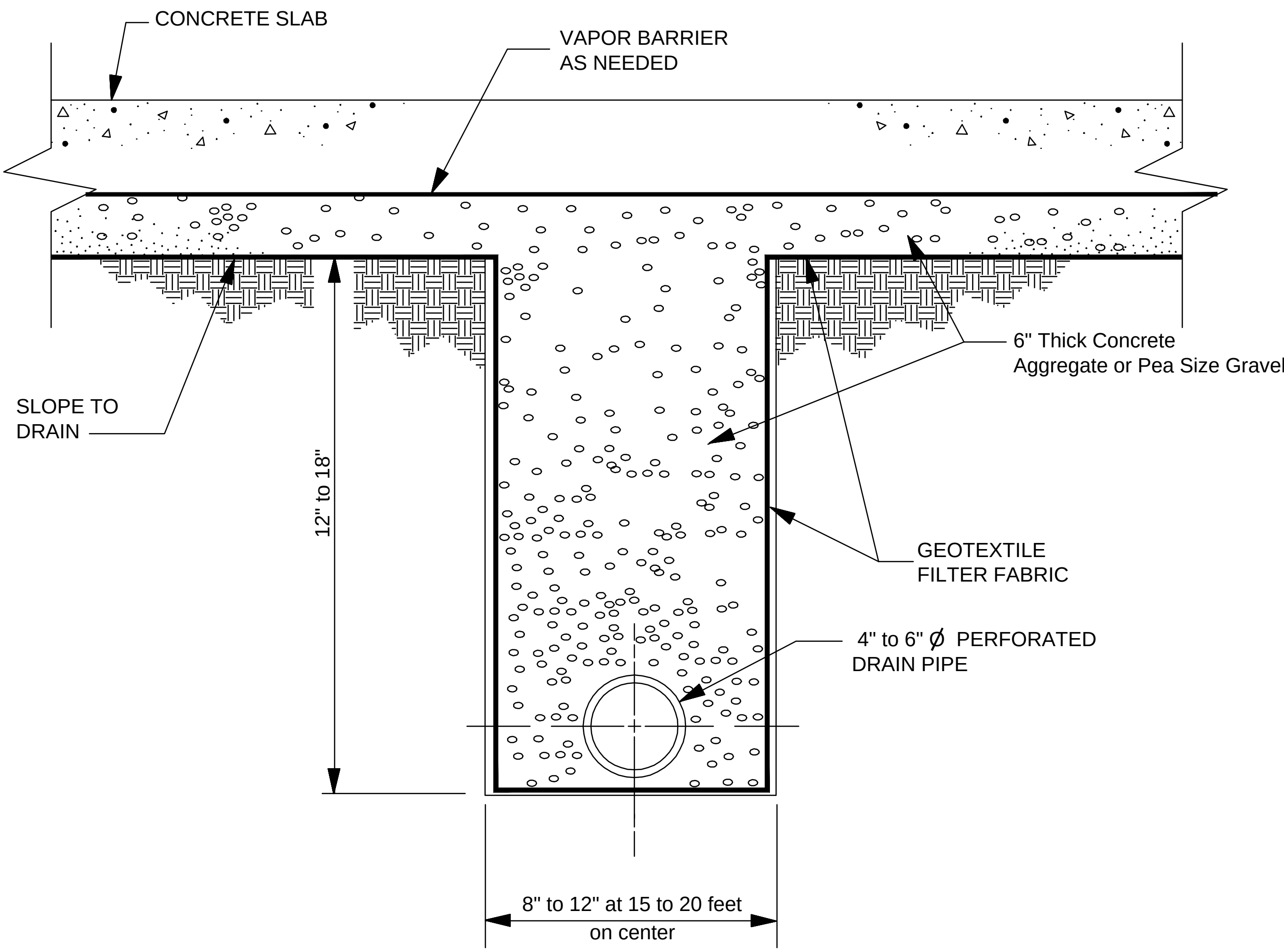
Scale: 1 inch = 10,000 feet

0 5,000 10,000 20,000 Feet

N

Coordinate System: State Plane Texas North Central FIPS 4202 Ft
Datum: D North American 1983

FUGRO Fugro Consultants, Inc.	**Tarrant County Civil Courts**			**GEOLOGIC MAP**
	Belknap Street			**Fort Worth, Texas**
	Source: **Geology: USGS, 2002**	Drawn By: **SE**	Date: **November 4, 2010**	Project No.: **04-4010-1069** — **PLATE C**



Not to Scale

Fugro Consultants, Inc.	**Tarrant County Civil Courts**		**FLOOR SLAB UNDERDRAIN DETAIL**
	Belknap Street		**Fort Worth, Texas**
	Date: **November 3, 2010**	Drawn By: **DF**	Project No.: **04-4010-1069**
			PLATE D

LOG OF BORING NO. B- 1

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				3" of Asphalt over 6" of Base								
			P = 4.25	**FAT CLAY (CH)**, dark brown	0.9	23						
				- with calcareous nodules below 2'		23	58	22	36			
			P = 3.5									
			N = 73	**LIMESTONE**, light gray, with clay seams and sand	4.0							
5												
			N = 50/5"	**LEAN CLAY (CL)**, brownish yellow, with limestone fragments	7.0	10	29	14	15			
			N = 50/3"									
10			N = 50/4"	**WEATHERED LIMESTONE**, very pale brown, with clay layers	10.0							
			100/0.75"									
15												
				LIMESTONE, medium gray, with shale layers	17.0							
			100/1.5"									
20												
			100/3.25"									

Fugro Consultants, Inc.

COMPLETION DEPTH: 58.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 1a

FUGRO STD 04-4010-1069.GPJ FUGRO DATA TEMPLATE 042610.GDT 11/9/10

LOG OF BORING NO. B- 1

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION SURF. ELEVATION: Unknown	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				LIMESTONE, medium gray, with shale layers *(continued)*								
			100/1.5"									
30												
			100/1"									
35												
				- hard limestone layers @ 36'								
			100/0.75"									
40												
			100/0.75"									
45												
			100/0.75"									

Fugro Consultants, Inc.

COMPLETION DEPTH: 58.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 1b

LOG OF BORING NO. B- 1

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				LIMESTONE, medium gray, with shale layers *(continued)*								
			100/0.75"									
55												
			100/0.5"		58.0							
60												
65												
70												

Fugro Consultants, Inc.

COMPLETION DEPTH: 58.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 1c

LOG OF BORING NO. B- 2

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	POCKET PEN, tsf / Blows/ft. / REC./RQD, %	STRATUM DESCRIPTION (SURF. ELEVATION: Unknown)	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
		2" Asphalt over 10" Base								
	P = 4.5	**FILL**, **FAT CLAY (CH)**, reddish brown	1.0	20	39	18	21			
	N = 50/5"									
		WEATHERED LIMESTONE, very pale brown, weathered	2.5							
	100/0.5"									
5	5'-10' 80/ 72									
10	10'-15' 80/ 80									
		LIMESTONE, gray	12.0							
				7					145	145.8
15	15'-20' 100/ 100	- shale layer from 15.5' to 16.4'								
				6					148	401.2
		- shale layer from 19.4' to 20.2'								
20	20'-25' 100/ 100			8					136	22.0
				9					133	11.5

Fugro Consultants, Inc.

COMPLETION DEPTH: 50.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 2a

FUGRO STD 04-4010-1069.GPJ FUGRO DATA TEMPLATE 042610.GDT 11/9/10

LOG OF BORING NO. B- 2

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
	25'-30' 100/ 100	**LIMESTONE**, gray *(continued)*		13					125	14.7
				10					132	19.5
30	30'-35' 100/ 100									
		- shale layer from 31.5' to 32'								
35	35'-40' 100/ 100									
				6					146	172.2
40	40'-45' 100/ 100									
45	45'-50' 100/ 100									

Fugro Consultants, Inc.

COMPLETION DEPTH: 50.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 2b

LOG OF BORING NO. B- 3

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	POCKET PEN, tsf / Blows/ft. / REC./RQD, %	STRATUM DESCRIPTION (SURF. ELEVATION: Unknown)	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
	P = 4.0	2" Asphalt over 6" Base (sand and gravel)								
	P = 4.0	**FILL**, **CLAYEY SAND (SC)**, dark brown, with gravel	0.8							
	P = 4.0	**FAT CLAY (CH)**, dark brown, few calcareous nodules	1.5	23						
				22	63	23	40			
	P = 3.5									
	N = 26			17	51	21	30			
5		**LEAN CLAY (CL)**, brownish yellow, with limestone fragments	4.5							
	100/4.5"	**WEATHERED LIMESTONE**, very pale brown, with clay layers	7.0							
10	10'-14' 88/ 54									
	14'-19'			16					116	4.8
15	80/ 37	**LIMESTONE**, medium gray, with shale layers	14.5							
		- 2" shale layer at 16.7'								
	19'-24' 100/ 100			8					137	31.4
20										
				14					124	9.9
	24'-29' 100/ 100	- shale layer from 24' to 24.9'								

Fugro Consultants, Inc.

COMPLETION DEPTH: 54.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 3a

LOG OF BORING NO. B- 3

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION (SURF. ELEVATION: Unknown)	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				LIMESTONE, medium gray, with shale layers *(continued)*		13					129	13.5
			29'-34' 100/ 100									
30						15					124	12.2
			34'-44' 100/ 80									
35												
				- 3" shale layer at 36.3'								
40				- shale layers from 40' to 41.7'		9					134	13.4
						5					154	334.4
			44'-49' 100/ 90									
45						5					147	46.2
			49'-54' 100/ 60									

Fugro Consultants, Inc.

COMPLETION DEPTH: 54.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 3b

LOG OF BORING NO. B- 3

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION (SURF. ELEVATION: Unknown)	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				LIMESTONE, medium gray, with shale layers *(continued)*	54.0							
55												
60												
65												
70												

Fugro Consultants, Inc.

COMPLETION DEPTH: 54.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 3c

LOG OF BORING NO. B- 4

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION SURF. ELEVATION: Unknown	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
				3" Asphalt over 6" Base								
			N = 6	**FILL**, **FAT CLAY (CH)**, dark brown, with gravel	0.9							
			P = 2.5	**FAT CLAY (CH)**, reddish brown	3.0	25	58	24	34			
			N = 50/4"	**WEATHERED LIMESTONE**, very pale brown, with clay seams	3.5	8						
5				Boring terminated at 5 feet due to soil contamination encountered.	5.0							
10												
15												
20												

Fugro Consultants, Inc.

COMPLETION DEPTH: 5.0
DATE DRILLED: 10-9-10 to 10-9-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 4

LOG OF BORING NO. B- 5

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
			P = 4.5	**FAT CLAY (CH)**, reddish brown, with calcareous nodules		23						
			P = 4.5			24	68	25	43			
			N = 50/4"	**WEATHERED LIMESTONE**, very pale brown, with clay seams	2.0							
			N = 50/0"	**WEATHERED LIMESTONE**, very pale brown - with clay layer from 4' to 5'	4.0	11	39	15	24			
5												
			100/0.75"									
10			100/1.5"									
				LIMESTONE, gray, with shale layers	12.0							
15			100/1.75"									
20			100/0.75"									

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COMPLETION DEPTH: 50.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 5a

LOG OF BORING NO. B- 5

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	SYMBOL	SAMPLES	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
			100/1.5"	**LIMESTONE**, gray, with shale layers *(continued)*								
30			100/0.5"									
35			100/0.5"									
			100/0.75"									
40												
45			100/0.75"									
			100/0.5"									

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COMPLETION DEPTH: 50.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 5b

LOG OF BORING NO. B- 6

TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS
PROJECT NO. 04-4010-1069

NORTHING: Unknown
EASTING: Unknown

SURF. ELEVATION: Unknown

DEPTH, FT	POCKET PEN, tsf Blows/ft. REC./RQD, %	STRATUM DESCRIPTION	LAYER ELEV./ DEPTH	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX (PI), %	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	UNCONFINED STRENGTH TSF
		3" of Asphalt, 15" Sand and Gravel Base, 6" possible black Tar								
	P = 2.0	**FILL**, **LEAN CLAY (CL)**, brownish yellow, with gravel	2.0							
	P = 4.5	**LEAN CLAY (CL)**, brownish yellow, with limestone fragments	3.0	21	42	18	24			
5	N = 37	**LIMESTONE**, very pale brown, with clay seams	4.0							
		Boring terminated at 7 feet due to soil contamination encountered.	7.0							
10										
15										
20										

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COMPLETION DEPTH: 7.0
DATE DRILLED: 10-10-10 to 10-10-10
WATER LEVEL / SEEPAGE: DRY
WATER LEVEL (UPON COMPLETION): DRY

KEY:
P = Pocket Penetrometer
Note: All depths are measured in feet.
Coordinate System: NAD 83

PLATE 6

TERMS AND SYMBOLS USED ON BORING LOGS FOR SOIL

Sampler Types

- Thin-walled Tube
- Standard Penetration Test (SPT)
- Texas Cone Penetration Test (TCP)
- Auger Sample
- Bag Sample

Material Types

- LEAN CLAY (CL)
- SANDY LEAN CLAY (CL)
- FAT CLAY (CH)
- SANDY FAT CLAY (CH)
- WELL-GRADED GRAVEL (GW)
- POORLY-GRADED GRAVEL (GP)
- SILTY GRAVEL (GM)
- CLAYEY GRAVEL (GC)
- WELL-GRADED SAND (SW)
- POORLY-GRADED SAND (SP)
- SILTY SAND (SM)
- CLAYEY SAND (SC)
- FILL (F)
- ASPHALT (A)
- CONCRETE (C)
- AGGREGATE BASE (AB)

Consistency

Strength of Fine Grained Soils

Consistency	SPT (# blows/ft)[1]	UCS (TSF)[1]	PP (Fugro DFW)
Very Soft	< 2	< 0.25	0.4
Soft	2 - 4	0.25 - 0.5	0.5 - 0.8
Medium Stiff	4 - 8	0.5 - 1.0	0.9 - 1.6
Stiff	8 - 15	1.0 - 2.0	1.7 - 3.3
Very Stiff	15 - 30	2.0 - 4.0	> 3.4
Hard	> 30	> 4.0	

Density of Coarse Grained Soils

Apparent Density	SPT (# blows/ft)	TCP (# blows/ft)[2]
Very Loose	0 - 4	< 8
Loose	4 - 10	8 - 20
Medium Dense	10 - 30	20 - 60
Dense	30 - 50	60 - 100
Very Dense	> 50	> 100

Moisture

Moisture Content adapted from (3)

Dry	No water evident in sample
Moist	Sample feels damp
Very Moist	Water visible on sample
Wet	Sample bears free water

Grain Size[3]

U.S. Standard Sieve

	12"	3"	3/4"	4	10	40	200	
Boulders	Cobbles	Gravel – Coarse	Gravel – Fine	Sand – Coarse	Sand – Medium	Sand – Fine	Silt	Clay
	300	75	19	4.75	2.00	0.425	0.075	0.002

Particle Grain Size in Milimeters

Structure[3]

Criteria for Describing Structure

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least 6 mm thick; note thickness
Laminated	Alternating layers of varying material or color with the layers less than 6 mm thick; note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

Secondary Components

Criteria for Describing Structure adapted from (3)

Trace	< 5% of sample
Few	5% to 10% of sample
Little	10% to 25% of sample
Some	25% to 50% of sample

Size Modifiers for Inclusions

Pocket	Inclusion of different material that is smaller than the diameter of the sample
Fragment	Pieces of a whole item - often used with shell and wood
Nodule	A concretion, a small, more or less rounded body that is usually harder than the surrounding soil (as in carbonate nodule) and was formed in the soil by a weathering process
Streak	A line or mark of contrasting color or texture. The mark or line should be paper thin, and it should be natural - not a smear caused by extruding or trimming the sample

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Note: Information on each boring log is a compilation of subsurface conditions and soil and rock classificaitons obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the times and places indicated, and may vary with time, geologic condition or construction activity.

References: [1] Peck, Hanson and Thornburn, (1974), Foundation Engineering.
[2] TxDOT, (1999), Tex-142-E, Laboratory Classification of Soils for Engineering Purposes.
[3] ASTM International, ASTM D 2488 Standard Practice for Description and Identification of Soils.

TERMS AND SYMBOLS USED ON BORING LOGS FOR ROCK

Sampler Types

Rock Core

Texas Cone Penetration Test (TCP)

Bag Sample

Notation for Rock Core Samples	
RC_	Rock Core sample + depth interval
Rec	Rock Core Sample Recovery (ASTM D2113)
RQD	Rock Quality Designation (ASTM D6032)

Material Types

LIMESTONE (L)

SHALE (SH)

SANDSTONE (SS)

MARL (M)

WEATHERED LIMESTONE (W)

WEATHERED SHALE (WSH)

WEATHERED SANDSTONE (WSS)

WEATHERED MARL (WM)

Weathering[4]

Weathering Grades of Rock Mass	
Slightly	Discoloration indicates weathering of rock material and discontinuity surfaces
Moderately	Less than half of the rock material is decomposed or disintegrated to a soil
Highly	More than half of the rock material is decomposed or disintegrated to a soil
Completely	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed

Hardness

Criteria for Field Hardness	
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	Can be grooved or gouged ¼" 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
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

Grain Size[3]

U.S. Standard Sieve

3" 3/4" 4 10 40 200

Gravel		Sand		
Coarse	Fine	Coarse	Medium	Fine

75 19 4.75 2.00 0.425 0.075

Particle Grain Size in Milimeters

Secondary Components[3]

Criteria for Describing Structure	
Trace	< 5% of sample
Few	5% to 10% of sample
Little	10% to 25% of sample
Some	25% to 50% of sample

Structure

Bedding Thickness and Spacing of Planar Features			
Type	**Spacing**	**Thickness**	**Fracture Spacing**
Parting	< 1/8 in.	Laminar	NA
Seam	1/8 to 3/4 in.	Extremely thin	Extremely close (< 3/4 in.)
	3/4 to 2 1/2 in.	Very thin	Very close
Layer	2 1/2 to 6 in.	Thin	Close
	6 to 24 in.	Medium	Moderate
Bed	2 to 7 ft.	Thick	Wide
	7 ft. to 20 ft.	Very thick	Very wide
	> 20 ft.	Extremely thick	Extremely wide
	Massive	No stratification observed	NA
	Occasional	Occurring once or less per foot	
	Frequently	Occurring more than once per foot	

Discontinuities	
Joint	A natural fracture along which no displacement has occurred. May occur in parallel groups called sets.
Fracture/ Shear	A natural fracture along which differential movement has occurred. May be slickensided or striated.
Fault	A natural fracture along which displacement has occurred. Usually lined with gouge and slickensides.

Surface Planarity	
Curved	A moderately undulating surface, with no sharp breaks or steps.
Planar	A flat surface
Stepped	A surface with asperities or steps. The height of the asperity should be estimated or measured.

Roughness	
Very Rough	Near vertical steps and ridges occur on the discontinuity
Rough	Some ridges and side-angle steps are evident; asperities are clearly visible, surface feels very abrasive.
Slightly Rough	Asperities on the discontinuity surfaces can be seen and felt.
Smooth	Surface appears smooth and feels smooth.
Slickensided	Evidence of polishing and movement are visible.

Aperture	
Tight	Core pieces on either side of fracture can be fitted together so that no visible void spaces remain.
Open	Core pieces on either side of fracture cannot be fitted tightly together and voids are visible.
Healed	A completely healed fracture or vein is not considered a discontinuity and should not be included when describing rock core fracturing or calculating RQD. This feature should be described including a record of dip, spacing, thickness, type of filling and any observed alteration.

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References: [1] Peck, Hanson and Thornburn, (1974), Foundation Engineering.
[3] ASTM International, ASTM D 2488 Standard Practice for Description and Identification of Soils.
[4] British Standard (1981), Code of Practice for Site Investigation BS 5930.

GEOTECHNICAL INVESTIGATION
TARRANT COUNTY CIVIL COURTS
BELKNAP STREET AND JONES STREET
FORT WORTH, TEXAS

SUMMARY OF FREE SWELL TEST RESULTS

Boring Number	Sample Depth, feet	Liquid Limit	Plastic Limit	Plasticity Index	Initial Moisture Content, %	Final Moisture Content, %	Applied Surcharge Pressure, psf	Percent Vertical Swell, %
B-1	2-3	58	22	36	23	24	375	1.6
B-3	2-3	63	23	40	22	25	375	2.4
B-4	3-3.5	58	24	34	25	26	500	0.5
B-5	1-2	68	25	43	24	25	250	1.8