

**GEOTECHNICAL ENGINEERING STUDY  
COOK CHILDREN'S MEDICAL OFFICE BUILDING  
FM 731 AT SH 174  
BURLESON, TEXAS**

Presented To:

**Baird, Hampton & Brown, Inc.**

October 2008

**PROJECT NO. 110-08-28**

October 10, 2008  
Report No. 110-08-28

Baird, Hampton & Brown, Inc  
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Attn: Mr. Konstantine Bakintas, P.E.

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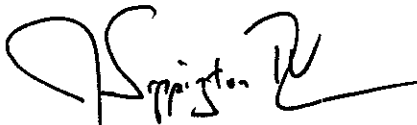
Dear Mr. Bakintas:

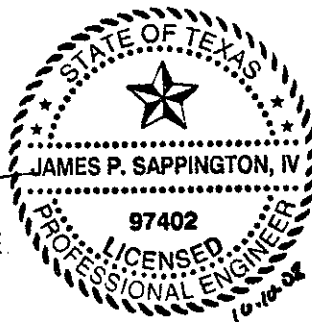
Submitted here are the results of a geotechnical engineering study for the referenced project. This study was performed in general accordance with our Proposal No. 08-2608 dated September 22, 2008. The geotechnical services were authorized on September 23, 2008 by Mr. Konstantine Bakintas, P.E., Principal of Baird, Hampton & Brown, Inc.

Engineering analyses and recommendations are contained in the text section of the report. Results of our field and laboratory services are included in the appendix of the report. We would appreciate the opportunity to be considered for providing the materials engineering and geotechnical observation services during the construction phase of this project.

We appreciate the opportunity to be of service to Baird, Hampton & Brown, Inc. Please contact us if you have any questions or if we may be of further service at this time.

Respectfully submitted,  
CMJ ENGINEERING, INC.

  
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## **1.0 INTRODUCTION**

### **1.1 Project Description**

The project site is located on the northeast side of John Jones Drive SE (FM 731), southeast of Wilshire Boulevard SW (SH 174) in Burleson, Texas. The project, as currently planned, will consist of a single story medical building with a footprint of approximately 5,000 square feet. Structural loads are anticipated to be relatively light and no basements are planned. Associated driveways and parking areas are also planned. Plate A 1, Plan of Borings, presents the approximate locations of the exploration borings.

### **1.2 Purpose and Scope**

The purpose of this geotechnical engineering study has been to determine the general subsurface conditions, evaluate the engineering characteristics of the subsurface materials encountered, and develop recommendations for the type or types of foundations suitable for the project.

To accomplish its intended purposes, the study has been conducted in the following phases: (1) drilling sample borings to determine the general subsurface conditions and to obtain samples for testing; (2) performing laboratory tests on appropriate samples to determine pertinent engineering properties of the subsurface materials; and (3) performing engineering analyses, using the field and laboratory data to develop geotechnical recommendations for the proposed construction.

The design is currently in progress and the locations and/or elevations of the structure could change. Once the final design is near completion (80-percent to 90-percent stage), it is recommended that CMJ Engineering, Inc. be retained to review those portions of the construction documents pertaining to the geotechnical recommendations, as a means to determine that our recommendations have been interpreted as intended.

### **1.3 Report Format**

The text of the report is contained in Sections 1 through 10. All plates and large tables are contained in Appendix A. The alpha-numeric plate and table numbers identify the appendix in which they appear. Small tables of less than one page in length may appear in the body of the text and are numbered according to the section in which they occur.

Units used in the report are based on the English system and may include tons per square foot (tsf), kips (1 kip = 1,000 pounds), kips per square foot (ksf), pounds per square foot (psf), pounds per cubic foot (pcf), and pounds per square inch (psi).

## **2.0 FIELD EXPLORATION AND LABORATORY TESTING**

### **2.1 Field Exploration**

Subsurface materials at the project site were explored by three (3) vertical soil borings. Borings B-1 and B-2 were drilled to depths of 20 feet in association with the proposed building. Boring B-3 is associated with the proposed pavement area and extends to a depth of 5 feet. The borings were drilled using continuous flight augers at the approximate locations shown on the Plan of Borings, Plate A.1. The boring logs are included on Plates A.4 through A.6 and keys to classifications and symbols used on the logs are provided on Plates A.2 and A.3.

Undisturbed samples of cohesive soils were obtained with nominal 3-inch diameter thin-walled (Shelby) tube samplers at the locations shown on the logs of borings. The Shelby tube sampler consists of a thin-walled steel tube with a sharp cutting edge connected to a head equipped with a ball valve threaded for rod connection. The tube is pushed into the soil by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency with a hand penetrometer, sealed, and packaged to limit loss of moisture.

The consistency of cohesive soil samples was evaluated in the field using a calibrated hand penetrometer. In this test a 0.25-inch diameter piston is pushed into the relatively undisturbed sample at a constant rate to a depth of 0.25 inch. The results of these tests, in tsf, are tabulated at respective sample depths on the logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

To evaluate the relative density and consistency of the harder formations, a modified version of the Texas Cone Penetration test was performed at selected locations. Texas Department of Transportation (TxDOT) Test Method Tex-132-E specifies driving a 3-inch diameter cone with a 170-pound hammer freely falling 24 inches. This results in 340 foot-pounds of energy for each blow. This method was modified by utilizing a 140-pound hammer freely falling 30 inches. This results in 350 foot-pounds of energy for each hammer blow. In relatively soft materials, the penetrometer cone is driven 1 foot and the number of blows required for each 6-inch penetration is

tabulated at respective test depths, as blows per 6 inches on the log. In hard materials (rock or rock-like), the penetrometer cone is driven with the resulting penetrations, in inches, recorded for the first and second 50 blows, a total of 100 blows. The penetration for the total 100 blows is recorded at the respective testing depths on the boring logs.

## **2.2 Laboratory Testing**

Laboratory soil tests were performed on selected representative samples recovered from the borings. In addition to the classification tests (liquid limits and plastic limits), moisture content, unit weight, and unconfined compressive strength tests were performed. Results of the laboratory tests conducted for this project are included on the boring logs.

A swell test was performed on a specimen from a selected sample of the clays. This test was performed to help in evaluating the swell potential of near-surface soils in the area of the proposed building. The results of the swell tests are presented on Plate A.7.

The above laboratory tests were performed in general accordance with applicable ASTM procedures, or generally accepted practice.

## **3.0 SUBSURFACE CONDITIONS**

### **3.1 Soil and Rock Conditions**

Specific types and depths of subsurface strata encountered at the boring locations are shown on the boring logs in Appendix A. The generalized subsurface stratigraphies encountered in the borings are discussed below. Note that depths on the borings refer to the depth from the existing grade or ground surface present at the time of the investigation, and the boundaries between the various soil types are approximate.

Natural overburden soils consist of dark brown and light brown clays and silty clays. Ironstone nodules and calcareous nodules are present in select reaches within the clays. Abundant gravel is present within the silty clays below depths of 7 to 9 feet. The surficial clays encountered in the borings had tested Liquid Limits (LL) ranging from 26 to 57 and Plasticity Indices (PI) ranging from 11 to 35 and are classified as CL and CH by the USCS. The various clayey soils were generally stiff to hard (soil basis) in consistency with pocket penetrometer readings of 2.0 to over 4.5 tsf.

Tested unit weight values within the overburden soils were 99 and 107 pcf and an unconfined compressive strength was 6,530 psf.

Tan limestone with clay seams is present beneath the overburden soils at depths of 9 to 11.5 feet. The tan limestone is weathered in Boring B-1, and is hard (rock basis) with a Texas Cone Penetration (THD) value of 1.25 inches per 100 blows.

Gray limestone is next present at a depth of 13 feet in Borings B-1 and B-2. The gray limestone contains shale seams and is very hard (rock basis), with THD values of 0.25 and 0.75 inch per 100 blows.

The Atterberg Limits tests indicate the clays encountered at this site are moderately active to highly active with respect to moisture induced volume changes. Active clays can experience volume changes (expansion or contraction) with fluctuations in their moisture content.

### **3.2 Ground-Water Observations**

The borings were drilled using continuous flight augers in order to observe ground-water seepage during drilling. Ground-water seepage was present at a depth of 9 feet in Borings B-1 and B-2 during drilling. Ground-water seepage was not encountered during drilling in Boring B-3 and all borings were dry at completion. While it is not possible to accurately predict the magnitude of subsurface water fluctuation that might occur based upon these short-term observations, it should be recognized that ground-water conditions will vary with fluctuations in rainfall.

Fluctuations of the ground-water level can occur due to seasonal variations in the amount of rainfall; site topography and runoff; hydraulic conductivity of soil strata; and other factors not evident at the time the borings were performed. The possibility of ground-water level fluctuations should be considered when developing the design and construction plans for the project. The possibility exists that perched water may occur atop the limestone through more granular seams, particularly after periods of heavy or extended rainfall.

## 4.0 FOUNDATION RECOMMENDATIONS

### 4.1 General Foundation Considerations

Two independent design criteria must be satisfied in the selection of the type of foundation to support the proposed structure. First, the ultimate bearing capacity, reduced by a sufficient factor of safety, must not be exceeded by the bearing pressure transferred to the foundation soils. Second, due to consolidation or expansion of the underlying soils during the operating life of the structures, total and differential vertical movements must be within tolerable limits.

The moisture induced volume changes associated with the moderately active to highly active clays present at this site indicate that shallow or near surface footings could be subject to differential movements of a potentially detrimental magnitude. The most positive foundation system for the proposed structure would be situated below the zone of seasonal moisture variations. A deep foundation system transferring column loads to a suitable bearing stratum is considered the most positive foundation system. Straight drilled reinforced concrete shafts penetrating the gray limestone with shale seams offer a positive foundation system and are recommended.

If differential movements can be tolerated, consideration can be given to the use of a monolithic slab-on-grade. The key to success of slab-on-grade construction is proper design/construction, and providing the most optimum conditions for reduced slab movements. Providing excellent drainage away from the structure, preventing ponding of water aside the slab, preventing excess drying of soils, and using onsite soil backfill to prevent water intrusion into utility line backfill will enhance slab performance. Recommendations these foundation systems are presented below.

### 4.2 Straight Shaft Design Parameters

#### 4.2.1 Design Criteria

Recommendations and parameters for the design of cast-in-place straight-shaft drilled piers are outlined below. Specific recommendations for the construction and installation of the drilled piers are included in the following section, and shall be followed during construction.

Bearing Stratum	Gray LIMESTONE with shale seams
Depth of Bearing Stratum:	Approximately 13 feet below <u>existing</u> grades
Required Penetration/Depth:	All piers should penetrate into the bearing stratum a minimum of 2 feet.

Allowable End Bearing Capacity: 30,000 psf

Allowable Skin Friction: Applicable below a minimum penetration of 2 feet into gray limestone; 5,500 psf for compressive loads and 4,000 psf for tensile loads.

The above values contain a safety factor of three (3). It should be anticipated that ground-water seepage will be encountered during installation of the straight shafts.

In order to develop full load carrying capacity in skin friction, adjacent shafts should have a minimum center-to-center spacing of 3 times the diameter of the larger shaft. Closer spacing may require some reductions in skin friction and/or changes in installation sequences. Closely spaced shafts should be examined on a case-by-case basis. As a general guide, the design skin friction will vary linearly from the full value at a spacing of 3 diameters to 50 percent of the design value at 1 diameter.

Settlements for properly installed and constructed straight shafts in the gray limestone will be primarily elastic and are estimated to be one inch or less.

#### 4.2.2 Soil Induced Uplift Loads

The drilled shafts could experience tensile loads as a result of post construction heave in the site soils. The magnitude of these loads varies with the shaft diameter, soil parameters, and particularly the in-situ moisture levels at the time of construction. For design purposes, an uplift load of 1,000 psf over a shaft length of 9 feet is estimated. This load must be resisted by the dead load on the shaft, continuous vertical reinforcing steel in the shaft, and a shaft adhesion developed within the bearing strata. In order to aid in the structural design of the reinforcement, minimum reinforcing should be equal to 0.5 percent of the shaft area.

#### 4.2.3 Drilled Shaft Construction Considerations

Drilled pier construction should be monitored by a representative of the geotechnical engineer to observe, among other things, the following items:

- Identification of bearing material
- Adequate penetration of the shaft excavation into the bearing layer
- The base and sides of the shaft excavation are clean of loose cuttings

- If seepage is encountered, whether it is of sufficient amount to require the use of temporary steel casing. If casing is needed it is important that the field representative observe that a high head of plastic concrete is maintained within the casing at all times during their extraction to prevent the inflow of water

Precautions should be taken during the placement of reinforcing steel and concrete to prevent loose, excavated soil from falling into the excavation. Concrete should be placed as soon as practical after completion of the drilling, cleaning, and observation. Excavation for a drilled pier should be filled with concrete before the end of the workday, or sooner if required to prevent deterioration of the bearing material. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the drilled pier excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface.

Excavations for the shafts should be maintained in the dry. It should be anticipated that ground-water seepage will be encountered during shaft installation of straight shafts and that temporary casing could be required for all straight shafts for proper shaft installation. The casing should be seated below the zone of seepage with all water and most loose material removed prior to beginning the design penetration. Care must then be taken that a sufficient head of plastic concrete is maintained within the casing during extraction.

The concrete should have a slump of 6 inches plus or minus 1 inch. The concrete should be placed in a manner to prevent the concrete from striking the reinforcing cage or the sides of the excavation. Concrete should be tremied to the bottom of the excavation to control the maximum free fall of the plastic concrete to less than 10 feet, or focus concrete in the middle of the reinforcing cage to prevent segregation.

A drilling rig of sufficient size and weight will be necessary for drilling and/or coring through the hard layers to reach the desired bearing stratum and achieve the required penetration.

In addition to the above guidelines, the specifications from the Association of Drilled Shaft Contractors Inc. "Standards and Specifications for the Foundation Drilling Industry" as Revised 1999 or other recognized specifications for proper installation of drilled shaft foundation systems should be followed.

#### 4.2.4 Grade Beams

All grade beams should be supported by the drilled shafts. A minimum 6-inch void space should be provided beneath all grade beams to prevent contact with the swelling clay soils. This void will serve to minimize distress resulting from swell pressures generated by the clays. A void space is not necessary beneath a grade beam situated on competent limestone.

Grade beams may be cast on cardboard carton forms or formed above grade. If cardboard carton forms are used, care should be taken to not crush the carton forms, or allow the carton forms to become wet prior to or during concrete placement operations. A soil retainer or trapezoidal void forms should be provided to help prevent in-filling of this void.

Backfill against the exterior face of grade beams or panels should be properly compacted on-site clays. Compaction should be a minimum of 93 percent of ASTM D 698, at a minimum of 2 percentage points above the optimum moisture content determined by that test. This clay fill is intended to reduce surface water infiltration beneath the structure.

### **4.3 Stiffened, Monolithic Slab-on-Grade**

#### 4.3.1 Design Parameters

A stiffened, monolithically placed slab-on-grade foundation, either rebar or post-tensioned, used at this site must be designed with exterior and interior grade beams to provide sufficient rigidity to tolerate the differential soil movements. These differential movements typically will occur between the periphery and interior of the slab-on-grade system. Foundation movements are anticipated to occur primarily due to post construction heave of the underlying soils but also can occur due to shrinkage of the clays around the perimeter of the slab. It is recommended that all fill soils be properly placed and compacted in accordance with this report section and Section 7.0 prior to foundation installation.

Slab-on-grade construction only should be considered if slab movement can be tolerated. The owner must fully understand that if the floor slab is placed on-grade, some movement and resultant cracking within the floor and interior wall partitions may occur. This upward slab movement and cracking usually is difficult and costly to repair, and may require continued maintenance expense.

The foundation should be designed by a structural engineer familiar with stiffened slabs-on-grade subject to differential movement. Design parameters are presented below for PVR and differential swell using the Post-Tensioning Institute's (PTI) slab-on-grade design method, 2<sup>nd</sup> Edition.

Estimated PVR:	2.5 inches
Edge Moisture Variation	
Approximate Center Lift:	5.5 feet
Approximate Edge Lift:	5.0 feet
Differential Swell	
Approximate Center Lift:	2.5 inches
Approximate Edge Lift:	1.2 inches

It should be recognized that a post tensioned or conventionally reinforced slab-on-grade foundation system placed at this site will be subject to differential movements as indicated above. If slab stiffness is not sufficient to resist the ground movements, these movements can cause cracking of interior sheet rock walls and exterior brick walls. Poor drainage, water leaks, free water sources, long-term percolation in recessed planter areas and/or trees can result in greater differential movements. For example, should leaks develop in underground water or sewer lines or the grades around the structures are changed and cause ponding of water, unacceptable slab movements could develop. A greater risk of unsatisfactory foundation performance exists with a slab-on-grade design than for a drilled shaft or spread foundation design.

Beams may be designed based on an allowable soil bearing pressure of 2,000 pounds per square foot or less within the shallow soils or atop tan limestone. The beams should extend at least 12 inches into natural, undisturbed soil or compacted and tested fill. The beam depth is given in regard to bearing capacity and is not intended to be a structural recommendation.

A properly engineered and constructed vapor barrier should be provided beneath slabs-on-grade which will be carpeted or receive moisture sensitive coverings or adhesives.

## **5.0 INTERIOR FLOOR SLABS AND EXTERIOR FLATWORK**

### **5.1 Potential Vertical Movements**

Lightly loaded floor slabs and exterior flatwork placed on-grade will be subject to movement as a result of moisture induced volume changes in the highly active clays. The clays expand (heave) with increases in moisture and contract (shrink) with decreases in moisture. The movement typically occurs as post construction heave. The potential magnitude of the moisture induced movements is rather indeterminate. It is influenced by the soil properties, overburden pressures, and to a great extent by soil moisture levels at the time of construction. The greatest potential for post-construction movement occurs when the soils are in a dry condition at the time of construction. Based on the conditions encountered in the borings, potential moisture induced movements are estimated to be on the order of 2.5 inches for soils in a dry condition. Soil movements, significantly larger than estimated, could occur due to inadequate site grading, poor drainage, ponding of rainfall, and/or leaking pipelines.

### **5.2 Structurally Suspended Floor Slab**

The most positive method of preventing slab distress due to swelling soils is to structurally suspend the interior slab. Due to the expansion potential of the site clays we recommend that the suspended floor slab be constructed on carton forms with a minimum 8-inch void space or a crawl space.

Care should be taken to assure that the void boxes are not allowed to become wet or crushed prior to or during concrete placement and finishing operations. Corrugated steel, placed on the top of the carton forms, could be used to reduce the risk of crushing of the carton forms during concrete placement and finishing operations. As a quality control measure during construction, "actual" concrete quantities placed should be checked against "anticipated" quantities. Significant concrete "overage" would be an early indication of a collapsed void.

Provisions should be made to provide drainage from under the building. Ventilation of the void below the floors should be provided if high humidity can cause problems with floor tile adhesives.

Vehicle or pedestrian ramps leading up to the building should be structurally connected to the building grade beams to avoid abrupt differential movement between the building slab and the ramps. Transitioning details will be required at the points where ramps connect with paving and

slab on-grade elements. In addition, ramp slabs should be constructed so that slopes sufficient for effective drainage of surface water are still provided after potential differential movements

### **5.3 Ground-Supported Floor Slabs and Exterior Flatwork**

In conjunction with drilled shafts, interior slabs and/or exterior flatwork can be placed on a prepared subgrade. Slab-on-grade construction should only be considered if slab movement can be tolerated. The level of acceptable movement varies with the user, but methods are normally selected with the goal of limiting slab movements to about one inch or less. Reductions in anticipated movements can be achieved by using methods developed in this area to reduce on-grade slab movements. The more commonly used methods consist of placing non-expansive select fill beneath the slab and moisture conditioning the soils. The use of these methods will not eliminate the risk of unacceptable movements.

Readers should understand that a ground-supported floor slab can heave considerably if placed on dry, expansive clays. The installation of a minimum of 1 foot of non-expansive select fill over a minimum of 4 feet of moisture conditioned clays should reduce potential movements to on the order of 1 inch. Moisture conditioning can be achieved by mechanically reworking the clays as discussed below. Slabs not capable of tolerating this level of movement should be structurally suspended. These recommendations should be reviewed once a grading plan is finalized.

Consideration should be given to extending the moisture conditioning process beyond the building line to include entrances or other areas sensitive to movement. Outside the building, a single lift of select fill (6 to 8 inches) is recommended to minimize drying during construction.

Soil treatments presented in this section are referenced as an alternative to the use of a structurally suspended floor slab. The owner must fully understand that if the floor slab is placed on-grade, some movement and resultant cracking within the floor and interior wall partitions may occur. This upward slab movement and cracking is usually difficult and costly to repair, and may require continued maintenance expense.

These methods of treatment are presented as an option for the owner's consideration. The options may or may not be practical or economically feasible, depending on the expected performance of the proposed structure. The owner should be aware that this method will not prevent movement of soil-supported foundation elements, and can only reduce the magnitude of the movement.

Placement of the floor slab-on-grade represents a compromise between construction cost and risk of floor distress.

A properly engineered and constructed vapor barrier should be provided beneath slabs-on-grade which will be carpeted or receive moisture sensitive coverings or adhesives.

The following moisture conditioning procedure is presented:

### 5.3.1 Mechanical Reworking of Near-Surface Clays with 1-Foot Select Fill Cap

In general, the procedure is performed as follows:

1. Remove all existing pavements, surface vegetation, trees and associated root mats, organic topsoil and any other deleterious material, particularly the wood mulch encountered in Boring B-2.
2. Excavate to a minimum of 4.5 feet below finished grade. Scarify the exposed subgrade at the base of the excavation to a depth of 8 inches, adjust the moisture, and compact at a minimum of 3 percentage points above optimum moisture to between 93 and 98 percent of Standard Proctor density (ASTM D 698). Over-compaction should not be allowed.
3. Fill pad to 1 foot below final grade using site excavated or similar clay soils. Compact in maximum 9-inch loose lifts at a minimum of 3 percentage points above optimum moisture to between 93 and 98 percent of Standard Proctor density (ASTM D 698). Over-compaction should not be allowed.
4. Complete pad fill using a minimum of 1 foot of sandy clay/clayey sand non-expansive select fill with a Liquid Limit less than 35 and a Plasticity Index (PI) between 5 and 16. The select fill should be compacted in maximum 9-inch loose lifts at minus 2 to plus 3 percentage points of the soil's optimum moisture content at a minimum of 95 percent of Standard Proctor density (ASTM D 698). The select fill should be placed within 48 hours of completing the installation of the moisture conditioned soils.

## **6.0 EXPANSIVE SOIL CONSIDERATIONS**

### **6.1 Site Drainage**

An important feature of the project is to provide positive drainage away from the proposed building. If water is permitted to stand next to or below the structure, excessive soil movements (heave) can occur. This could result in differential floor slab or foundation movement.

A well-designed site drainage plan is of utmost importance and surface drainage should be provided during construction and maintained throughout the life of the structure. Consideration should be given to the design and location of gutter downspouts, planting areas, or other features.

which would produce moisture concentration adjacent to or beneath the structure or paving. Consideration should be given to the use of self-contained, watertight planters. Joints next to the structure should be sealed with a flexible joint sealer to prevent infiltration of surface water. Proper maintenance should include periodic inspection for open joints and cracks and resealing as necessary.

Rainwater collected by the gutter system should be transported by pipe to a storm drain or to a paved area. If downspouts discharge next to the structure onto flatwork or paved areas, the area should be watertight in order to eliminate infiltration next to the building.

## **6.2 Additional Design Considerations**

The following information has been assimilated after examination of numerous projects constructed in active soils throughout the area. It is presented here for your convenience. If these features are incorporated in the overall design of the project, the performance of the structure should be improved.

- Special consideration should be given to completion items outside the building area, such as stairs, sidewalks, signs, etc. They should be adequately designed to sustain the potential vertical movements mentioned in the report.
- Roof drainage should be collected by a system of gutters and downspouts and transmitted away from the structure where the water can drain away without entering the building subgrade.
- Sidewalks should not be structurally connected to the building. They should be sloped away from the building so that water will drain away from the structure.
- The paving and the general ground surface should be sloped away from the building on all sides so that water will always drain away from the structure. Water should not be allowed to pond near the building after the slab has been placed.
- Every attempt should be made to limit the extreme wetting or drying of the subsurface soils since swelling and shrinkage will result. Standard construction practices of providing good surface water drainage should be used. A positive slope of the ground away from the foundation should be provided to carry off the run-off water both during and after construction.
- Backfill for utility lines or along the perimeter beams should consist of on-site material so that they will be stable. If the backfill is too dense or too dry, swelling may form a mound along the ditch line. If the backfill is too loose or too wet, settlement may form a sink along the ditch line. Either case is undesirable since several inches of movement is possible and floor cracks are likely to result. The soils should be processed using the previously discussed compaction criteria.

## **7.0 EARTHWORK**

### **7.1 Site Preparation**

The existing ground surface should be stripped of vegetation, roots, deleterious materials, and old construction debris. It is estimated that the depth of stripping will be on the order of 4 to 8 inches. The actual stripping depth should be based on field observations with particular attention given to old drainage areas, uneven topography, and excessively wet soils. The stripped areas should be observed to determine if additional excavation is required to remove weak or otherwise objectionable materials that would adversely affect the fill placement or other construction activities.

The subgrade should be firm and able to support the construction equipment without displacement. Soft or yielding subgrade should be corrected and made stable before construction proceeds. The subgrade should be proof rolled to detect soft spots, which if exist, should be excavated to provide a firm and otherwise suitable subgrade. Proof rolling should be performed using a heavy pneumatic tired roller, loaded dump truck, or similar piece of equipment. The proof rolling operations should be observed by the project geotechnical engineer or his/her representative.

### **7.2 Placement and Compaction**

Fill material should be placed in loose lifts not exceeding 8 inches in uncompacted thickness. The uncompacted lift thickness should be reduced to 4 inches for structure backfill zones requiring hand-operated power compactors or small self-propelled compactors. The fill material should be uniform with respect to material type and moisture content. Clods and chunks of material should be broken down and the fill material mixed by diskings, blading, or plowing, as necessary, so that a material of uniform moisture and density is obtained for each lift. Water required for sprinkling to bring the fill material to the proper moisture content should be applied evenly through each layer.

The on-site soils are suitable for use in general site grading. Imported fill material should be clean soil with a Liquid Limit less than 50 and no rock greater than 4 inches in maximum dimension. The fill materials should be free of vegetation and debris.

The fill material should be compacted to a density ranging from 95 to 100 percent of maximum dry density as determined by ASTM D 698, Standard Proctor. In conjunction with the compacting operation, the fill material should be brought to the proper moisture content. The moisture content for general earth fill should range from 2 percentage points below optimum to 5 percentage points above optimum (-2 to +5). These ranges of moisture contents are given as maximum recommended ranges. For some soils and under some conditions, the contractor may have to maintain a more narrow range of moisture content (within the recommended range) in order to consistently achieve the recommended density.

Field density tests should be taken as each lift of fill material is placed. As a guide, one field density test per lift for each 5,000 square feet of compacted area is recommended. For small areas or critical areas the frequency of testing may need to be increased to one test per 2,500 square feet. A minimum of 2 tests per lift should be required. The earthwork operations should be observed and tested on a continuing basis by an experienced geotechnician working in conjunction with the project geotechnical engineer.

Each lift should be compacted, tested, and approved before another lift is added. The purpose of the field density tests is to provide some indication that uniform and adequate compaction is being obtained. The actual quality of the fill, as compacted, should be the responsibility of the contractor and satisfactory results from the tests should not be considered as a guarantee of the quality of the contractor's filling operations.

### **7.3 Trench Backfill**

Trench backfill for pipelines or other utilities should be properly placed and compacted. Overly dense or dry backfill can swell and create a mound along the completed trench line. Loose or wet backfill can settle and form a depression along the completed trench line. Distress to overlying structures, pavements, etc. is likely if heaving or settlement occurs. On-site soil fill material is recommended for trench backfill. Care should be taken not to use free draining granular material, to prevent the backfilled trench from becoming a french drain and piping surface or subsurface water beneath structures, pipelines, or pavements. If a higher class bedding material is required for the pipelines, a lean concrete bedding will limit water intrusion into the trench and will not require compaction after placement. The soil backfill should be placed in approximately 4- to 6-inch loose lifts. The density and moisture content should be as recommended for fill in Section 7.2,

Placement and Compaction, of this report. A minimum of one field density test should be taken per lift for each 150 linear feet of trench, with a minimum of 2 tests per lift

#### 7.4 Excavation

The side slopes of excavations through the overburden soils should be made in such a manner to provide for their stability during construction. Existing structures, pipelines or other facilities, which are constructed prior to or during the currently proposed construction and which require excavation, should be protected from loss of end bearing or lateral support.

Temporary construction slopes and/or permanent embankment slopes should be protected from surface runoff water. Site grading should be designed to allow drainage at planned areas where erosion protection is provided, instead of allowing surface water to flow down unprotected slopes.

Permanent slopes at the site should be as flat as practical to reduce creep and occurrence of shallow slides. The following slope angles are recommended as maximums.

<b>Height (ft.)</b>	<b>Horizontal to Vertical</b>
0 – 3	1:1
3 – 6	2:1
6 – 9	3:1
> 9	4:1

The presented angles refer to the total height of a slope. Site improvement should be maintained away from the top of the slope to reduce the possibility of damage due to creep or shallow slides.

Trench safety recommendations are beyond the scope of this report. The contractor must comply with all applicable safety regulations concerning trench safety and excavations including, but not limited to, OSHA regulations.

#### 7.5 Acceptance of Imported Fill

Any soil imported from off-site sources should be tested for compliance with the recommendations for the particular application and approved by the project geotechnical engineer prior to the materials being used. The owner should also require the contractor to obtain a written, notarized

certification from the landowner of each proposed off-site soil borrow source stating that to the best of the landowner's knowledge and belief there has never been contamination of the borrow source site with hazardous or toxic materials. The certification should be furnished to the owner prior to proceeding to furnish soils to the site. Soil materials derived from the excavation of underground petroleum storage tanks should not be used as fill on this project.

## **7.6 Soil Corrosion Potential**

Specific testing for soil corrosion potential was not included in the scope of this study. However, based upon past experience on other projects in the vicinity, the soils at this site may be corrosive. Standard construction practices for protecting metal pipe and similar facilities in contact with these soils should be used.

## **7.7 Erosion and Sediment Control**

All disturbed areas should be protected from erosion and sedimentation during construction, and all permanent slopes and other areas subject to erosion or sedimentation should be provided with permanent erosion and sediment control facilities. All applicable ordinances and codes regarding erosion and sediment control should be followed.

# **8.0 PAVEMENTS**

## **8.1 Pavement Subgrade Preparation**

The surficial soils typically consisted of highly active clays. These clays are subject to loss in support value with the moisture increases which occur beneath pavement sections. They react with hydrated lime, which serves to improve and maintain their support value. Treatment of these soils with hydrated lime will improve their subgrade characteristics to support area paving.

Lime treatment is recommended to support Portland cement concrete subject to heavy truck or bus traffic, although concrete paving subject to automobile and light truck traffic only generally will perform satisfactorily if placed on a prepared (untreated) subgrade.

Prior to lime stabilization or compaction, the subgrade should be proofrolled with heavy pneumatic equipment. Any soft or pumping areas should be undercut to a firm subgrade and properly backfilled as described in the Section 7, Earthwork. The subgrade, stabilized or unstabilized, should be scarified

to a minimum depth of 6 inches and uniformly compacted to a minimum of 95 percent of Standard Proctor density (ASTM D 698), near minus 2 to plus 4 percentage points of the optimum moisture content determined by that test. It should then be protected and maintained in a moist condition until the pavement is placed.

We recommend a minimum of 7 percent hydrated lime be used to modify the clay subgrade soils. The estimated amount of hydrated lime required to stabilize the subgrade should be on the order of 32 pounds per square yard for a 6-inch depth, based on a soil dry unit weight of 100 pcf. The hydrated lime should be thoroughly mixed and blended with the upper 6 inches of the clay subgrade (TxDOT Item 260). The hydrated lime should meet the requirements of Item 260 in the Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets and Bridges, 2004 Edition. Lime treatment should extend beyond exposed pavement edges to reduce the effects of shrinkage and associated loss of subgrade support.

We recommend that subgrade stabilization extend to at least one foot beyond pavement edges to aid in reducing pavement movements and cracking along the curb line due to seasonal moisture variations after construction. Each construction area should be shaped to allow drainage of surface water during earthwork operations, and surface water should be pumped immediately from each construction area after each rain and a firm subgrade condition maintained. Water should not be allowed to pond in order to prevent percolation and subgrade softening, and lime should be added to the subgrade after removal of all surface vegetation and debris. Sand should be specifically prohibited beneath pavement areas, since these more porous soils can allow water inflow, resulting in heave and strength loss of subgrade soils (lime stabilized soil will be allowed for fine grading). After fine grading each area in preparation for paving, the subgrade surface should be lightly moistened, as needed, and recompact to obtain a tight non-yielding subgrade.

## **8.2 Pavement Sections**

The project may include the construction of parking lots and/or drives. At the time of this investigation, site paving plans or vehicle traffic studies were not available. Therefore, several rigid pavement sections are presented for a 20-year design life based on our experience with similar facilities for Light Duty Parking Areas, Medium Duty Parking Areas, and Medium to Heavy Duty Drives. In general, these areas are defined as follows:

Light-Duty Parking Areas are those lots and drives subjected almost exclusively to passenger cars, with an occasional light- to medium-duty truck (2 to 3 per week)

Medium-Duty Parking Areas are those lots subjected to a variety of light-duty vehicles to medium-duty vehicles and an occasional heavy-duty truck (1 to 2 per week).

Medium to Heavy-Duty Drives are those drives subjected to a variety of light to heavy-duty vehicles. These pavements include areas subject to significant truck traffic or trash vehicles

We recommend that rigid pavements be utilized at this project whenever possible, since they tend to provide better long-term performance when subjected to significant slow moving and turning traffic.

For Portland cement concrete pavement, a minimum thickness of 5 inches of concrete is recommended for light-duty parking areas, 6 inches for medium-duty parking areas, and 7 inches for medium to heavy-duty areas

A California Bearing Ratio or other strength tests were not performed because they were not within the scope of our services on this project. A subgrade modulus of 100 psi was considered appropriate for the near-surface soils. If heavier vehicles are planned, the above cross sections can be confirmed by performing strength tests on the subgrade materials once the traffic characteristics are established. Periodic maintenance of pavement structures normally improves the durability of the overall pavement and enhances its expected life.

The above sections should be considered minimum pavement thicknesses and higher traffic volumes and heavy trucks may require thicker pavement sections. Additional recommendations can be provided after traffic volumes and loads are known. Periodic maintenance should be anticipated for minimum pavement thickness. This maintenance should consist of sealing cracks and timely repair of isolated distressed areas.

### **8.3 Pavement Material Requirements**

Reinforced Portland Cement Concrete: Reinforced Portland cement concrete pavement should consist of Portland cement concrete having a 28-day compressive strength of at least 3,500 psi. The mix should be designed in accordance with the ACI Code 318 using 3 to 6 percent air entrainment. The pavement should be adequately reinforced with temperature steel and all

construction joints or expansion/contraction joints should be provided with load transfer dowels. The spacing of the joints will depend primarily on the type of steel used in the pavement. We recommend using No. 3 steel rebar spaced at 18 inches on center in both the longitudinal and transverse direction. Control joints formed by sawing are recommended every 12 to 15 feet in both the longitudinal and transverse direction. The cutting of the joints should be performed as soon as the concrete has "set-up" enough to allow for sawing operations.

Lime Stabilized Subgrade: Item 260, Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2004 Edition

#### **8.4 General Pavement Considerations**

The design of the pavement drainage and grading should consider the potential for differential ground movement due to future soil swelling of up to 2.5 inches. In order to minimize rainwater infiltration through the pavement surface, and thereby minimizing future upward movement of the pavement slabs all cracks and joints in the pavement should be sealed on a routine basis after construction.

### **9.0 CONSTRUCTION OBSERVATIONS**

In any geotechnical investigation, the design recommendations are based on a limited amount of information about the subsurface conditions. In the analysis, the geotechnical engineer must assume the subsurface conditions are similar to the conditions encountered in the borings. However, quite often during construction anomalies in the subsurface conditions are revealed. Therefore, it is recommended that CMJ Engineering, Inc. be retained to observe earthwork and foundation installation and perform materials evaluation during the construction phase of the project. This enables the geotechnical engineer to stay abreast of the project and to be readily available to evaluate unanticipated conditions, to conduct additional tests if required and, when necessary, to recommend alternative solutions to unanticipated conditions. Until these construction phase services are performed by the project geotechnical engineer, the recommendations contained in this report on such items as final foundation bearing elevations, proper soil moisture condition, and other such subsurface related recommendations should be considered as preliminary.

It is proposed that construction phase observation and materials testing commence by the project geotechnical engineer at the outset of the project. Experience has shown that the most suitable method for procuring these services is for the owner or the owner's design engineers to contract directly with the project geotechnical engineer. This results in a clear, direct line of communication between the owner and the owner's design engineers and the geotechnical engineer.

## **10.0 REPORT CLOSURE**

The boring logs shown in this report contain information related to the types of soil encountered at specific locations and times and show lines delineating the interface between these materials. The logs also contain our field representative's interpretation of conditions that are believed to exist in those depth intervals between the actual samples taken. Therefore, these boring logs contain both factual and interpretive information. Laboratory soil classification tests were also performed on samples from selected depths in the borings. The results of these tests, along with visual-manual procedures were used to generally classify each stratum. Therefore, it should be understood that the classification data on the logs of borings represent visual estimates of classifications for those portions of each stratum on which the full range of laboratory soil classification tests were not performed. It is not implied that these logs are representative of subsurface conditions at other locations and times.

With regard to ground-water conditions, this report presents data on ground-water levels as they were observed during the course of the field work. In particular, water level readings have been made in the borings at the times and under conditions stated in the text of the report and on the boring logs. It should be noted that fluctuations in the level of the ground-water table can occur with passage of time due to variations in rainfall, temperature and other factors. Also, this report does not include quantitative information on rates of flow of ground water into excavations, on pumping capacities necessary to dewater the excavations, or on methods of dewatering excavations. Unanticipated soil conditions at a construction site are commonly encountered and cannot be fully predicted by mere soil samples, test borings or test pits. Such unexpected conditions frequently require that additional expenditures be made by the owner to attain a properly designed and constructed project. Therefore, provision for some contingency fund is recommended to accommodate such potential extra cost.

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our field investigation and further on the assumption that the exploratory borings are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the borings at the time they were completed. If, during construction, different subsurface conditions from those encountered in our borings are observed, or appear to be present in excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between submission of this report and the start of the work at the site, if conditions have changed due either to natural causes or to construction operations at or adjacent to the site, or if structure locations, structural loads or finish grades are changed, we urge that we be promptly informed and retained to review our report to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse

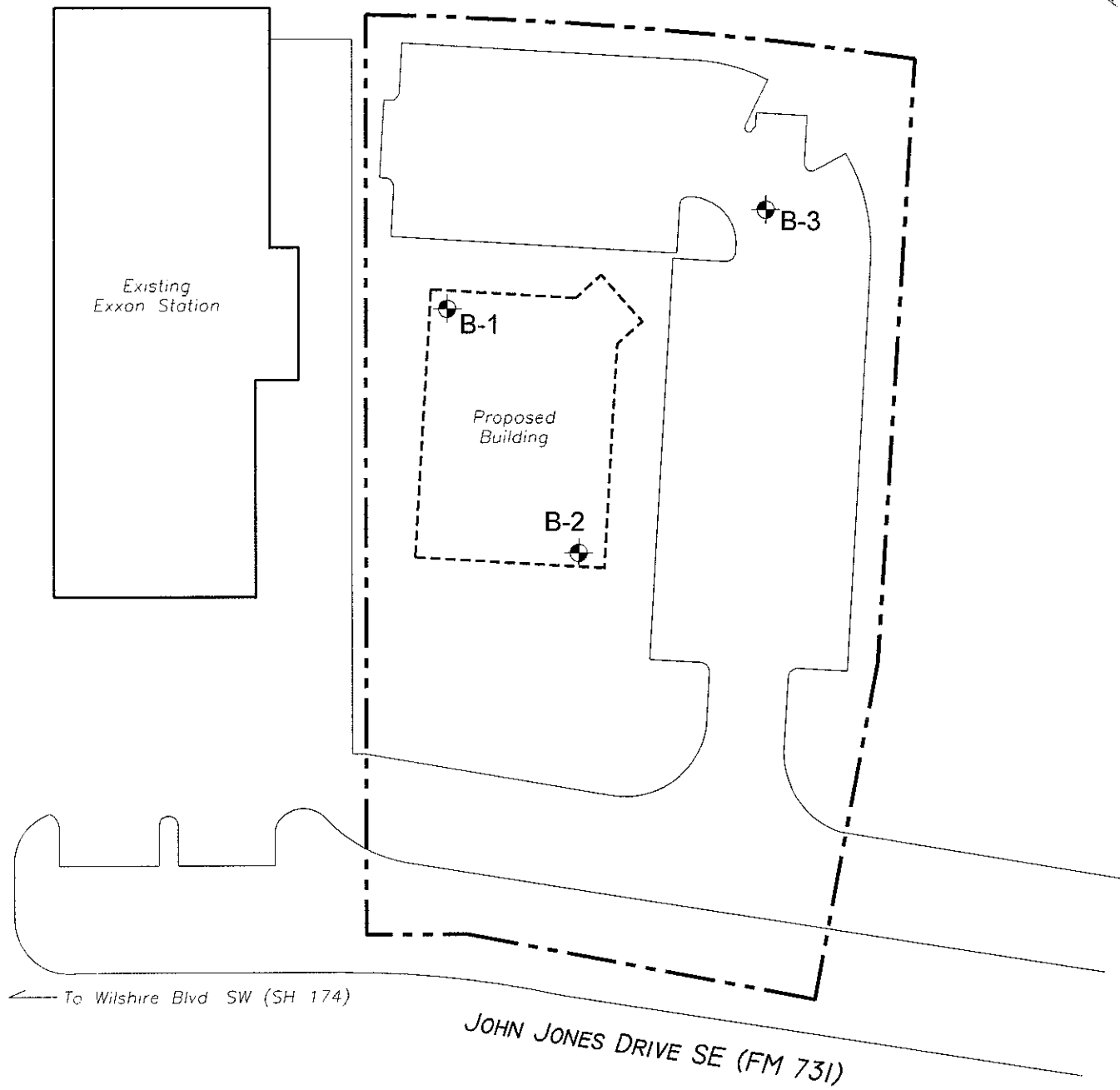
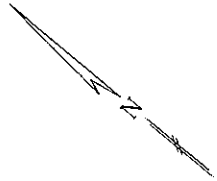
Further, it is urged that CMJ Engineering, Inc. be retained to review those portions of the plans and specifications for this particular project that pertain to earthwork and foundations as a means to determine whether the plans and specifications are consistent with the recommendations contained in this report. In addition, we are available to observe construction, particularly the compaction of structural fill, or backfill and the construction of foundations as recommended in the report, and such other field observations as might be necessary

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, ground water or air, on or below or around the site.

This report has been prepared for use in developing an overall design concept. Paragraphs, statements, test results, boring logs, diagrams, etc. should not be taken out of context, nor utilized without a knowledge and awareness of their intent within the overall concept of this report. The reproduction of this report, or any part thereof, supplied to persons other than the owner, should indicate that this study was made for design purposes only and that verification of the subsurface conditions for purposes of determining difficulty of excavation, trafficability, etc. are responsibilities of the contractor.

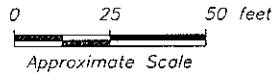
This report has been prepared for the exclusive use of Baird, Hampton & Brown, Inc. for specific application to design of this project. The only warranty made by us in connection with the services provided is that we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, expressed or implied, is made or intended.

\* \* \* \*



LEGEND.

- Boring Location
- Approximate Site Boundary



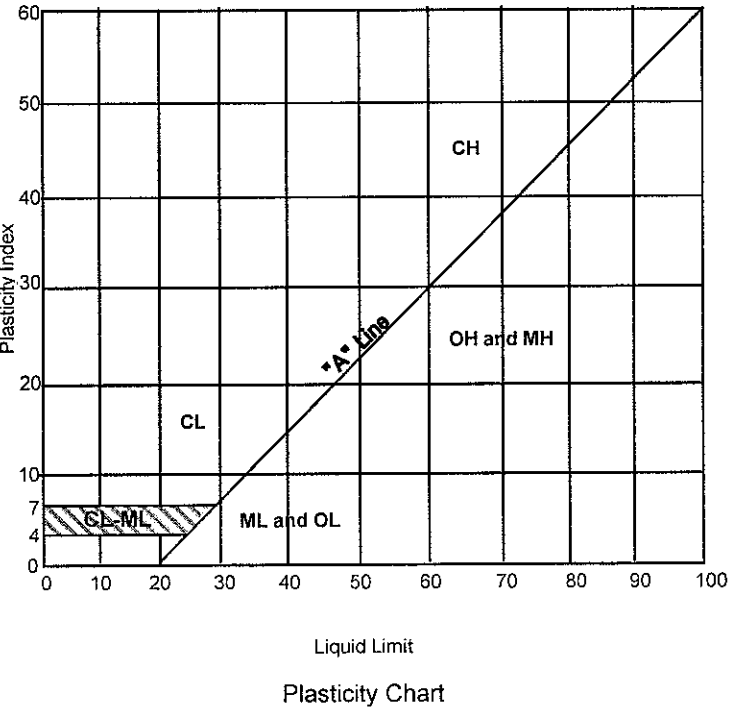
**CMJ** ENGINEERING, INC.  
CMJ PROJECT No. 110-08-28

**PLAN OF BORINGS**  
COOK'S CHILDREN'S MOB  
JOHN JONES DRIVE SE AT  
WILSHIRE BOULEVARD SW  
BURLESON, TEXAS

**PLATE**  
**A.1**

Major Divisions		Grp Sym.	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (more than half of the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4: $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
		GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW		
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures	Liquid and Plastic limits below "A" line or P.I. greater than 4  Liquid and Plastic limits above "A" line with P.I. greater than 7	Liquid and plastic limits plotting in hatched zone between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		
		Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6: $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP	Poorly graded sands; gravelly sands, little or no fines	Not meeting all gradation requirements for SW	
	Sands with fines (Appreciable amount of fines)		SM	Silty sands, sand-silt mixtures	Liquid and Plastic limits below "A" line or P.I. less than 4  Liquid and Plastic limits above "A" line with P.I. greater than 7	Liquid and plastic limits plotting between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		
	Fine-grained soils (More than half of material is smaller than No. 200 sieve)		Sils and clays (Liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	<p>Plasticity Index</p> <p>Liquid Limit</p> <p>Plasticity Chart</p>
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays	
		OL		Organic silts and organic silty clays of low plasticity		
		Sils and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
CH				Inorganic clays of high plasticity, fat clays		
OH				Organic clays of medium to high plasticity, organic silts		
Highly Organic soils		Pt	Peat and other highly organic soils			

Determine percentages of sand and gravel from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:  
 Less than 5 percent.....GW, GP, SW, SP  
 More than 12 percent.....GM, GC, SM, SC  
 5 to 12 percent.....Borderline cases requiring dual symbols



SOIL OR ROCK TYPES											
	GRAVEL		LEAN CLAY		LIMESTONE						
	SAND		SANDY		SHALE						
	SILT		SILTY		SANDSTONE						
	CLAYEY		HIGHLY PLASTIC CLAY		CONGLOMERATE	Shelby Tube	Auger	Split Spoon	Rock Core	Cone Pen	No Recovery

### TERMS DESCRIBING CONSISTENCY, CONDITION, AND STRUCTURE OF SOIL

Fine Grained Soils (More than 50% Passing No. 200 Sieve)

Descriptive Item	Penetrometer Reading, (tsf)
Soft	0.0 to 1.0
Firm	1.0 to 1.5
Stiff	1.5 to 3.0
Very Stiff	3.0 to 4.5
Hard	4.5+

Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

Penetration Resistance (blows/foot)	Descriptive Item	Relative Density
0 to 4	Very Loose	0 to 20%
4 to 10	Loose	20 to 40%
10 to 30	Medium Dense	40 to 70%
30 to 50	Dense	70 to 90%
Over 50	Very Dense	90 to 100%

### Soil Structure

Calcareous	Contains appreciable deposits of calcium carbonate; generally nodular
Slickensided	Having inclined planes of weakness that are slick and glossy in appearance
Laminated	Composed of thin layers of varying color or texture
Fissured	Containing cracks, sometimes filled with fine sand or silt
Interbedded	Composed of alternate layers of different soil types, usually in approximately equal proportions

### TERMS DESCRIBING PHYSICAL PROPERTIES OF ROCK

#### Hardness and Degree of Cementation

Very Soft or Plastic	Can be remolded in hand; corresponds in consistency up to very stiff in soils
Soft	Can be scratched with fingernail
Moderately Hard	Can be scratched easily with knife; cannot be scratched with fingernail
Hard	Difficult to scratch with knife
Very Hard	Cannot be scratched with knife
Poorly Cemented or Friable	Easily crumbled
Cemented	Bound together by chemically precipitated material; Quartz, calcite, dolomite, siderite, and iron oxide are common cementing materials.

#### Degree of Weathering

Unweathered	Rock in its natural state before being exposed to atmospheric agents
Slightly Weathered	Noted predominantly by color change with no disintegrated zones
Weathered	Complete color change with zones of slightly decomposed rock
Extremely Weathered	Complete color change with consistency, texture, and general appearance approaching soil

Project No <b>110-08-28</b>		Boring No <b>B-1</b>		Project <b>Cook Children's Medical Office Building FM 731 at SH 174 - Burleson, Texas</b>								
Location <b>See Plate A.1</b>			Water Observations <b>Seepage at 9' during drilling; dry at completion</b>									
Completion Depth <b>20.0'</b>		Completion Date <b>9-29-08</b>										
Depth, Ft.	Symbol	Surface Elevation <b>N/A</b>	Type <b>B-53, w/ 6" CFA</b>									
		<b>Stratum Description</b>										
			REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		<b>CLAY / SILTY CLAY</b> , dark brown w/ ironstone nodules, very stiff to hard			4.5+					13		
					4.5+					16		
					3.75					18		
					3.75		57	22	35	20	99	
					3.75					21		
5		<b>SILTY CLAY</b> , light brown, w/ ironstains and ironstone nodules, very stiff										
					3.5					20		
		-w/ gray and abundant gravel, 9' to 11 5'			4.0		40	15	25	15		
10												
		<b>LIMESTONE</b> , tan, weathered w/ clay seams										
		<b>LIMESTONE</b> , gray, w/ shale seams, very hard										
15					100/0.25'							
20					100/0.5"							

LOG OF BORING 110-08-28.GPJ CMJ.GDT 10/8/08

Project No <b>110-08-28</b>		Boring No. <b>B-2</b>		Project <b>Cook Children's Medical Office Building FM 731 at SH 174 - Burleson, Texas</b>											
Location <b>See Plate A.1</b>		Water Observations <b>Seepage at 9' during drilling; dry at completion</b>													
Completion Depth <b>20.0'</b>		Completion Date <b>9-29-08</b>													
Surface Elevation <b>N/A</b>		Type <b>B-53, w/ 6" CFA</b>													
Depth, Ft.	Symbol	Samples	<b>Stratum Description</b>			REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
			<b>CLAY / SILTY CLAY</b> , dark brown, w/ ironstone nodules and calcareous nodules very stiff to hard			4.5+						13			
			<b>SILTY CLAY</b> , light brown, w/ calcareous nodules hard			4.0						18	107	6530	
						4.5+						15			
						4.5+						12			
			-w/ abundant gravel stiff 7' to 9'			4.5+		26	15	11	13				
			<b>LIMESTONE</b> , tan, w/ clay seams, hard			2.0						13			
						100/1.25'									
			<b>LIMESTONE</b> , gray, w/ shale seams, very hard			100/0.5"									
						100/0.75'									

LOG OF BORING 110-08-28.GPJ CMJ.GDT 10/8/08

Project No <b>110-08-28</b>	Boring No. <b>B-3</b>	Project <b>Cook Children's Medical Office Building FM 731 at SH 174 - Burleson, Texas</b>	
Location <b>See Plate A.1</b>		Water Observations <b>Dry during drilling; dry at completion</b>	
Completion Depth <b>5.0'</b>	Completion Date <b>9-29-08</b>		

Depth, Ft.	Symbol	Samples	Surface Elevation <b>N/A</b>	Type <b>B-53, w/ 6" CFA</b>	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
<b>Stratum Description</b>														
5	[Hatched Box]			<b>SILTY CLAY / CLAY</b> , dark brown w/ calcareous nodules, hard			4.5+		46	17	29	9		
							4.5+					19		
							4.5+					18		
							4.5+					20		
							4.5+					17		

LOG OF BORING 110-08-28.GPJ CMJ.GDT 10/6/08

## FREE SWELL TEST RESULTS

Project: Cook Children's Medical Office Building  
FM 731 at SH 174 - Burleson, Texas

Project No.: 110-08-28

Boring No.	Depth Interval (ft.)	Sample Description	Liquid Limit	Plastic Limit	Plasticity Index	Moisture Content %		Percent Swell (%)
			LL	PL	PI	Initial	Final	
B-1	3 - 4	Clay / Silty Clay	57	22	35	20.5	25.2	0.9

Free swell tests performed at approximate overburden pressure