

GEOTECHNICAL ENGINEERING REPORT



SHAW'S JEWELRY
HOUSTON, TEXAS

GEOTECHNICAL ENGINEERING REPORT

Shaw's Jewelry
Houston, Texas

Prepared by:



Gorrondona & Associates, Inc.

Prepared for:

Hermes Architects
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Houston, Texas 77027

Attention: Mr. Daren Penewitt

October 2011

G&AI Project No. 11-0285



Gorrondona & Associates, Inc.

Subsurface Utility Engineering • Utility Coordination • Geotechnical Engineering • Materials Testing

October 11, 2011

Mr. Daren Penewitt
Hermes Architects
1177 West Loop South, Suite 500
Houston, Texas 77027

**Re: GEOTECHNICAL ENGINEERING REPORT
Shaw's Jewelry
Houston, Texas
G&AI Project No. 11-0285**

Dear Mr. Penewitt:

Gorrondona & Associates, Inc. (G&AI) is pleased to submit our Geotechnical Engineering Report for the above-referenced project. This investigation was performed in accordance with G&AI Proposal No. P11-0250 dated August 5, 2011.

We appreciate the opportunity to work with you on this project. We look forward to providing you with geotechnical and materials testing and inspection services during the construction phase of this project.

Please contact us if you have any questions or require additional services.

Respectfully submitted,

Kenneth B. Riner, P.E.
Director – Geotechnical, Environmental & CMT Services

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1.0 INTRODUCTION

Project Location. The project is located at the southeast corner of Montrose Boulevard and West Gray Street in Houston, Texas. The general location and orientation of the site are provided in Appendix A - Project Location Diagrams.

Project Description. The project consists of a proposed single-story, 6,000 SF retail building and associated parking and drive areas.

Project Authorization. This geotechnical investigation was authorized by Mr. Daren Penewitt with Hermes Architects and performed in accordance with G&AI Proposal No. P11-0250 dated August 5, 2011.

Purpose and Methodology. The principal purposes of this investigation were to evaluate the general soil conditions at the proposed site and to develop geotechnical engineering design recommendations. To accomplish its intended purposes, the study was conducted in the following phases: (1) drill sample borings to evaluate the soil conditions at the boring locations and to obtain soil samples; (2) conduct laboratory tests on selected samples recovered from the borings to establish the pertinent engineering characteristics of the soils; and (3) perform engineering analyses, using field and laboratory data, to develop design criteria.

Cautionary Statement Regarding Use of this Report. As with any geotechnical engineering report, this report presents technical information and provides detailed technical recommendations for civil and structural engineering design and construction purposes. G&AI, by necessity, has assumed the user of this document possesses the technical acumen to understand and properly utilize information and recommendations provided herein. G&AI strives to be clear in its presentation and, like the user, does not want potentially detrimental misinterpretation or misunderstanding of this report. Therefore, we encourage any user of this report with questions regarding its content to contact G&AI for clarification. Clarification will be provided verbally and/or issued by G&AI in the form of a report addendum, as appropriate. Any necessary clarification to this report will be provided at no charge.

2.0 FIELD INVESTIGATION

The subsurface investigation for this project is summarized below. Boring locations are provided in Appendix B - Boring Location Diagram.

Boring Nos.	Depth, feet bgs ¹	Date Drilled	Location ²
B-01 and B-02	20	9/26/2011	Building Area
B-03 to B-05	5	9/26/2011	Paving Area

Notes:

1. bgs = below ground surface
2. Boring locations provided in Appendix B - Boring Location Diagram were not surveyed and should be considered approximate. Borings were located by recreational hand-held GPS unit. Horizontal accuracy of such units is typically on the order of 20-feet.

Subsurface conditions were defined using the sample borings. Boring logs generated during this study are included in Appendix C - Boring Logs and Laboratory Results. Borings were advanced between sample intervals using continuous flight auger drilling procedures.

Cohesive soil samples were generally obtained using Shelby tube samplers in general accordance with American Society for Testing and Materials (ASTM) D1587. 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 undisturbed soils by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency using a hand penetrometer, sealed and packaged to maintain "in situ" moisture content.

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 undisturbed sample at a constant rate to a depth of 0.25-inch. The results of these tests are tabulated at the respective sample depths on the boring logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

Granular soil samples were generally obtained using split-barrel sampling procedures in general accordance with ASTM D1586. In the split-barrel procedure, a disturbed sample is obtained in a standard 2-inch outside diameter (OD) split barrel sampling spoon driven 18-inches into the ground using a 140-pound (lb) hammer falling freely 30 inches. The number of blows for the last 12-inches of a standard 18-inch penetration is recorded as the Standard Penetration Test resistance (N-value). The N-values are recorded on the boring logs at the depth of sampling. Samples were sealed and returned to our laboratory for further examination and testing.

Groundwater observations are shown on the boring logs.

Upon completion of the borings, the boreholes were backfilled from the top and plugged at the surface.

3.0 LABORATORY TESTING

G&AI performs visual classification and any of a number of laboratory tests, as appropriate, to define pertinent engineering characteristics of the soils encountered. Tests are performed in general accordance with ASTM or other standards and the results included at the respective sample depths on the boring logs or separately tabulated, as appropriate, and included in Appendix C - Boring Logs and Laboratory Results. Laboratory tests and procedures routinely utilized, as appropriate, for geotechnical investigations are tabulated below.

Test Procedure	Description
ASTM D421	Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
ASTM D422	Standard Test Method for Particle-Size Analysis of Soils
ASTM D1140	Standard Test Methods for Amount of Material in Soils Finer than the No. 200 (75- μ m) Sieve
ASTM D2166	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
ASTM D2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D2217	Standard Practice for Wet Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
ASTM D2487	Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2850	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soil
ASTM D4220	Standard Practices for Preserving and Transporting Soil Samples
ASTM D4318	Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D4643	Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method
Manufacturer's Instructions	Soil Strength Determination Using a Torvane

4.0 SUBSURFACE CONDITIONS

4.1 Geology

Geologic Formation. Based on available surface geology maps and our experience, it appears this site is located in the Beaumont Formation, areas predominantly clay. A geologic atlas and formation description are provided in Appendix D - Geologic Atlas and Formation Description. Soils within the Beaumont Formation, areas predominantly clay, can generally be characterized as lower permeability clay with beds and lenses of fine sand and silt.

Geologic Faults. A review of the attached geologic map indicates the nearest geologic fault is about 2-miles north of the project site. A geologic fault study was beyond the scope of this investigation.

4.2 Soil

Stratigraphy. Descriptions of the various strata and their approximate depths and thickness per the Unified Soil Classification System (USCS) are provided on the boring logs included in Appendix C - Boring Logs and Laboratory Results. Terms and symbols used in the USCS are presented in Appendix E - Unified Soil Classification System. A brief summary of the stratigraphy indicated by the borings is provided below.

Generalized Subsurface Conditions at Proposed Building Location (Borings B-01 and B-02)		
Depth, feet bgs (unless noted)		Description
Top of Layer	Bottom of Layer	
0	4-inches	CONCRETE.
4-inches	Boring Termination (20)	<u>FAT CLAY</u> – Stiff to hard FAT CLAY (CH), hard FAT CLAY WITH SAND (CH), and stiff to hard SANDY FAT CLAY (CH).

Generalized Subsurface Conditions at Proposed Paving Locations (Borings B-03 to B-05)		
Depth, feet bgs (unless noted)		Description
Top of Layer	Bottom of Layer	
0	6-inches	CONCRETE.
6-inches	Boring Termination (20)	Stiff to hard FAT CLAY (CH).

Swell Potential. Atterberg (plastic and liquid) limits were performed on 6 shallow soil samples obtained between 0- and 8-feet bgs. The plasticity index of the samples was between 35 and 70 with an average of 52 indicating that the soils have a high potential for shrinking and swelling with changes in soil moisture content.

4.3 Groundwater

The borings were advanced using auger drilling and intermittent sampling methods in order to observe groundwater seepage levels. Groundwater levels encountered in the borings during this investigation are identified below.

Boring No.	Depth Groundwater Initially Encountered (feet, bgs)	Groundwater Depth after 15 Minutes (feet, bgs)
B-01 to B-05	Not Encountered	Not Encountered

Long-term monitoring of groundwater conditions via piezometers was not performed during this investigation and was beyond the scope of this study. Long-term monitoring can reveal groundwater levels materially different than those encountered during measurements taken while drilling the borings.

Future construction activities may alter the surface and subsurface drainage characteristics of this site. It is difficult to accurately predict the magnitude of subsurface water fluctuations that might occur based upon short-term observations. The groundwater level should be expected to fluctuate throughout the years with variations in precipitation.

5.0 ANALYSIS AND RECOMMENDATIONS

5.1 Seismic Site Classification

The seismic site classification is based on the 2006 International Building Code (IBC) and is a classification of the site based on the type of soils encountered at the site and their engineering properties. Per Table 1613.5.2 of the 2006 IBC, the seismic site classification for this site is C.

5.2 Potential Vertical Soil Movements

TxDOT Method Tex-124-E. Potential Vertical Rise (PVR) calculations were performed in general accordance with the Texas Department of Transportation (TxDOT) Method Tex-124-E. The Tex-124-E method is empirical and is based on the Atterberg limits and moisture

content of the subsurface soils. The calculated PVR is an empirical estimate of a soil's potential for swell based upon the soil's plasticity index, applied loading (due to structures or overburden), and antecedent moisture condition. The wetter a soil's antecedent moisture condition, the lower its calculated PVR will be for a given plasticity index and load. However, soil with a higher antecedent moisture content will be more susceptible to shrinkage due to drying. Maintaining a consistent moisture content in the soil is the key to minimizing both heave and shrinkage related structural problems.

Calculated PVR using TxDOT Method Tex-124-E. The PVR calculated using TxDOT Method Tex-124-E is about 3.5-inches assuming an average antecedent moisture condition. The calculated PVR is consistent with soil moisture conditions at the time this investigation was conducted.

Soil Moisture Confirmation Prior to Construction. The calculated PVR can vary considerably with prolonged wet or dry periods. We recommend the moisture content for the upper 10 feet of soils within the building pad be assessed for consistency with this report prior to construction if: (1) an extended period of time has elapsed between the performance of this investigation and construction of the foundation, or (2) unusually wet or dry weather is experienced between the performance of this investigation and construction of the foundation.

5.3 Construction Excavations

Sloped Excavations. All sloped short-term construction excavations on-site should be designed in accordance with Occupational Safety and Health Administration (OSHA) excavation standards. Borings from this investigation indicated that the soils may be classified per OSHA regulations as Type B from the ground surface to a depth of 10-feet bgs. Short-term construction excavations may be constructed with a maximum slope of 1:1, horizontal to vertical (H:V), to a depth of 10-feet bgs. If excavations are to be deeper than 10-feet, we should be contacted to evaluate the excavation. Recommendations provided herein are not valid for any long-term or permanent slopes on-site.

Shored Excavations. As an alternative to sloped excavations, vertical short-term construction excavations may be used in conjunction with trench boxes or other shoring systems. Shoring systems should be designed using an equivalent fluid weight of 95 pounds per cubic foot (pcf). Surcharge pressures at the ground surface due to dead and live loads should be added to the lateral earth pressures where they may occur. Lateral surcharge pressures should be assumed to act as a uniform pressure along the upper 10-feet of the excavation based on a lateral earth coefficient of 0.5. Surcharge loads set back behind the excavation at a horizontal distance equal to or greater than the excavation depth may be ignored. We recommend that no more than 200-feet of unshored excavation should be open at any one time to prevent the possibility of failure and excessive ground movement to

occur. We also recommend that unshored excavations do not remain open for a period of time longer than 24-hours.

Limitations. Recommendations provided herein assume there are no nearby structures or other improvements which might be detrimentally affected by the construction excavation. Before proceeding, we should be contacted to evaluate construction excavations with the potential to affect nearby structures or other improvements.

Excavation Monitoring. Excavations should be monitored to confirm site soil conditions consistent with those encountered in the borings drilled as part of this study. Discrepancies in soil conditions should be brought to the attention of G&AI for review and revision of recommendations, as appropriate.

5.4 Groundwater Control

Groundwater was not encountered during the subsurface investigation. If groundwater is encountered during excavation, dewatering to bring the groundwater below the bottom of excavations may be required. Dewatering could consist of standard sump pits and pumping procedures, which may be adequate to control seepage on a local basis during excavation. Supplemental dewatering will be required in areas where standard sump pits and pumping is not effective. Supplemental dewatering could include submersible pumps in slotted casings, well points, or eductors. The contractor should submit a groundwater control plan, prepared by a licensed engineer experienced in that type of work.

5.5 Earthwork

5.5.1 Site Preparation

In the area of improvements, all concrete, trees, stumps, brush, debris, septic tanks, abandoned structures, roots, vegetation, rubbish and any other undesirable matter should be removed and properly disposed. All vegetation should be removed and the exposed surface should be scarified to an additional depth of at least 6 inches. It is the intent of these recommendations to provide a loose surface with no features that would tend to prevent uniform compaction by the equipment to be used.

5.5.2 Proofroll

Building pad and paving subgrades should be proofrolled with a fully loaded tandem axle dump truck or similar pneumatic-tire equipment to locate areas of loose subgrade. In areas to be cut, the proofroll should be performed after the final grade is established. In areas to be filled, the proofroll should be performed prior to fill placement. Areas of loose or soft

subgrade encountered in the proofroll should be removed and replaced with engineered fill, moisture conditioned (dried or wetted, as needed) and compacted in place.

5.5.3 Grading and Drainage

Every attempt should be made to limit the extreme wetting or drying of the subsurface soils because swelling and shrinkage of these soils will result. Standard construction practices of providing good surface water drainage should be used. A positive slope of the ground away from any foundation should be provided. Ditches or swales should be provided to carry the run-off water both during and after construction. Stormwater runoff should be collected by gutters and downspouts and should discharge away from the buildings.

Root systems from trees and shrubs can draw a substantial amount of water from the clay soils at this site, causing the clays to dry and shrink. This could cause settlement beneath grade-supported slabs such as floors, walks and paving. Trees and large bushes should be located a distance equal to at least one-half their anticipated mature height away from grade slabs.

Lawn areas should be watered moderately, without allowing the clay soils to become too dry or too wet.

5.5.4 Wet Weather/Soft Subgrade

Soft and/or wet surface soils may be encountered during construction, especially following periods of wet weather. Wet or soft surface soils can present difficulties for compaction and other construction equipment. If specified compaction cannot be achieved due to soft or wet surface soils, one of the following corrective measures will be required:

1. Removal of the wet and/or soft soil and replacement with select fill,
2. Chemical treatment of the wet and/or soft soil to improve the subgrade stability, or
3. If allowed by the schedule, drying by natural means.

Chemical treatment is usually the most effective way to improve soft and/or wet surface soils. G&AI should be contacted for additional recommendations if chemical treatment is planned due to wet and/or soft soils.

5.5.5 Fill

Select Fill. Any fill placed in building pad areas should consist of select fill. Select fill should consist of soil with a liquid limit less than 35 and a Plasticity Index between 7 and 20. The select fill should be placed in loose lifts not exceeding 8-inches and should be compacted to at least 95 percent maximum dry density (per ASTM D-698) and at a moisture content between optimum and 4 percent above optimum moisture content. The subgrade to

receive select fill should be scarified to a depth of 6 inches and compacted to at least 95 percent maximum dry density (per ASTM D-698) and at a moisture content between optimum and 4 percent above optimum.

Lime-treated Native Clay Soil. Based on the laboratory testing conducted for this investigation, the native clay on-site soils will not meet specifications for select fill outlined in the section titled "Fill". As an alternative to importing select fill, the native clay soil may be blended with lime to reduce the plasticity index to meet select fill requirements. Based on our experience, we expect that it will require between 4- and 6-percent lime (by dry unit weight) to reduce the plasticity index of the native clay soils to select fill requirements. However, we recommend that lime series determination tests be performed to assess the amount of lime required.

General Fill. General fill may be placed in improved areas outside of building pad areas. General fill should consist of material approved by the Geotechnical Engineer with a liquid limit less than 50. General fill should be placed in loose lifts not exceeding 8-inches and should be uniformly compacted to a minimum of 95 percent maximum dry density (per ASTM D-698) and within ± 2 percent of the optimum moisture content.

Fill Restrictions. Select fill and general fill should consist of those materials meeting the requirements stated. Select fill and general fill should not contain material greater than 4-inches in any direction, debris, vegetation, waste material, environmentally contaminated material, or any other unsuitable material.

Unsuitable Materials. Materials considered unsuitable for use as select fill or general fill include low and high plasticity silt (ML and MH), silty clay (CL-ML), organic clay and silt (OH and OL) and highly organic soils such as peat (Pt). These soils may be used for site grading and restoration in unimproved areas as approved by the Geotechnical Engineer. Soil placed in unimproved areas should be placed in loose lifts not exceeding 10-inches and should be compacted to at least 92 percent maximum dry density (per ASTM D-698) and at a moisture content within ± 4 percentage points of optimum.

Cautionary Note. It is extremely important that select fill placed within building pads be properly characterized using one or more representative proctor samples. The use of a proctor sample which does not adequately represent the select fill being placed can lead to erroneous compaction (moisture and density) results which can significantly increase the potential for swelling of the select fill. The plasticity index of select fill soils placed during construction should be checked every lift to confirm conformance to the project specifications and consistency with the proctor being utilized.

5.5.6 Testing

Field compaction tests should be made by the Geotechnical Engineer or his representative. Compaction tests should be performed in each lift of the compacted material. We

recommend the following minimum soil compaction testing be performed: one test per lift per 2,500 square feet (SF) in the area of the building pad, one test per lift per 5,000 SF outside the building pad, and one test per lift per 100 linear feet of utility backfill. If the materials fail to meet the density or moisture content specified, the course should be reworked as necessary to obtain the specified compaction.

5.6 Buried Structures

Uplift. Buried structures are subjected to uplift forces caused by differential water levels adjacent to and within the structure. Soils with any appreciable silt or sand content will likely become saturated during periods of heavy rainfall and the effective static water level will be at the ground surface. For design purposes, we recommend the groundwater level be assumed at the ground surface. Resistance to uplift pressure is provided by soil skin friction and the dead weight of the structure. Skin friction should be neglected for the upper 3 feet of soil. A skin friction of 200 pounds per square foot (psf) may be used below a depth of 3 feet.

Lateral Pressure. Lateral pressures on buried structures due to soil loading can be determined using an equivalent fluid weight of 105 pounds per cubic foot (pcf). This includes hydrostatic pressure but does not include surcharge loads. The lateral load produced by a surcharge may be computed as 50 percent of the vertical surcharge pressure applied as a constant pressure over the full depth of the buried structure. Surcharge loads located a horizontal distance equal to or greater than the buried structure depth may be ignored.

Vertical Pressure. Vertical pressures on buried structures due to soil loading can be determined using an equivalent fluid weight of 130 pcf. This does not include surcharge loads. The vertical load produced by a surcharge may be computed as 100 percent of the vertical surcharge pressure applied as a constant pressure over the full width of the buried structure.

5.7 Retaining Structures

Applicability. G&AI was not made aware of any specific retaining structures in conjunction with this project. Recommendations provided in this section are applicable to structures 5-feet or less in height. Retaining structures in excess of 5-feet should be brought to the attention of G&AI for a more detailed assessment. *It is imperative that global stability be reviewed by G&AI on any retaining structure in excess of 5-feet in height.*

Lateral Pressure. Lateral pressures on retaining structures due to soil loading can be determined using an equivalent fluid weight of 105 pounds per cubic foot (pcf). This

includes hydrostatic pressure but does not include surcharge loads. The lateral load produced by a surcharge may be computed as 50 percent of the vertical surcharge pressure applied as a constant pressure over the full depth of the buried structure. Surcharge loads set back behind the retaining structure at a horizontal distance equal to or greater than the structure height may be ignored.

Lateral Resistance. Resistance to lateral loads may be provided by the soil adjacent to the structure. We recommend using an equivalent fluid weight of 200 pcf for lateral resistance.

Bearing Capacity. Assuming a minimum embedment depth of 24-inches, an allowable bearing capacity of 2,000 psf may be used for retaining structure footings (using a Factor of Safety of 3). An allowable coefficient of sliding friction of 0.3 (using a Factor of Safety of 2) between the concrete footings and underlying soil may be combined with the passive lateral resistance.

5.8 Foundation System

The following foundation types are appropriate for the building based on the geotechnical conditions encountered:

- Slab foundation,
- Shallow footings, or
- Underreamed drilled piers.

We have assumed that structural loads will be typical for the type and size of building proposed. Recommendations for the foundation types are presented below. Final determination of the foundation type to be utilized for this project should be made by the Structural Engineer based on loading, economic factors and risk tolerance.

We have also assumed that cuts of less than 1-foot will be required to bring the site to grade. In the event cuts in the building pad exceed 1-foot, we should be notified and allowed to review the design to assess the suitability of the foundation recommendations provided.

5.8.1 Foundation System - Slab

General. The proposed building can be supported on a reinforced ground-supported slab foundation. The slab foundation should be conventionally reinforced or post-tension reinforced. The slab foundation should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system to sustain the vertical soil movements expected at this site as described above. All grade beams and floor slabs should

be adequately reinforced with steel to minimize cracking as normal movements occur in the foundation soils.

Bearing Capacity and Foundation Depth. The slab should be designed using a net dead load plus sustained live load bearing pressure of 1,500 psf or a net total load pressure of 2,250 psf, whichever condition results in a larger bearing surface. These bearing pressures are based on a safety factor of 3 and 2, respectively, against shear failure of the foundation bearing soils. Grade beams should be founded a minimum of 18 inches below surrounding grade (supported on select fill) The bottom of the beam trenches should be free of any loose or soft material prior to the placement of the concrete.

PTI Recommendations Assuming Subgrade Treatment is Performed. A slab constructed on-grade will be subject to potential slab movements of about 3.5-inches based upon the information gathered during this investigation. Subgrade treatment (excavation of natural ground and replacement with select fill) should be performed to reduce the PVR to 1-inch or less. Subgrade treatment recommendations are provided in the section titled “Slab-on-Grade”. Assuming subgrade treatment suggestions are performed, the recommended foundation design parameters based on information published in the Post Tensioning Institute (PTI) Design of Post-Tensioned Slabs-on-Ground, 3rd Edition, are as follows:

	Edge Moisture Variation Distance (feet)		Differential Soil Movement (inches)	
	Center Lift	Edge Lift	Center Lift	Edge Lift
PTI 3rd Edition	8.5	4.7	0.6	0.4

IMPORTANT: The above foundation design parameters assume required subgrade treatment provided in the Slab-on-Grade section has been performed. The recommended foundation design parameters are applicable to climate controlled soil conditions only. These parameters are not applicable when non-climate related factors, such as vegetation, landscaping, trees, drainage, construction methods, land use, or other factors, may influence soil movement. G&AI should be contacted to evaluate the effect of non-climate related factors.

Deflection Analysis. Slab deflections should be analyzed per recommendations provided in Section 5.9.

5.8.2 Foundation System - Shallow Footings

Bearing Capacity and Foundation Depth. Provided that the recommendations in the section titled “Slab-on-Grade” are followed, shallow strip and spread footing foundations can be used for support of the proposed structure bearing on select fill at a depth of 2-feet below surrounding grade. Continuous strip footings can be proportioned using a net dead load plus sustained live load bearing pressure of 2,000 psf or a net total load bearing pressure of 3,000

psf, whichever condition results in a larger bearing surface. Individual spread footings can be proportioned using a net dead load plus sustained live load bearing pressure of 2,600 psf or a net total load bearing pressure of 3,900 psf, whichever condition results in a larger bearing surface. These bearing pressures are based on a safety factor of 3 and 2, respectively. Individual spread footings should be at least 30 inches wide and continuous strip footing foundations should be at least 16 inches wide.

Settlement. Settlement of footing foundations is influenced by a number of factors, including: load (pressure), soil consolidation properties, depth to groundwater, geometry (width and length), depth, spacing, and quality of construction. Although a detailed settlement analysis is beyond the scope of this study, settlement for foundations constructed as described above should be about 1 inch or less.

Lateral Resistance. Resistance to lateral loads may be provided by the soil adjacent to the footings. We recommend using an equivalent fluid weight of 200 pcf for lateral resistance. An allowable coefficient of sliding friction of 0.3 (using a Factor of Safety of 2) between the concrete footings and underlying soil may be combined with the passive resistance.

Construction and Observation. The geotechnical engineer should monitor foundation construction to verify conditions are as anticipated and that the materials encountered are suitable for support of foundations. Soft or unsuitable soils encountered at the foundation bearing level should be removed to expose suitable, firm soil. Foundation excavations should be dry and free of loose material. Excavations for foundations should be filled with concrete before the end of the workday or sooner if necessary to prevent deterioration of the bearing surface. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface. If more than 24 hours of exposure of the bearing surface is anticipated in the excavation, a "mud slab" should be used to protect the bearing surfaces. If a mud slab is used, the foundation excavations should initially be over-excavated by approximately 4 inches and a lean concrete mud slab of approximately 4 inches in thickness should be placed in the bottom of the excavation immediately following exposure of the bearing surface by excavation. The mud slab will protect the bearing surface, maintain more uniform moisture in the subgrade, facilitate dewatering of excavations if required and provide a working surface for the placement of formwork and reinforcing steel.

5.8.3 Foundation System - Underreamed Drilled Piers

Bearing Capacity and Foundation Depth. Drilled pier foundations (auger-excavated, underreamed, steel reinforced, cast-in-place concrete piers) bearing in native soil may be utilized at this site for the proposed structure. We recommend that underreamed piers should be founded at a depth of 10- to 12-feet beneath the existing grade. The piers may be proportioned using a net dead load plus sustained live load bearing pressure of 4,000 psf or a net total load pressure of 6,000 psf, whichever condition results in a larger bearing surface.

These bearing pressures are based on a safety factor of 3 and 2, respectively, against shear failure of the foundation bearing soils.

Settlement. Foundation settlement for drilled piers constructed as described above should be less than 1 inch.

Uplift and Lateral Capacity. Because of the potential for the upper two feet of the soil to shrink and pull away from drilled piers during dry periods, we recommend soil resistance to uplift and lateral loads on drilled piers be ignored in the upper 2-feet of the soil profile. For resistance of uplift and lateral loads on drilled piers, we recommend the following parameters (Factor of Safety = 3 for cohesion).

Depth (feet)	Soil Type	Effective Soil Unit Weight (pcf) ⁽¹⁾	Allowable Cohesion, c (psf)	Angle of Internal Friction, Φ (degrees)	Strain at ½ Peak Strength, ε ₅₀	Modulus of Subgrade Reaction, k (for lateral loads) (pci)
0 – 2	Clay	120	0	0	NA	NA
2 – 12	Clay	120	800	0	0.007	200
(1) – Effective soil unit weight based on assumed groundwater depth greater than 12-feet. pcf = pounds per cubic foot; psf = pounds per square foot; pci = pounds per cubic inch						

The uplift resistance provided by an underreamed drilled pier is the sum of resistance provided by the shear strength of the soil, the weight of the soil above the bell and the weight of the drilled pier itself. The following equation may be used to calculate the allowable uplift resistance:

$$F_a = cN_u A_u + \frac{W_s}{FS_1} + \frac{W_c}{FS_2}$$

Where:

F_a = allowable uplift resistance, lbs

c = allowable cohesion, psf

N_u = bearing capacity factor = $\frac{3.5D_b}{B_b} \leq 9$

D_b = depth to base of bell, ft

B_b = diameter of the bell, ft

A_u = projected area of bell, ft² = $\frac{\pi(B_b^2 - B_s^2)}{4}$

B_s = diameter of the drilled pier shaft, ft

W_s = weight of soil above bell, lbs

W_c = weight of drilled pier, lbs

FS_1, FS_2 = safety factors

A safety factor of 3 has been applied to the allowable cohesion value and is appropriate for sustained loading conditions. However, the allowable cohesion values may be increased by 50 percent, resulting in a safety factor of 2, for transient loading conditions. We recommend a soil weight safety factor, FS_1 , of 1.2 and a drilled pier weight safety factor, FS_2 , of 1.1.

The uplift force on the piers due to swelling of the active clays can be approximated by assuming a uniform uplift pressure of 1,000 psf acting over the perimeter of the shaft to a depth of 8 feet. The shafts should contain sufficient full length reinforcing steel to resist uplift forces.

The piers should be provided with an underream diameter to shaft diameter ratio not less than 2 to 1 and not greater than 3 to 1. For uplift considerations, piers should not be spaced closer than two underream diameters (edge to edge) based on the diameter of the larger underream. Closer pier spacings may result in reduced uplift capacity. We should be contacted to review closer pier spacings on a case-by-case basis.

Construction and Observation. The construction of all piers should be observed as a means to verify compliance with design assumptions and to verify:

1. the bearing stratum;
2. underream size;
3. the removal of all smear zones and cuttings;
4. that groundwater seepage, when encountered, is correctly handled; and
5. that the shafts are vertical (within acceptable tolerance).

Groundwater was not encountered the boring locations during the subsurface investigation. However, groundwater may be encountered during pier excavation and the risk of groundwater seepage is increased during or after periods of precipitation. Submersible pumps may be capable of controlling seepage in the pier excavation to allow for concrete placement.

Drilled pier foundations should be constructed in accordance with the requirements of TxDOT Item 416 (standard specification for construction of drilled pier foundations). This specification includes requirements for construction using casing or the slurry displacement method, as appropriate. We should be contacted for further evaluation and recommendations if soils other than those anticipated to be encountered at the design foundation bearing level, or if groundwater seepage and/or underream collapse occurs.

Concrete should be placed in the shafts immediately after excavation to reduce the risk of significant groundwater seepage, deterioration of the foundation-bearing surface and underream collapse. In no event should a pier excavation be allowed to remain open for more than 8 hours. Concrete should have a slump of 5 to 7 inches and should not be allowed to strike the shaft sidewall or steel reinforcement during placement.

5.9 Slab-on-Grade

Potential Vertical Slab Movements. Based on the information gathered during this investigation, a slab constructed on-grade will be subject to potential vertical slab movements of about 3.5-inches.

Subgrade Treatment. To reduce the PVR to 1-inch or less, we recommend that the upper 5-feet of the building pad subgrade consist of select fill. In areas where less than 5-feet of fill is required to bring the building pad to grade, we recommend that the existing soils be excavated to a depth of 5-feet below the top of the finished building pad subgrade and replaced with select fill. Subgrade treatment should extend at least 3-feet horizontally beyond the perimeter of the building.

Subgrade Treatment at Exterior Doorways. Subgrade treatment should extend beneath sidewalk areas that abut exterior doorways to the building. Failure to perform subgrade treatment in these areas can increase the probability of differential heaving between exterior sidewalks and doorways, resulting in exterior doors that won't or have difficulty opening outward due to "sticking" caused by heaving sidewalk slabs.

Subgrade Moisture. The slab subgrade is prone to drying after being exposed and should be kept moist prior to slab placement.

Moisture Barrier. A moisture barrier should be used beneath the slab foundation in areas where floor coverings will be utilized (such as, but not limited to, wood flooring, tile, linoleum and carpeting).

Slab Deflection Analysis. We recommend that slab deflections be analyzed using a coefficient of subgrade reaction, k , of 125 pounds per cubic inch (pci) for the subgrade soils. We have assumed that the slab subgrade will not be chemically stabilized.

5.10 Structural Slab

In lieu of the construction of a slab-on-grade which requires select fill to reduce the PVR to 1-inch or less, a structural floor slab supported on piers installed as previously described may be utilized. If a structural floor slab is to be installed, a minimum 7-inch void space between the bottom of the slab or grade beams and the underlying ground surface should be maintained.

5.11 Flooring

Concrete floors, even with adequate curing time, can present an unacceptable moisture condition by allowing excessive amounts of moisture vapor to pass through to the surface. Moisture moves through concrete in a partially adsorbed or condensed state by diffusion, not simply as unbound, free water vapor or liquid. The rate of moisture transmission depends on the degree of saturation, which is a function of the relative humidity on each side of the concrete. Therefore, the driving force for water vapor movement through a slab is the relative humidity differential through the slab's depth, not simply the vapor pressure differential.

When a floor covering is placed on top of a slab, it restricts evaporation from the top surface of the slab; moisture within the slab then distributes itself to achieve equilibrium due to temperature and chemical interactions from the top to the bottom of the slab. In the long run, adhesive and flooring are then exposed to the equilibrium moisture level at the top of the slab.

Concrete is naturally alkaline. Some additives promoting shortened cure time or easier/longer concrete workability may also increase concrete's natural alkalinity. Floor failure risks are increased if moisture movement transports this alkali to be in contact with the floor/adhesive system.

Typically, concrete floors should be tested for moisture emission rate by utilizing an anhydrous calcium chloride moisture test kit available from installation supplies and accessories distributors. This quantitative method is precise and must be conducted carefully with strict attention to the test kit manufacturer's detailed instructions. Moisture emission rate is expressed in lbs/1000 square feet/24 hours. Because the calcium chloride test for emission rate requires 3 days to conduct, proper installation planning is a must. As a general guideline, an emission rate of 3 lbs or less is acceptable for most flooring. In the range from 3 to 5 lbs., flooring with porous backings can usually be installed successfully; however, the risk of moisture related problems increases. Since some floor covering products are less tolerant of moisture than others, individual manufacturers should be consulted to determine the acceptable emission rate for specific products.

For adhesive backed floors, alkalinity testing should typically be performed prior to installation. A pH range of 5-9 is generally acceptable. Readings above 9 typically require corrective measures prior to floor installation. The adhesive manufacturer should always be consulted for recommended testing and corrective procedures.

Prior to installation of new flooring, slab moisture and alkalinity testing should be performed in accordance with manufacturer's instructions. Test results should be reviewed and approved by the flooring and/or adhesive manufacturer prior to placement of the floor.

Moisture and/or alkali levels may influence the type of flooring which can be used. Different types of flooring have different tolerances to moisture and/or alkali levels.

If slab moisture and/or alkalinity levels are too high for the manufacturer approved placement of the flooring selected, remedial action will be required. There are a number of products available to treat for alkalis and/or prepare and seal the concrete slab surface to reduce moisture migration rates to acceptable levels. Due to the myriad of flooring types available, alkali treatment and/or floor sealants should be reviewed and approved by the manufacturer prior to floor placement.

5.12 Pavement

Recommendations for rigid pavement and preparation of the pavement subgrade are provided in the following sections. A traffic study indicating the number and type of vehicles on which to base the pavement design was not provided. Therefore, our recommendations are based upon our experience with similar projects assuming normal vehicular loading. Any unusual loading conditions should be brought to our attention prior to finalizing the pavement design so that we may assess and modify our recommendations as necessary.

5.12.1 Rigid Pavement

Portland cement concrete (PCC) with a minimum 28-day compressive strength of 3,500 pounds per square inch (psi) should be utilized for rigid pavement. Grade 60 reinforcing steel should be utilized in the transverse and longitudinal directions. The following pavement thicknesses and reinforcing are recommended:

Paving Use	Thickness (inches)	Reinforcing
Parking Areas for Automobiles and Light Trucks	5	No. 3 bars spaced on 24-inch intervals or 12 x 12 – W5 x W5 welded wire fabric
Drive Lanes and Areas Subjected to Light to Medium Trucks	6	No. 3 bars spaced on 18-inch intervals or 12 x 12 – W6 x W6 welded wire fabric
Areas Receiving Heavy Trucks and Dumpsters	7	No. 3 bars spaced on 18-inch intervals or 12 x 12 – W6.5 x W6.5 welded wire fabric

Contraction joints should be spaced at about 30 times the pavement thickness up to a maximum of 15 feet in any direction. Saw cut control joints should be cut within 6 to 12 hours of concrete placement. Expansion joints should be spaced a maximum of 60-feet in any direction and should be placed where the pavement abuts any structure. Dowels should

have a diameter equal to $\frac{1}{8}$ the slab thickness, be spaced on 12-inch intervals, and be embedded at least 9-inches. Where not specified herein, concrete pavement should comply with Harris County Standard Specifications, Item 360, "Concrete Pavement", or equivalent.

5.12.2 Pavement Subgrade

Fat clay is expected to be encountered or exposed at pavement subgrade. The pavement subgrade should be placed in loose lifts not exceeding 8-inches and should be uniformly compacted to a minimum of 95 percent maximum dry density (per ASTM D-698) and within ± 2 percent of the optimum moisture content. We recommend the subgrade be stabilized using the following:

Reagent	Application Rate (pounds per square yard)	Application Depth (inches)
Lime	27	6

Lime stabilization should be performed in accordance with Harris County Standard Specifications, Item 220, "Lime Stabilized Subgrade", or equivalent.

6.0 CONSTRUCTION OBSERVATIONS

By necessity, geotechnical engineering design recommendations are based on a limited amount of information about subsurface conditions. In the analysis, the geotechnical engineer must assume subsurface conditions are similar to those encountered in the borings. Anomalies in subsurface conditions are often revealed during construction. G&AI should be retained to observe earthwork and foundation installation and perform materials evaluation and testing 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, final depth of undercut of expansive soils for non-expansive earth fill pads 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 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.

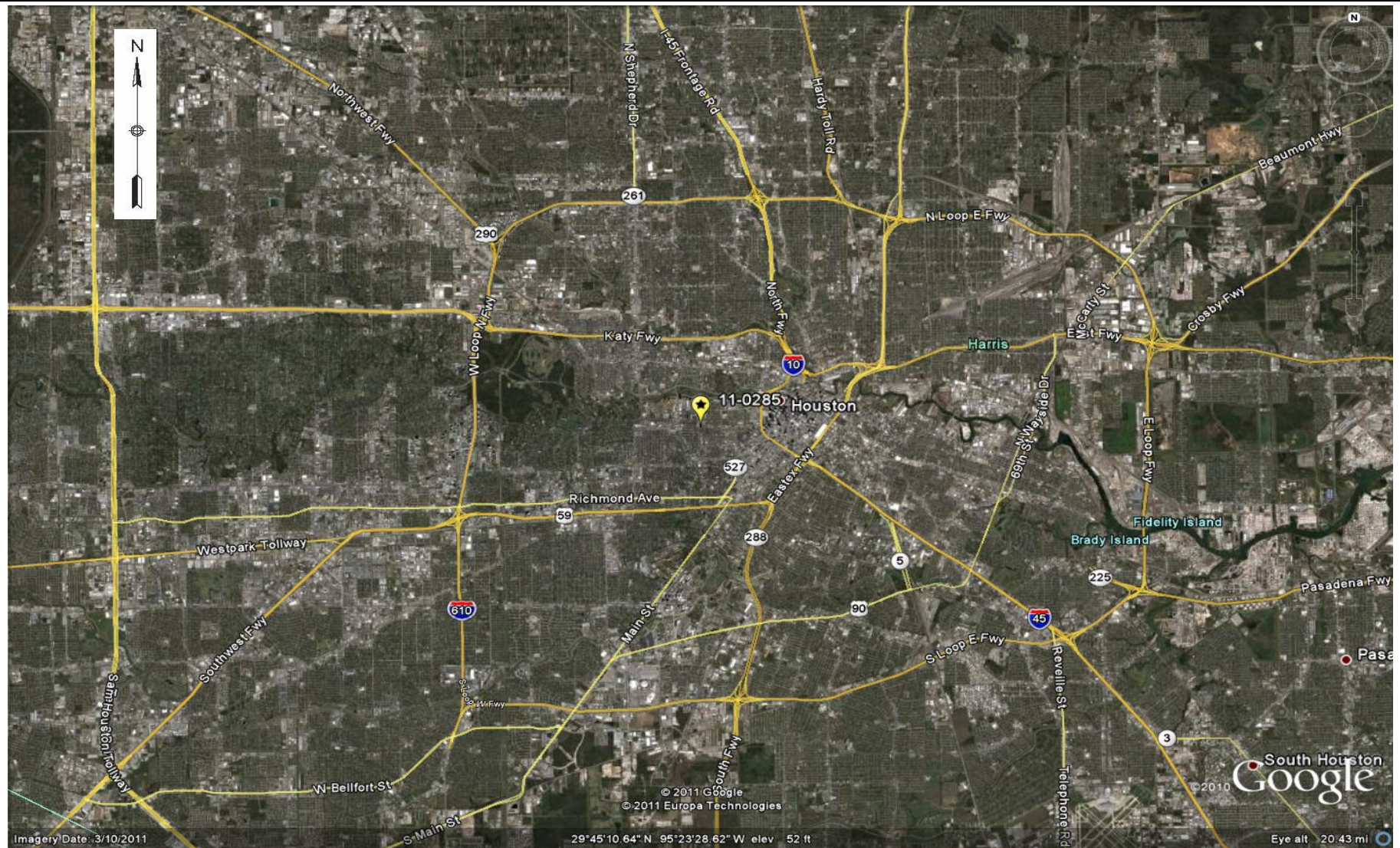
7.0 GENERAL COMMENTS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the field investigation and 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 should 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.

G&AI, Inc. should 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.

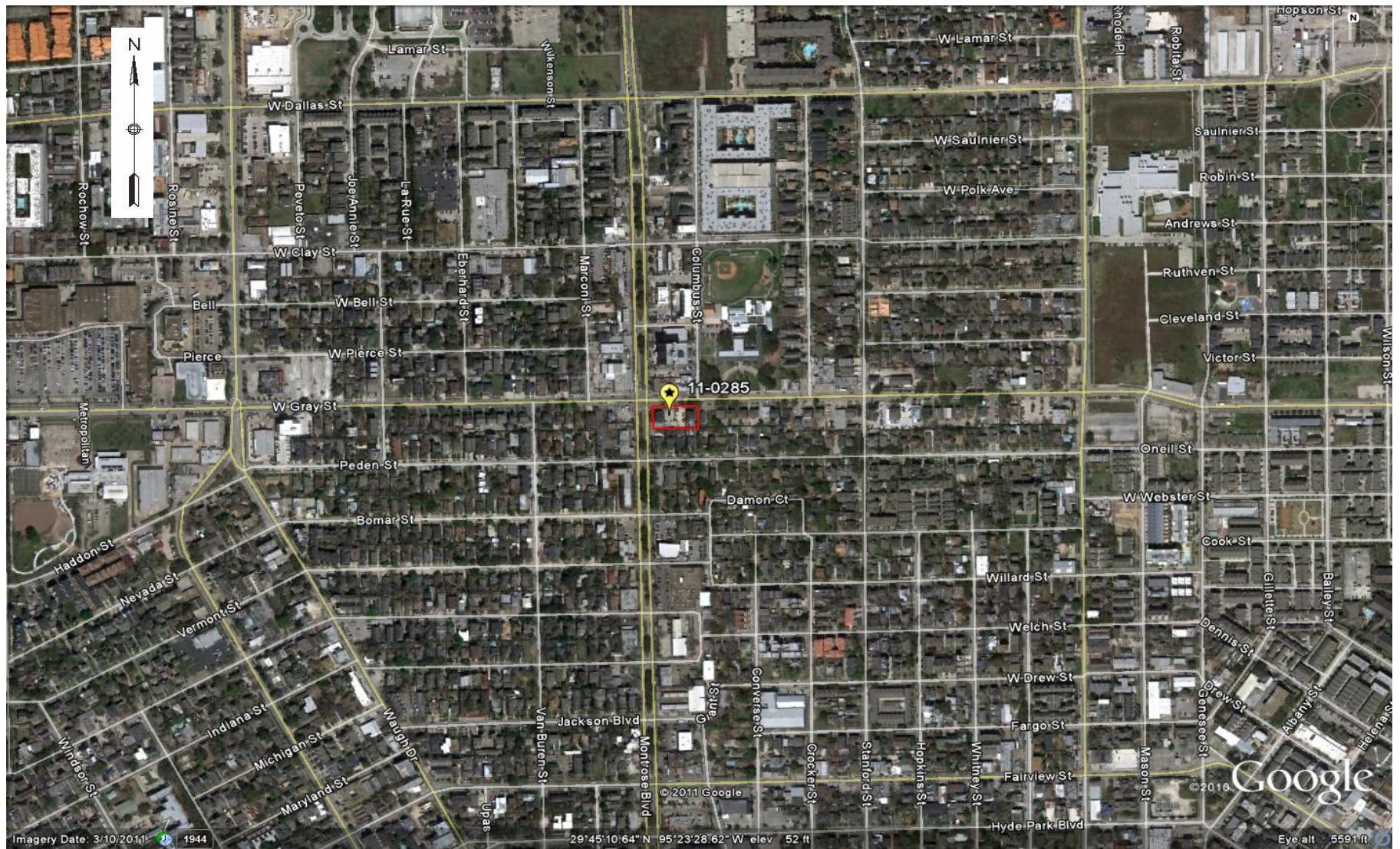
This report has been prepared for the exclusive use of the Client and their designated agents for specific application to design of this project. We have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No warranty, expressed or implied, is made or intended.

Appendix A - Project Location Diagrams



PROJECT LOCATION DIAGRAM - GENERAL
Shaw's Jewelry
Montrose Boulevard and West Gray Street, Houston, Texas

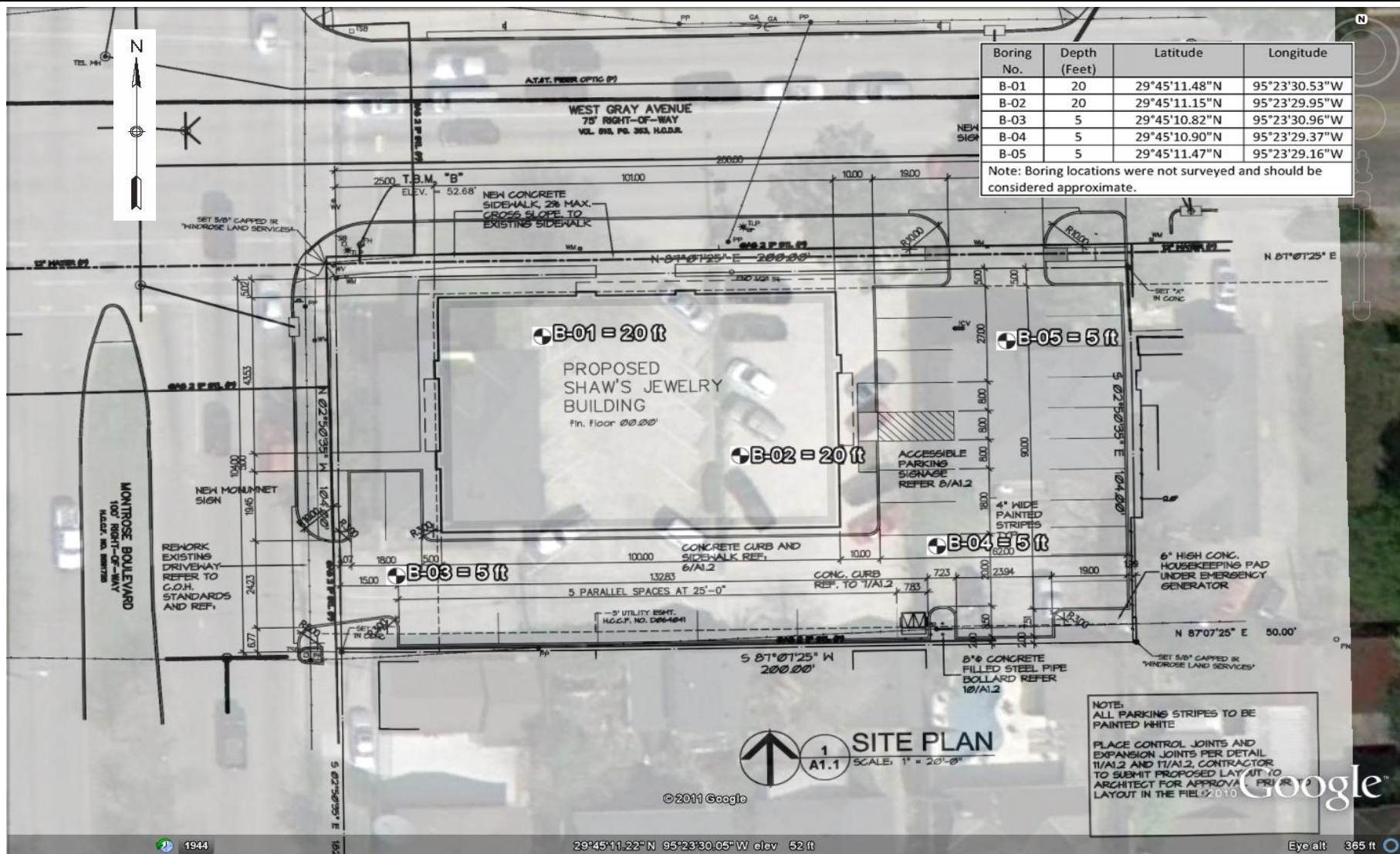
Project No. 11-0285
Date - 9/20/2011



PROJECT LOCATION DIAGRAM - LOCAL
Shaw's Jewelry
Montrose Boulevard and West Gray Street, Houston, Texas

Project No. 11-0285
Date - 9/20/2011

Appendix B - Boring Location Diagram



BORING LOCATION DIAGRAM
Shaw's Jewelry
Montrose Boulevard and West Gray Street, Houston, Texas

Project No. 11-0285
Date - 9/20/2011

Appendix C - Boring Logs and Laboratory Results



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BORING NUMBER B-01

PAGE 1 OF 1

CLIENT Hermes Architects

PROJECT NAME Shaw's Jewelry

PROJECT NUMBER 11-0285

PROJECT LOCATION Montrose Blvd. and West Gray St., Houston, Texas

DATE STARTED 9/26/11

COMPLETED 9/26/11

GROUND ELEVATION _____

HOLE SIZE _____

DRILLING CONTRACTOR Jet-Core

GROUND WATER LEVELS:

DRILLING METHOD Auger

AT TIME OF DRILLING Not encountered

LOGGED BY Shawn

CHECKED BY JSS

AT END OF DRILLING ---

NOTES _____

AFTER DRILLING ---

REV. GEO LOG W TOR & UC 11-0285 SHAW'S JEWELRY.GPJ GINT US 29 JAN 07.GDT 10/10/11

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
												LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		CONCRETE - 4-inches.													
		FAT CLAY WITH SAND (CH) - Hard, brown, with iron nodules.	ST			4.5+	2.3+				11				
		Gray and brown below 2 feet.	ST			4.5+	2.3+				14	50	15	35	83
5		FAT CLAY (CH) - Hard, tan and gray, with calcareous nodules and iron nodules.	ST			4.5+	2.3+				19				
		Reddish brown and gray below 6 feet.	ST			4.5+	2.3+				16	70	18	52	95
		Tan below 8 feet.	ST			4.5+	2.3+				12				
10															
		SANDY FAT CLAY (CH) - Stiff to hard, reddish brown and gray.	ST			4.5+	2.3+	11.5	6	121	12				
15															
20			SS		7-6-13 (19)						28				

Bottom of hole at 20.0 feet.



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BORING NUMBER B-02

PAGE 1 OF 1

CLIENT <u>Hermes Architects</u>	PROJECT NAME <u>Shaw's Jewelry</u>
PROJECT NUMBER <u>11-0285</u>	PROJECT LOCATION <u>Montrose Blvd. and West Gray St., Houston, Texas</u>
DATE STARTED <u>9/26/11</u> COMPLETED <u>9/26/11</u>	GROUND ELEVATION _____ HOLE SIZE _____
DRILLING CONTRACTOR <u>Jet-Core</u>	GROUND WATER LEVELS:
DRILLING METHOD <u>Auger</u>	AT TIME OF DRILLING <u>Not encountered</u>
LOGGED BY <u>Shawn</u> CHECKED BY <u>JSS</u>	AT END OF DRILLING <u>---</u>
NOTES _____	AFTER DRILLING <u>---</u>

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
												LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		CONCRETE - 4-inches.													
		FAT CLAY (CH) - Stiff to hard, brown, with iron nodules.	ST			1.75	1.3				26				
		Gray below 2 feet.	ST			2.0	1.4				26	84	15	70	88
5		Tan and reddish brown, with calcareous nodules below 4 feet.	ST			2.5	1.3				30				
		Reddish brown and gray below 8 feet.	ST			2.5	1.5				24	71	14	57	88
10			ST			2.5	1.5				31				
		SANDY FAT CLAY (CH) - Stiff to hard, reddish brown and gray.	ST			4.5+	2.3+	4.2	6	122	15				
15															
20			SS		10-12-10 (22)						27				

Bottom of hole at 20.0 feet.

REV. GEO LOG W TOR & UC 11-0285 SHAW'S JEWELRY.GPJ GINT US 29 JAN 07.GDT 10/10/11



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BORING NUMBER B-03

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CLIENT <u>Hermes Architects</u>	PROJECT NAME <u>Shaw's Jewelry</u>
PROJECT NUMBER <u>11-0285</u>	PROJECT LOCATION <u>Montrose Blvd. and West Gray St., Houston, Texas</u>
DATE STARTED <u>9/26/11</u> COMPLETED <u>9/26/11</u>	GROUND ELEVATION _____ HOLE SIZE _____
DRILLING CONTRACTOR <u>Jet-Core</u>	GROUND WATER LEVELS:
DRILLING METHOD <u>Auger</u>	AT TIME OF DRILLING <u>Not encountered</u>
LOGGED BY <u>Shawn</u> CHECKED BY <u>JSS</u>	AT END OF DRILLING <u>---</u>
NOTES _____	AFTER DRILLING <u>---</u>

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
												LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		CONCRETE - 6-inches.													
		FAT CLAY (CH) - Stiff to hard, brown, with iron nodules.	ST			2.5	1.3				25	59	15	44	86
			ST			4.5+	2.3+				14				
5			ST			4.5+	2.3+				16				
		Bottom of hole at 5.0 feet.													



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BORING NUMBER B-04

PAGE 1 OF 1

CLIENT Hermes Architects

PROJECT NAME Shaw's Jewelry

PROJECT NUMBER 11-0285

PROJECT LOCATION Montrose Blvd. and West Gray St., Houston, Texas

DATE STARTED 9/26/11

COMPLETED 9/26/11

GROUND ELEVATION _____

HOLE SIZE _____

DRILLING CONTRACTOR Jet-Core

GROUND WATER LEVELS:

DRILLING METHOD Auger

AT TIME OF DRILLING Not encountered

LOGGED BY Shawn

CHECKED BY JSS

AT END OF DRILLING ---

NOTES _____

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
												LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		CONCRETE - 6-inches.													
		FAT CLAY (CH) - Hard, brown, with iron nodules.	ST			4.5+	2.3+				17				
		Brown and gray below 2 feet.	ST			4.5+	2.3+				24				
		Gray below 4 feet.	ST			4.5+	2.3+				21				
5		Bottom of hole at 5.0 feet.													



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BORING NUMBER B-05

PAGE 1 OF 1

CLIENT Hermes Architects

PROJECT NAME Shaw's Jewelry

PROJECT NUMBER 11-0285

PROJECT LOCATION Montrose Blvd. and West Gray St., Houston, Texas

DATE STARTED 9/26/11

COMPLETED 9/26/11

GROUND ELEVATION _____

HOLE SIZE _____

DRILLING CONTRACTOR Jet-Core

GROUND WATER LEVELS:

DRILLING METHOD Auger

AT TIME OF DRILLING Not encountered

LOGGED BY Shawn

CHECKED BY JSS

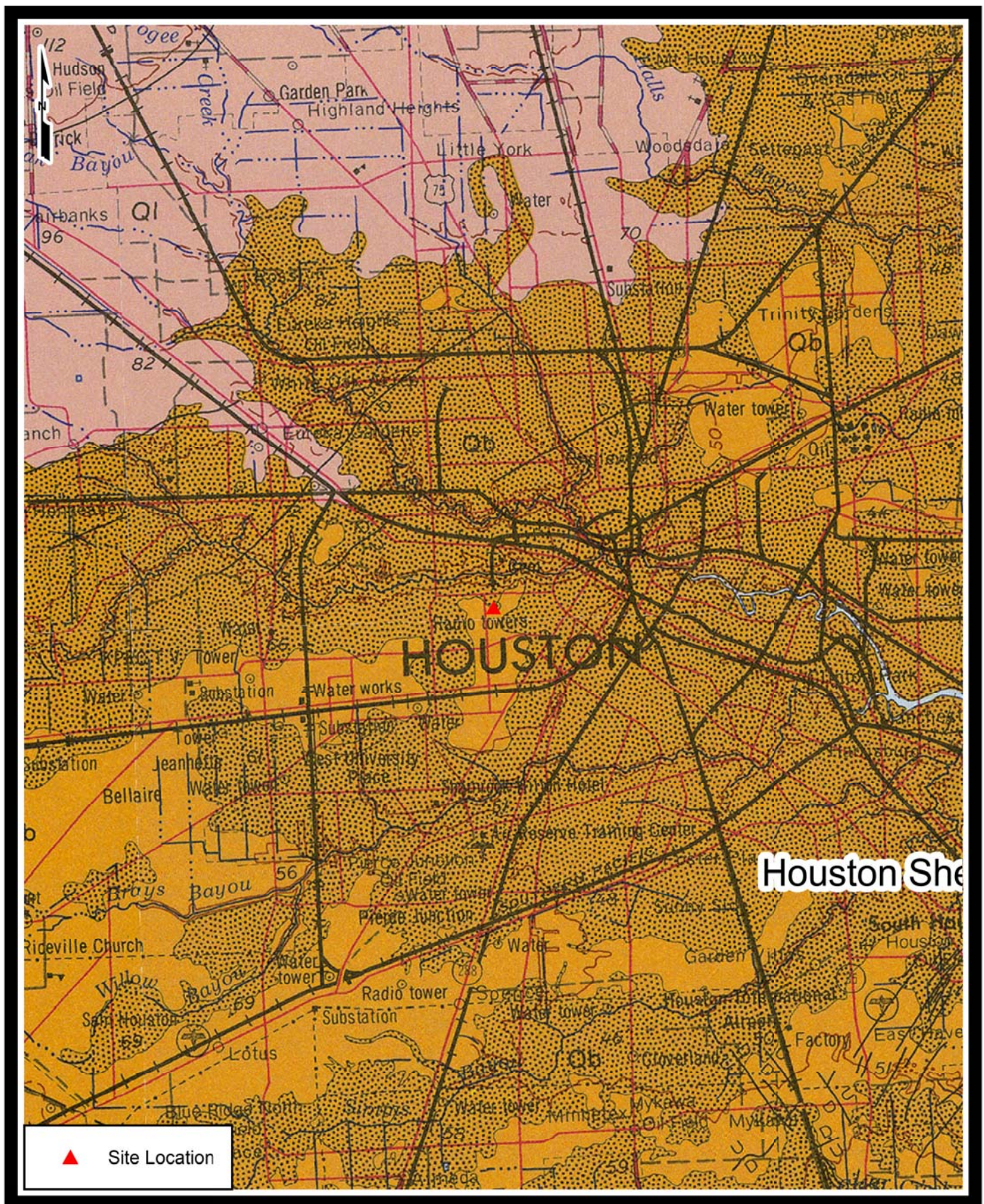
AT END OF DRILLING ---

NOTES _____

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
												LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		CONCRETE - 6-inches.													
		FAT CLAY (CH) - Hard, brown.	ST			4.5+	2.3+				20				
			ST			4.5+	2.3+				16	67	15	51	87
5			ST			4.5+	2.3+				18				
		Bottom of hole at 5.0 feet.													

Appendix D - Geologic Atlas and Formation Description



0 1.25 2.5 5 Miles

11-0285

Geologic Atlas of Texas





Mineral Resources On-Line Spatial Data

[Mineral Resources](#) > [Online Spatial Data](#) > [Geology](#) > [by state](#) > [Texas](#)

Beaumont Formation, areas predominantly clay

Beaumont Formation, areas predominantly clay

State [Texas](#)

Name Beaumont Formation, areas predominantly clay

Geologic age Phanerozoic | Cenozoic | Quaternary | Pleistocene-Late

Original map label Qbc

Comments On McAllen-Brownsville Sheet (1976) dominantly clay and mud of low permeability. (from Moore and Wermund, 1993a, 1993b): Light- to dark-gray and bluish- to greenish-gray clay and silt, intermixed and interbedded; contains beds and lenses of fine sand, decayed organic matter, and many buried organic-rich, oxidized soil(?) zones that contain calcareous and ferruginous nodules. Very lt. gray to v. lt. yell-gray sediment cemented by calcium carbonate present in varied forms, veins, laminar zones, burrows, root casts, nodules. Locally, small gypsum crystals present. Includes plastic and compressible clay and mud deposited in flood basins, coastal lakes, and former stream channels on a deltaic plain. Disconformably overlies Lissie Fm. Thickness 5-10 m along north edge of outcrop; thickens southward in subsurface to more than 100 m.

Primary rock type [clay or mud](#)

Secondary rock type [silt](#)

Other rock types

Map references Bureau of Economic Geology, 1992, Geologic Map of Texas: University of Texas at Austin, Virgil E. Barnes, project supervisor, Hartmann, B.M. and Scranton, D.F., cartography, scale 1:500,000

Unit references Bureau of Economic Geology, 1975, Corpus Christi Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Moore, D.W. and Wermund, E.G., Jr., 1993a, Quaternary geologic map of the Austin 4 x 6 degree quadrangle, United States: U.S. Geological Survey Miscellaneous Investigations Series Map I-1420 (NH-14), scale 1:1,000,000.

[[http://pubs.er.usgs.gov/publication/i1420\(NH14\)](http://pubs.er.usgs.gov/publication/i1420(NH14))]

Bureau of Economic Geology, 1976, McAllen-Brownsville Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Bureau of Economic Geology, 1975, Beeville-Bay City Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Bureau of Economic Geology, 1982, Houston Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Geographic coverage [Aransas](#) - [Austin](#) - [Bee](#) - [Brazoria](#) - [Calhoun](#) - [Cameron](#) - [Chambers](#) - [Colorado](#) - [Fort Bend](#) - [Galveston](#) - [Hardin](#) - [Harris](#) - [Hidalgo](#) - [Jackson](#) - [Jasper](#) - [Jefferson](#) - [Jim Wells](#) - [Kenedy](#) - [Kleberg](#) - [Liberty](#) - [Live Oak](#) - [Matagorda](#) - [Newton](#) - [Nueces](#) - [Orange](#) - [Refugio](#) - [San Patricio](#) - [Victoria](#) - [Waller](#) - [Wharton](#) - [Willacy](#)
















U.S. Department of the Interior | U.S. Geological Survey

URL: <http://tin.er.usgs.gov/geology/state/ak/akgeo-unit.php?unit=>

Page Contact Information: [Peter Schweitzer](#)

Appendix E - Unified Soil Classification System

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART			LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)				
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)			
		GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3
		GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines	
	Gravels with fines (More than 12% fines)			
		GM	Silty gravels, gravel-sand-silt mixtures	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
		GC	Clayey gravels, gravel-sand-clay mixtures	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)			
		SW	Well-graded sands, gravelly sands, little or no fines	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3
		SP	Poorly graded sands, gravelly sands, little or no fines	
	Sands with fines (More than 12% fines)			
		SM	Silty sands, sand-silt mixtures	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
		SC	Clayey sands, sand-clay mixtures	
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)				
SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity	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
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
SILTS AND CLAYS Liquid limit 50% or greater		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	

LABORATORY CLASSIFICATION CRITERIA	
GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3
GP	Not meeting all gradation requirements for GW
GM	Atterberg limits below "A" line or P.I. less than 4
GC	Atterberg limits above "A" line with P.I. greater than 7
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3
SP	Not meeting all gradation requirements for GW
SM	Atterberg limits below "A" line or P.I. less than 4
SC	Atterberg limits above "A" line with P.I. greater than 7

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

PLASTICITY CHART	
PLASTICITY INDEX (PI) (%)	LIQUID LIMIT (LL) (%)
60	0
50	10
40	20
30	30
20	40
10	50
0	60
0	70
0	80
0	90
0	100

CH

CL

MH&OH

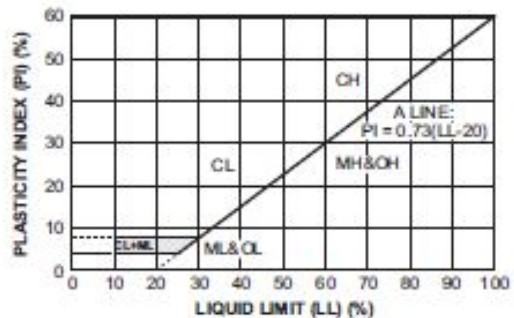
ML&OL

CL&ML

A LINE:
PI = 0.73(LL - 20)

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

PLASTICITY CHART



TERMS DESCRIBING SOIL CONSISTENCY

Fine Grained Soils			Coarse Grained Soils	
Description	Penetrometer Reading (tsf)	Penetration Resistance (blows/ft)	Description	Relative Density
Soft	0.0 to 1.0	0 to 4	Very Loose	0 to 20%
Firm	1.0 to 1.5	4 to 10	Loose	20 to 40%
Stiff	1.5 to 3.0	10 to 30	Medium Dense	40 to 70%
Very Stiff	3.0 to 4.5	30 to 50	Dense	70 to 90%
Hard	4.5+	Over 50	Very Dense	90 to 100%