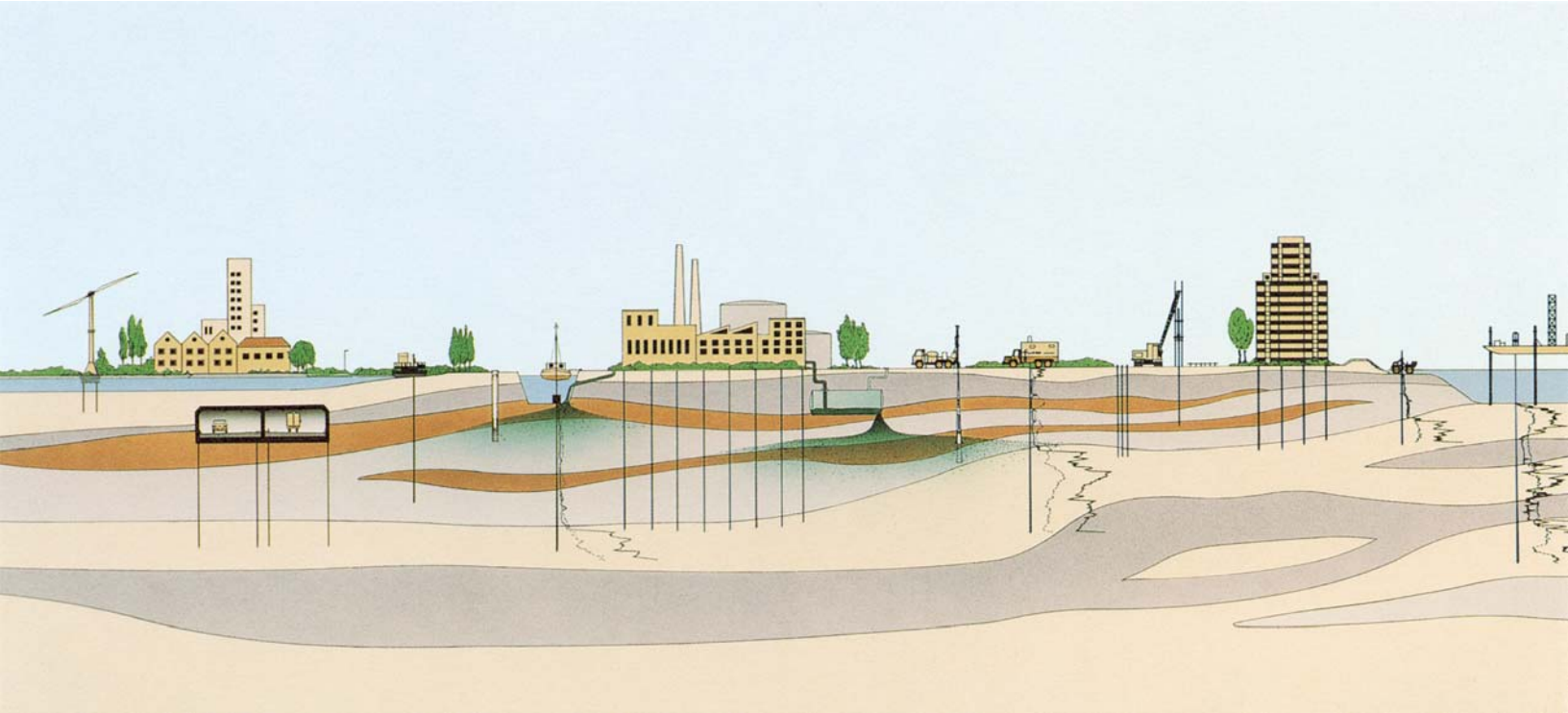
FUGRO CONSULTANTS, INC.

GEOTECHNICAL STUDY
PROPOSED PARKING GARAGE
BEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

HOK
HOUSTON, TEXAS



FUGRO CONSULTANTS, INC.

6100 Hillcroft (77081)
P.O. Box 740010
Houston, Texas 77274
Tel: (713) 369-5400
Fax: (713) 369-5518

Report No. 04.12100060
February 11, 2011

HOK
2800 Post Oak Boulevard, Suite 3700
Houston, Texas 77056

Attention: Mr. John Cooper, AIA, ACHA
Vice President | Healthcare

Geotechnical Study
Proposed Parking Garage
Michael E. DeBakey VA Medical Center
Houston, Texas

Fugro Consultants, Inc. (Fugro) is pleased to present this report of our geotechnical study for the above-referenced project. Mr. Will Chmylak, AIA of HOK requested our services in an email to Mr. Scott Marr, P.E., LEED AP of Fugro on August 3, 2010. This study was performed in general accordance with our Proposal No. 04.12100060p (Revision No. 1) dated August 4, 2010.

This report contains the results of our field and laboratory testing as well as our geotechnical recommendations for the design and construction of foundations for the proposed parking garage at the Michael E. DeBakey VA Medical Center in Houston, Texas.

We appreciate the opportunity to be of continued service to HOK. Please call us if you have any questions or comments concerning this report or when we may be of further assistance.

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

Dennis J. Turner, P.E.
Senior Geotechnical Engineer

11 FEB 2011

Scott A. Marr, P.E., LEED AP
Project Manager

STATE OF TEXAS
SCOTT A. MARR
99309
LICENSED
PROFESSIONAL ENGINEER

Copies Submitted: Addressee (4)
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1.0 INTRODUCTION

1.1 Project Description

We understand that the Department of Veterans Affairs has selected HOK as the Architect and Rogers Moore Engineers as the Structural Engineer to design a new parking garage within the existing parking lot north of the main hospital building at the Michael E. DeBakey VA Medical Center in Houston, Texas. Current plans indicate the garage will be cast-in-place or a precast concrete structure with the first parking level at grade (no basement level). We understand a 200-car garage with expansion capability up to 1,200 cars is planned. The maximum footprint of the garage is 180 ft by 350 ft with column loads on the order of 950 kips dead load and 280 kips live load.

1.2 Purposes and Scope

The purposes of our geotechnical study were to: 1) explore and evaluate subsurface soil and depth-to-water conditions at the site of the proposed structure, and 2) develop geotechnical engineering recommendations to guide others in the design and construction of a foundation system for the proposed structure. The scope of this study included the following:

- Reviewing soil borings and laboratory tests results from nearby geotechnical studies at the Michael E. DeBakey VA Medical Center;
- Drilling five (5) soil borings to explore subsurface conditions and obtain samples for geotechnical laboratory testing;
- Performing laboratory testing on selected soil samples to assess pertinent engineering properties;
- Analyzing the field and laboratory data to develop geotechnical engineering recommendations for a foundation system; and
- Preparing this report summarizing our findings and geotechnical recommendations.

Environmental assessments, compliance with state and federal regulatory requirements, and/or environmental analyses including those with mold, fungi, and other biologic agents were beyond the scope of our services. A geologic fault study was outside the scope of our services.

1.3 Previous Geotechnical Studies

Fugro has performed several geotechnical studies at the DeBakey VA Medical Center dating back to the 1950's. The projects included site geology and faulting information, geotechnical foundation and pavement recommendations, as well as construction monitoring and materials testing. Fugro completed geotechnical studies for the following projects:

- Fugro (formerly Greer & McClelland) Report No. 5477 – *Geotechnical Study Additions to the Veterans Administration Hospital*, dated September 24, 1954.
- Fugro (formerly McClelland Engineers) Report No. 0183-1184 – *Geotechnical Investigation Main Hospital Building*, dated June 18, 1984.
- Fugro (formerly McClelland Engineers) Report No. 0185-6078 – *Geotechnical Investigation Buildings 102, 103, 104, and Service Tunnel*, dated July 10, 1985.
- Fugro (formerly McClelland Engineers) Report No. 0185-6177 – *Settlement Analysis Main Hospital Building*, dated October 10, 1985.
- Fugro (formerly McClelland Engineers) Report No. 0185-6078-1 – *Preliminary Study of Odorous Soil Samples Veterans Administration Replacement Hospital*, dated March 26, 1986.

1.4 Applicability of Report

The explorations and analyses for this study, as well as the conclusions and recommendations in this report, were selected or developed based on our understanding of the project as described above. If there are differences in location or design features as we understand them, or if the locations or design features change, we should be authorized to review the changes and, if necessary, to modify our conclusions and recommendations.

We have prepared this report exclusively for HOK to guide others in the design and construction of a foundation for the proposed structure as described herein. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. We intend for this report, including all illustrations, to be used in its entirety. This report should be made available to prospective contractors for information only and ***not*** as a warranty of subsurface conditions. This report should ***not*** be used as a stand-alone construction specifications document.

2.0 FIELD EXPLORATION

Our field activities related to geotechnical soil borings are discussed in this section. We have included discussion relating to drilling methods, sampling methods, depth-to-water observations, and borehole completion.

2.1 General

Our field exploration plan was based on the information provided to us by HOK. We drilled five (5) soil borings designated B-1 through B-5 in an attempt to characterize subsurface conditions. The approximate boring locations are shown on the *Plan of Borings* presented on Plate 2. Boring depths ranged from 6 to 60 feet below existing grade. Materials that refused auger advancement were encountered at a depth of 6 feet during drilling of Boring B-2. A total of four attempts were made to advance this boring after offsetting horizontal distances of about 10 feet. These locations are designated B-2, B-2A, B-2B, and B-2C on Plate 2. Each of the four boring attempts encountered materials that refused auger advancement at depths between 6 and 10 feet. Boring logs were only prepared for Borings B-2 and B-2A.

Detailed descriptions of the soils encountered in the borings drilled for this project are presented on the boring logs in Appendix A. A key identifying the terms and symbols used on the boring logs is presented on Plates A-7a and A-7b. Depths for each boring are noted on the boring logs. Elevation information was not available at the time of this report.

2.2 Drilling Methods

Prior to drilling each soil boring we cored through the existing concrete pavement at the site using a 8-inch diameter diamond core barrel. The thickness of the pavement section at each boring location is noted on the boring logs presented in Appendix A. Below the pavement, we drilled each boring using our truck-mounted drilling equipment. The borings were generally drilled using a combination of dry-auger and wet-rotary drilling techniques. Dry-auger techniques were used in an effort to identify the depth-to-water in the open borehole as discussed later in this report.

2.3 Sampling Methods

Soil samples were generally taken at 2-foot intervals at a depth of about 16 feet and at 5-foot intervals thereafter to the completion depth of the borings. Undisturbed samples of cohesive soils were generally obtained by hydraulically pushing a 3-inch diameter, thin-walled tube a distance of about 24 inches. Our field procedure for sampling cohesive soil was conducted in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Soils*. The soil samples were extruded in the field and visually classified by our specialist field technician. We obtained field estimates of the undrained shear strength of the recovered samples using a calibrated hand-held penetrometer. The field estimates were modified for stiff to hard, over-

consolidated, *natural*, cohesive soils, as described on Plate A-7b. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory for additional geotechnical testing.

Granular soil samples were generally obtained using the Standard Penetration Test (SPT) as described on Plate A-7b. Our field procedure for granular soil sampling was conducted in general accordance with the ASTM D1586 (Standard Method for Penetration Test and Split-Barrel Sampling of Soils). Our field technician recorded the hammer blows for each sampling interval. The SPT N-values, as described on Plate A-7b, are recorded on the boring logs. Soil samples obtained from the split-barrel sampler were visually classified, packaged by the technician, and transported to our laboratory for testing.

Detailed descriptions of the soils encountered in the borings drilled for this project are presented on the boring logs in Appendix A. A key identifying the terms and symbols used on the boring logs is presented on Plates A-7a and A-7b.

2.4 Depth-to-Water Observations

Depth-to-water observations were performed in the open borehole of each boring as identified on the boring logs in Appendix A. If free water was encountered, drilling activities were halted to allow for depth-to-water measurements. Once depth-to-water measurements were completed, drilling then continued using wet-rotary techniques. Discussion of our interpretation of the depth-to-water conditions at the project site is presented later in Section 4.0.

2.5 Borehole Completion

After completing the field activities, each boring was backfilled with cement-bentonite grout. We grouted each borehole from the bottom up, using a tremie pipe. When grout returned to the surface, the concrete core was replaced in the grouted borehole to complete the boring.

3.0 LABORATORY TESTING

We directed our laboratory program toward classifying the foundation soils including evaluating the undrained shear strength and compressibility of the cohesive subsurface soils. Our laboratory tests were performed in general accordance with the appropriate standards as tabulated at the end of this section.

3.1 Classification Tests

The classification tests included tests for natural water content, liquid and plastic limits (collectively termed Atterberg limits), and material finer than the No. 200 sieve (percent fines). These tests aid in classifying the soils and are used to correlate the results of other tests performed on samples taken from different borings and/or different depths. The results of the classification tests are recorded on the boring logs in Appendix A. In addition, a phenolthalein indicator was applied to three samples of encountered fill materials to detect the presence of chemical stabilization agents (e.g. lime, flyash, Portland cement).

3.2 Shear Strength Tests

We measured the undrained shear strength from selected undisturbed samples of cohesive soils by performing unconfined compression and unconsolidated-undrained triaxial compression tests. The natural water content and dry unit weights were determined as routine parts of the compression tests. The results of the laboratory shear strength tests, along with the field estimates of shear strength, are presented on the boring logs in Appendix A.

3.3 Summary

Table 3-1 gives the types and number of laboratory tests as well as the applicable test method standard performed for this study.

Table 3-1. Laboratory Test Summary

Laboratory Test	***Quantity***	***Testing Standard***
Water Content	23	ASTM D2216
Atterberg Limits	8	ASTM D4318
Percent Passing the No. 200 Sieve	10	ASTM D1140
Unconfined Compression	9	ASTM D2166
Unconsolidated-Undrained Triaxial Compression	5	ASTM D2850

4.0 GENERAL SITE CONDITIONS

The interpreted site and subsurface conditions based on the field exploration and laboratory testing programs are discussed in this section. This section also includes a discussion of the depth-to-water conditions at the project site.

4.1 Site Location and Description

The proposed parking garage structure location is within the existing concrete parking lot north of the main hospital building at the Michael E. DeBakey VA Medical Center in Houston, Texas. The site is also south of Mixon Avenue and west of the existing Fisher House building. A *Vicinity Map* of the site location is presented on Plate 1 and a layout of the site location is presented on the *Plan of Borings* on Plate 2.

4.2 Historical Site Usage

According to our review of project information, there have been existing structures within the footprint of the proposed parking garage. The structures have been demolished; however, we are unaware of how the foundations were abandoned and the soils were treated. It is possible that some of the previous structures may have had a basement. Contractors should be made aware of the potential to encounter buried features from previous site usage. ***Specifically, note that we encountered materials that refused auger advancement at depths of 6 to 10 feet when drilling Boring B-2 during our field exploration.***

4.3 Subsurface Conditions

The subsurface conditions at the project site generally consist of concrete pavement over cohesive and granular fill overlying alternating layers of natural cohesive and granular soils. The subsurface stratigraphy is similar to those encountered in our geotechnical studies for adjacent projects within the DeBakey VA Medical Center. The subsurface stratigraphy can be generalized as follows.

4.3.1 Stratum I. Stratum I is composed primarily of cohesive and granular fill. The fill varies in depth from about 2 to 10 feet below the existing concrete pavement. The fill generally consists of sand, clayey sand, and clay. From observations of a phenolthalein indicator applied to samples of the fill soils, the sand and clayey sand layer immediately beneath the concrete pavement may be chemically stabilized.

The result of Atterberg Limits tests performed on samples of the clay fill indicate very high plasticity with liquid limits of 76 and 91, plastic limits of 17 and 22, and corresponding plasticity indices of 59 and 69. A single sample of the clay fill had 52 percent passing the No. 200 sieve (percent fines). Field estimates and laboratory tests indicate that the undrained shear strength of the clay fill soils

range from stiff (1,250 psf) to very stiff (2,500 psf). The fill was also noted to contain sand pockets and shell fragments.

4.3.2 Stratum II. Stratum II consists of primarily natural cohesive soils. Stratum II generally extends from below Stratum I to depths of about 4 to 13 feet below the existing concrete pavement. Atterberg Limits performed on samples of the natural clay soil indicate high to very high plasticity with liquid limits of 49 to 94, plastic limits of 14 to 20, and corresponding plasticity indices of 35 to 74. The very high plasticity soils had 88 to 90 percent passing the No. 200 sieve. Field estimates and laboratory tests indicate that the undrained shear strength of the clay fill soils range from firm (900 psf) to very stiff (2,100 psf). Calcareous nodules, ferrous nodules, organic material, silt pockets, and slickensided structures were encountered in these soils.

4.3.3 Stratum III. Stratum III consists of primarily granular soils. This stratum was generally encountered below Stratum II to depths of about 12 to 18 feet below the existing concrete pavement. The granular soils primarily consisted of silty sand, clayey sand, and silt. These soils had 39 to 83 percent passing the No. 200 sieve. SPT N-values ranged from 7 to 22 blows per foot, indicating a loose to medium dense insitu relative density. Field estimates indicate that the undrained shear strength of these soils range from stiff (1,500 psf) to very stiff (2,300 psf). Clay seams, calcareous nodules, and sand pockets were encountered in these soils.

4.3.4 Stratum IV. Stratum IV consists of primarily cohesive soils. This stratum was generally encountered below Stratum III to depths of about 43 to 49 feet below the existing concrete pavement. The cohesive soils primarily consisted of clay and sandy clay with 60 to 90 percent passing the No. 200 sieve. Field estimates and laboratory tests indicate that the undrained shear strength of these soils range from firm (900 psf) to very stiff (2,800 psf). Sand pockets, calcareous nodules, slickensided structures, silt pockets, and silt seams were encountered in these soils.

4.3.5 Stratum V. Stratum V consists of both granular and cohesive soils. This stratum was generally encountered below Stratum IV to depths of about 53 to 58 feet below the existing concrete pavement. The soils consist of clayey sand and sandy clay with 25 to 60 percent passing the No. 200 sieve. SPT N-values ranged from 3 to 19 blows per foot, indicating a very loose to medium dense insitu relative density. Field estimates indicate that the undrained shear strength of the cohesive soils is firm (600 psf to 900 psf). Clay seams and sand seams were also encountered in these soils.

4.3.6 Stratum VI. Stratum VI consists of primarily cohesive soils. This stratum was generally encountered below Stratum V and extended to the termination depths of the deeper borings (60 feet). Field estimates and laboratory tests indicate that the undrained shear strength of the clay soils is very stiff (2,700 psf to 3,400 psf). Siltstone nodules and silt pockets were encountered in these soils.

4.4 Depth-to-Water Conditions

Depth-to-water observations were taken in the open boreholes of each boring during our field exploration activities for this study. These measurements are presented on the boring logs in Appendix A. Free water was not encountered in any of the borings to depths of 6 to 30 feet below existing grade before encountering materials that refused auger advancement or switching to wet-rotary drilling techniques.

It should be noted that short-term depth-to-water observations recorded in open boreholes should ***not*** be considered to represent a long-term condition. The time associated with short-term observations may not be sufficient for the conditions in the open borehole to reach equilibrium. More accurate determinations of groundwater levels are usually made using long-term standpipe piezometers readings. It should also be noted that groundwater levels will fluctuate with seasonal variations in rainfall (heavy rainfall and extended periods of dry weather) and surface runoff. At this site, we believe the ground water level is also likely influenced by dewatering systems for nearby structures. For foundation design purposes, the groundwater level should be assumed at the ground surface.

4.5 Variations in Subsurface Conditions

Our interpretations of soil conditions, as described in this report, are based on data obtained from our visual observations, sample borings, laboratory tests, and our experience. Although we have allowed for minor variations in the subsurface conditions, our recommendations may ***not*** be appropriate for subsurface conditions other than those reported herein. It is possible that some undisclosed variations in soil or groundwater conditions might occur outside the boring locations, **especially with respect to the presence, depth, consistency, and extent of fill material across the site and the presence of buried structures from previous site usage.** We recommend careful observations during construction to verify our interpretations. Should variations from our interpretations be found, we recommend that we be notified and authorized to evaluate what, if any, revisions should be made to our recommendations.

5.0 IMPACT OF PREVIOUS SITE USAGE

The location of the proposed parking garage is currently occupied by a concrete parking lot. According to our review of project information, there were several existing structures on the site that have since been removed. There is a possibility of encountering existing foundations and other buried obstructions in the area during site preparation and foundation installation. Also note that materials that refused auger advancement were encountered at depths of 6 to 10 feet when drilling Boring B-2 during our field exploration. We recommend consideration be given to relocating the proposed structure to avoid influence of buried structures. Additional exploratory study or geophysical survey may also be used to identify the presence and location of existing buried structures. During construction, we should be given the opportunity to help evaluate any buried features, if encountered. The impact of the buried structures will need to be evaluated once a foundation scheme is chosen and designed. If a shallow foundation system is used, the depth of existing buried features will need to be evaluated with regard to the depth of the shallow foundations.

If a deep foundation system is used, and if existing foundations are found to be shallow (3 feet or less), then it may be feasible to excavate and remove the footings and backfill with properly placed and compacted structural clay fill. However, if the existing footings are found to be deeper than 3 feet, we should be retained to evaluate the impact of those footings on the proposed structures and to offer recommendations. Deeper foundations (greater than 3 feet) should be left in-place and cut-off below grade to avoid disturbing the foundation soils under the guidance of the Geotechnical Engineer-of-Record or their qualified representative. We also recommend that a cushion of properly placed and compacted structural clay fill should be placed on top of the cut-off portion of any existing footings to prevent the development of hard spots within new foundation. Fugro representatives should observe the site preparation and foundation installation. It should also be noted that additional soil samples may be required if abandoned foundations or other buried structures are encountered.

6.0 SHALLOW FOUNDATION RECOMMENDATIONS

This section presents our recommendations for a shallow foundation system consisting of spread and strip footings to support the parking garage. We do not recommend placing shallow foundations above existing buried structures without further evaluation.

Spread footings refer to square and rectangular footings used to support a single column load. Strip footings refer to footings where the length-to-width ratio is generally greater than 10. It should be noted that the length-to-width ratio generally refers to the length-to-width of a monolithically poured and constructed footing section. Strip footings are generally used to support wall loads. Combined footings usually refer to rectangular footings that are used to support two or more column loads (from the same column line) on a single monolithically poured and constructed footing section.

6.1 Foundation Depth

Strip and spread footings should be placed at a depth of 6 feet below existing grade if deep foundations are also used to support the parking garage. The soil conditions encountered during construction should be of similar consistency and strength of the soil conditions reported on the boring logs. The Geotechnical Engineer-of-Record or their qualified representative should observe all foundation excavations. We recommend excavation grade be such that all foundations have an embedment of at least 3 feet below adjacent grade.

Weak, soft, wet, disturbed, or otherwise unsuitable soils encountered at the foundation depth should be over-excavated and replaced with lean concrete. Alternatively, the footing depth could be increased under the guidance of the Geotechnical Engineer-of-Record. Additional temporary groundwater control may be required for deeper foundation depths. Discussion relating to footing excavations is provided in this report in Section 10.2.

6.2 Allowable Net Bearing Pressure

From a geotechnical perspective, the performance of a foundation system should provide an adequate factor of safety against shear failure of the foundation soils and reduce the potential for excessive settlements due to overstressing of the underlying foundation soils. Shallow foundations should be designed such that their applied bearing pressures do ***not*** exceed the allowable net bearing pressure of the underlying soils.

The allowable net bearing pressure for shallow foundations is a function of, among other items, the bearing surface, the strength of the foundation soils, the location of the foundation, the shape of the foundation, and the recommended factor of safety. Shallow foundations should be proportioned so the maximum contact pressure under dead, live, and transient loads, including wind loads, does ***not*** exceed the allowable net bearing pressure of the foundations soils.

It should be noted that the presented allowable net bearing pressures are for foundations supported on undisturbed, competent natural cohesive soils. Foundations should ***not*** be placed on fill material. The allowable net bearing pressures presented in this report include a factor of safety ranging between 1.5 to 2 with respect to shear failure of the foundation soils.

Total loading conditions as described in this report refers to the combination of properly factored dead and live loads. Transient loading conditions refer to the combination of properly factored dead, live, and infrequent transient (*e.g.*, wind) loads.

Table 6-1. Allowable Net Bearing Pressure

Type of Foundation	***Foundation Depth*** [1]	***Loading Conditions*** [2]	
		Total	***Transient***
		(F.S. of 2.0)	*(F.S. of 1.5)*
Spread and Strip Footings	6 feet	4,500 psf	6,000 psf

Notes:

1. Foundation depth referenced from existing grade at the time of our study. Potential changes to site grade should be addressed during final design phase. All foundations should be placed on observed stiff cohesive soils.
2. Refer to report text for additional information relating to loading conditions.
3. F.S. refers to factor of safety.

Foundations should be proportioned so that the maximum contact pressure under various load combinations does not exceed the allowable net bearing pressure given above. Bearing pressure, as used in this report, is defined on Plate 3. To calculate values of W_e, W_s, and W_f from Plate 3, use effective unit weights of 60 pcf for soil and 90 pcf for concrete.

6.3 Resistance to Lateral Loads

Foundations constructed in open excavations will resist lateral loads by developing adhesion (cohesive soil) or friction (granular soil) along the base of the foundation and passive soil pressure against the foundation side perpendicular to the applied lateral loads. Shallow foundations at this site will bear on cohesive soils. We anticipate that seal slabs of lean concrete will be used to protect exposed foundation soils and facilitate construction of shallow foundations. Seal slabs should be roughly finished to promote bonding with the foundation concrete.

For design purposes, the resistance due to adhesion and passive soil pressure can be assumed to develop simultaneously. Base adhesion should be neglected where positive pressure is not maintained beneath the foundation. For concrete foundations placed in good contact with undisturbed, natural cohesive soil, the allowable base adhesion may be taken as 200 psf. We recommend that the allowable net passive pressure be taken as 1,000 psf for natural cohesive soils and 800 psf if properly placed and compacted structural clay fill is used to backfill the footings. Passive resistance within the top 5 feet (below the lowest adjacent final grade) should be neglected unless area paving is provided for a distance of at least 5 feet beyond the edge of the

foundations. We expect that these allowable values will provide a factor of safety of at least 2 with respect to ultimate values.

6.4 Resistance to Overturning Loads

The design of shallow foundations subjected to overturning moments should consider a trapezoidal distribution of contact pressure between the base of the foundation and the foundation soil such that full soil contact and positive pressure is maintained under the foundation. The maximum contact pressure should not exceed the appropriate allowable net bearing pressure for the foundation soils as discussed in Section 6.2.

6.5 Resistance to Uplift Loads

The ultimate uplift capacity of spread and strip footings is limited to the weight of the foundation plus the weight of any soil directly above the foundation. We recommend an effective unit weight of 60 pcf for soil and 90 pcf for concrete to compute the ultimate uplift capacity. We recommend that the calculated ultimate uplift capacities for shallow foundations are reduced by a factor of 1.2 to compute the allowable uplift resistance.

6.6 Estimated Settlement

Detailed settlement analyses for the recommended shallow foundations were outside the scope of this study. However, settlement estimates were determined based on the results of our experience near the site and with other sites with similar subsurface conditions and structures. Total consolidation settlements on the order of 1 inch are expected for the spread and strip footings described in this report. Differential settlements are expected to be approximately ½ of the total settlement. These estimates are based on the assumption that the foundations act as isolated foundations, that is, the clear spacing between foundations is equal to or greater than the width for diameter of the foundation.

7.0 DEEP FOUNDATION RECOMMENDATIONS

Deep foundations can be also be used to support the proposed parking garage. Due to the proximity of nearby structures we recommend using either ACIP piles or drilled shafts. The following sections provide our recommendations for foundation depth, axial capacity, axial group effects, lateral group effects, and load testing for deep foundations.

7.1 Static Axial Capacity

The ***ultimate*** axial capacity, in both compression and tension, of a unit diameter (1-foot) ACIP pile and drilled shaft were computed using the static method of analysis in general accordance with the API Method[1]. In this method, the ultimate compressive capacity of the pile/shaft is taken as the sum of the skin friction along the pile/shaft. When computing ultimate compressive and tensile capacity for ACIP piles and drilled shafts, the end-bearing component is neglected. The weight of the pile/shaft is also neglected in the computations. We also neglected the strength of the soil to a depth of 5 feet to account for soil variability and construction disturbances. Potential changes to site grade should be considered when using the axial capacity curves.

The ***ultimate*** axial capacity curves of an ACIP pile and drilled shaft are presented on Plate 4. We recommend a factor of safety of 2 be applied to the ultimate axial capacity of piles/shafts loaded in compression (transient and sustained) and transient tension. A factor of safety of 3 should be applied for sustained tension loads. In addition, tension capacity should be limited to the depth of steel reinforcement that can develop the proposed tension load.

The tensile (uplift) capacity of ACIP piles and drilled shafts may be increased by adding the weight of the reinforced portion of the pile/shaft to the computed tensile capacity presented on Plate 4. The effective weight of the pile/shaft should be used. An effective unit weight of 90 pcf is typically used for grout/concrete. A factor of safety of 1.2 should be applied to the pile/shaft weight.

7.2 Axial Group Effects

The overall allowable load-carrying capacity of a group of ACIP piles or drilled shafts may, in some cases, be less than the sum of the individual allowable capacities. A reduction in the individual pile/shaft capacity, to allow for group effects, is usually not necessary for piles/shafts having a center-to-center spacing of 3 to 5 or more pile/shaft diameters. The reduction in individual capacity depends on many factors including the configuration of the group, number of piles/shafts in the group, pile/shaft size, the depth of installation, and the pile/shaft spacing. We recommend ACIP piles and drilled shafts for this project be spaced at least 3 diameters (center-to-center) to reduce substantial group effects. If piles/shafts are spaced closer or if pile/shaft groups larger than 5-by-5

[1] American Petroleum Institute. (1986). *Recommended Practice for Planning, Designing, and Construction of Fixed Offshore Platforms.* API Recommended Practice 2A (RP 2A), 16th ed., Washington, D.C.

are anticipated, we would be pleased to review the design and comment on group effects. Piles/shafts should also be sufficiently spaced from any foundations of the proposed adjacent structures.

7.3 Settlement Considerations

A detailed settlement analysis for ACIP pile or drilled shaft foundations was beyond the scope of our services. However, based on our experience and results from our consolidation tests, we expect settlements due to consolidation for properly installed individual, isolated ACIP piles or drilled shafts to be less than about 1/4 to 1/2 inch. It should be noted, that groups of piles/shafts will likely settle more than individual piles subjected to the same load per pile depending on actual soil conditions, pile/shaft dimensions, and group configuration. The increase in settlement between individual piles/shafts and groups is generally negligible for groups of piles/shafts that are less than about 5-by-5. The settlement of pile/shaft groups is dependent on several variables including: dimension of the pile/shaft group, the pile/shaft length, the sustained structural load, and the compressibility characteristics of the foundation soils. We would be pleased to perform a detailed pile/shaft group settlement analysis on a case-by-case basis under separate cover.

7.4 Load Test

We recommend that axial load tests be performed in general accordance with ASTM procedures. The advantage of performing a load test can be realized in using a reduced factor of safety applied to the ***ultimate*** axial pile capacity (for certain loading conditions), therefore, resulting in shorter pile lengths. Load testing should be supervised by the Geotechnical Engineer-of-Record or their qualified representative. Each test pile should be loaded to produce enough movement at the top of the pile to determine the ***ultimate*** capacity or at least 3 times the design load. We recommend that we be retained to review the results of load tests to: 1) evaluate the load/unload-displacement response of the piles, 2) evaluate the ***ultimate*** capacity of test piles, and 3) compare measured capacities and deflections with design criteria.

7.5 Pile Caps

We anticipate that if deep foundations are used, the design will likely use clusters of rectangular pile caps to support loads from interior columns and a perimeter strip footing pile cap will be used to support loads from exterior columns. We expect that the interior pile caps will be constructed similar to a mat foundation and that the exterior perimeter pile cap will be constructed similar to a strip footing. In each case, the reinforced concrete pile caps will likely be placed at a depth of approximately 4 to 6 feet below existing grade and will likely be approximately 4- to 5-feet thick. We do not recommend placing the pile caps shallower than 4 feet below existing grade.

7.5.1 Bearing Capacity. The available contribution of bearing capacity contribution from the pile cap will be dependent upon several factors including, pile-soil interaction, pile settlement, pile group behavior, the rigidity of the piles and pile cap, and the underlying foundation soils.

Generally, the movement associated with mobilizing end bearing beneath the pile cap is beyond tolerable structural limits. This is especially true for “floating” piles, *i.e*. piles that have been designed for skin friction only, similar to those planned at this site. For the planned pile caps to mobilize end bearing, the pile groups would have to approach their ultimate capacity. We expect that this would translate in significant movement of the pile groups. The movement would vary depending on loading conditions, pile sizes, and group configurations. Thus, differential movement would be expected between varying pile groups. In addition, the magnitude of the differential movement would likely vary based on the variability of fill material within the footprint of the building. As such, we do not recommend utilizing bearing capacity beneath the pile cap.

7.5.2 Subgrade Preparation. We recommend that the exposed subgrade between the pile caps and the perimeter strip cap be observed and probed prior to placement of backfill. The Geotechnical Engineer-of-Record or their qualified representative should observe the subgrade for competency. Areas of weak, soft, or otherwise disturbed soils should be over-excavated and replaced with structural clay fill or flowable fill. The intention of subgrade preparation activities should be to create a uniform base for the placement of backfill between the pile caps.

7.5.3 Fill Between Pile Caps. We recommend that the area between the pile caps be backfilled with either structural clay fill or flowable fill. Recommendations for the selection and placement of structural clay fill and flowable fill are provided in Section 10.3. If should be noted that structural clay fill placed as described in this report may settle up to 1 to 2 percent of its height due to its own weight over the life of the structure. As such, it may be more beneficial to use flowable fill, which is not expected to settle appreciably, as backfill between the pile caps. Flowable fill is also generally easier to place in confined areas, such as between individual pile caps.

8.0 FLOOR SLAB RECOMMENDATIONS

The parking garage may be designed as a conventional slab-on-grade with proper site preparation. The following paragraphs present our discussions and geotechnical recommendations for floor slab design, including subgrade modulus, base materials, and control of water pressures.

8.1 General

We expect that once the pile caps or shallow foundations are completed the over-excavated areas between individual pile caps or shallow foundations will be backfilled. An additional 1 to 2 feet of structural clay fill or flowable fill will be placed on top of the pile caps or shallow foundations and underlying backfill soils to facilitate utility placement and construction of the first floor slab.

8.2 Pad for Slab-on-Grade

We recommend that a 1 to 2-foot-thick pad of structural clay fill or flowable fill will be placed above the pile cap or shallow foundation backfill to facilitate the installation of utilities and the construction of the first floor slab. Leveling sand and a moisture barrier should be used to separate the structural clay fill or flowable fill from the slab-on-grade. The influence of the fill pad for the slab-on-grade on the installation of utilities, especially gravity flow lines, should be considered.

8.3 Subgrade Modulus

For floor slabs bearing on a properly prepared subgrade, we recommend that a modulus of subgrade reaction of 75 pci be used for design. This value of subgrade modulus is based on a 1-foot by 1-foot bearing area and, depending on the design software, may need to be scaled for size effects. The modulus is also based on movements from the floor slab pressure only. It does not consider any settlement of the fill material due to its own weight.

8.4 Bearing Pressure

We recommend limiting the applied pressure beneath the first floor slab-on-grade supported on properly placed and compacted structural fill to 800 psf. Applied pressures greater than 800 psf will likely require the use of supplemental foundations. It should also be noted that applied pressures greater that 800 psf may also cause isolated areas of differential movement.

8.5 Additional Considerations

Positive drainage should be provided away from the building and floor slab. We recommend using structural clay fill as backfill outside the building footprint. The low permeability of structural clay fill will help reduce the potential for moisture fluctuations along the building perimeter foundation. Moisture fluctuations can adversely affect floor slab performance.

9.0 PAVEMENT RECOMMENDATIONS

The following subsections provide our general pavement recommendations for this project site. We have included general recommendations for flexible and rigid pavements subjected to conventional pavement loads as well as recommendations for pavements subjected to heavy loads. We have also included discussions relating to subgrade preparation, general pavement sections, soil stabilization, and drainage. It should be noted that we were not provided specific traffic loading conditions for the proposed pavements. As such, the following are general recommendations for the generalized soil conditions at the site of the proposed development.

We recommend that consideration be first given to the use of rigid pavements, rather than flexible pavement, in high traffic areas at this site. Our experience has shown that flexible pavements have a higher potential for developing distress, such as pavement rutting and shoving, especially during summer months, as well as at stopping and turning areas. The long-term performance of rigid pavements is also favorable for heavy-duty applications.

9.1 Subgrade Preparation

The performance of an asphalt or concrete pavement ultimately depends on the underlying subgrade. Subgrade preparation for pavements should include clearing and stripping all significantly organic material, debris, and other deleterious materials from the site. The subgrade preparation should extend (where possible) laterally at least 5 feet beyond the edges of the pavements. After removing deleterious materials and stripping, the exposed subgrade should be proofrolled with a fully loaded dump truck or water truck with a weight of at least 20 tons and a tire pressure of at least 70 psi and observed by the Geotechnical Engineer-of-Record or their qualified representative to evaluate the condition of the subgrade. We do not recommend using off road earth moving equipment (i.e. loaders and scrapers), compactors, or track-mounted vehicles (i.e. bull dozers and front end loaders) for proofrolling. Proofrolling specifications should provide for rut depths less than 1 inch and no visual evidence of pumping. We recommend performing these activities during a relatively dry period. We do not recommend that subgrade preparation activities begin immediately after or during a significant rain event.

The Geotechnical Engineer-of-Record or their qualified representative should be contacted to evaluate the site conditions if a large rainfall event occurs during subgrade preparation. It may be necessary to wait for the site to dry prior to restarting site preparation activities depending on the effectiveness of onsite drainage.

Areas of the subgrade that are observed to be soft, wet, weak, or contain deleterious materials should be over-excavated to expose competent soils. Areas of the subgrade in which pumping or significant deflections are observed should also be over-excavated to expose competent soils. Over-excavated areas should be backfilled with properly placed and compacted structural clay fill. Recommendations for structural clay fill are presented later Section 10.3.

Grading activities near catch basins and inlets should be conducted such that the recommended pavement sections (*i.e.* layer thickness) are adhered to at the catch basins and inlet locations. We recommend that the Geotechnical Engineer-of-Record or their qualified representative be present onsite to observe subgrade preparation for pavements. Equally important, the backfill soil around catch basins should properly placed and compacted according to recommendations presented herein.

9.2 Conventional Pavement Sections

We were not provided with specific traffic loadings for the proposed pavements at this project site. As such, a detailed pavement design was beyond the scope of this study. If desired, we can perform a detailed pavement design based on the type and frequency of vehicles anticipated. Table 9-1 provides typical pavement sections based on our experience with similar subsurface conditions and expected traffic. These sections are ***not*** based on a specific loading conditions (*e.g.* equivalent-single-axle-loads, *ESALs*) or pavement life expectancy.

Table 9-1. Conventional Pavement Sections

Pavement Type and Recommended Use	***Material***	***Thickness***	***Reference / Specification***
Flexible Pavement (parking areas for cars and light-duty trucks)	Hot-Mix Asphaltic Concrete	2 to 3 inches	TxDOT Item 340 Type C or D
	Over		
	Crushed Limestone Base	8 inches	TxDOT Item 247 Type A, Grade 1
	Over		
	Lime-Stabilized Subgrade	8 inches	See Report Text
Rigid Pavement (parking areas for cars and light duty trucks)	Portland Cement Concrete (Flexural strength > 550 psi)	5 inches	TxDOT Item 360
	Over		
	Lime-Stabilized Subgrade	8 inches	See Report Text
Rigid Pavement (truck/heavy traffic areas)	Portland Cement Concrete (Flexural strength > 550 psi)	7 inches	TxDOT Item 360
	Over		
	Lime-Stabilized Subgrade	8 inches	See Report Text

9.2.1 Lime-Stabilized Subgrade. The lime-stabilized subgrade should be placed and compacted in accordance with the recommendations presented in Section 10.3.2. Care should be taken not to over-compact the subgrade soils adjacent to foundations or buried utilities.

9.2.2 Crushed Stone Base. The crushed limestone base should be in accordance with Texas Department of Transportation (TxDOT) *Standard Specifications for Construction of Highways,*

Streets, and Bridges[2] Item 247. Crushed limestone should be compacted to 98 percent of the maximum dry density as determined by TxDOT Test Method Tex-113-E.

9.2.3 Hot Mix Asphalt. Hot mix asphaltic concrete (HMAC) should be placed in accordance with TxDOT Item 340[2]. The HMAC should be either a Type D or Type C surface course mix. The asphaltic concrete should be compacted to between 91 and 95 percent of the maximum theoretical density as described by TxDOT Item 340 Specifications.

9.2.4 Concrete. Portland cement concrete should be in accordance with TxDOT Item 360[2]. The concrete should have a compressive strength of 3,500 psi or greater at 28 days and be placed in accordance with American Concrete Institute (ACI) guidelines. Reinforcement and joint spacing for the concrete section should be evaluated by the Structural Engineer-of-Record for the project. We recommend that the Portland cement concrete pavement be steel reinforced and that the concrete slabs have sufficient joints to allow for contraction and expansion of the concrete.

9.3 Drainage

The importance of drainage to the proper operation and function of any pavement cannot be overemphasized. The pavement and subgrade surface should be raised above adjacent grade and properly sloped into drainage inlets or lateral ditches. Water should not be allowed to pond on or adjacent to the pavement whereby the subgrade may become saturated. If the pavement sub-layers do become saturated, the bearing capacity will be greatly reduced and the useful life of the pavement will be decreased. Periodic inspections and repair of cracks in pavement sections should be performed as part of routine maintenance.

[2] Texas Department of Transportation, *Standard Specifications for Construction of Highways, Streets, and Bridges*, 2004.

10.0 CONSTRUCTION CONSIDERATIONS

Recommendations regarding site preparation, shallow open-cut excavations, fill selection and placement, installation of augered cast-in-place piles, installation of drilled shafts, sonic integrity logging, and construction monitoring are provided in the following paragraphs.

10.1 Site Preparation

The degree of site preparation to be completed will depend on the foundation contractor's requirements. Positive surface drainage away from the location of the proposed structure should be established to prevent surface water runoff from flooding the prepared site and from ponding around the completed foundations. All existing utilities should be identified and, if necessary, relocated prior to construction of new foundations, or abandoned in place. We recommend that the Geotechnical Engineer-of-Record or their qualified representative be onsite to observe site preparation activities, especially in the event that abandoned foundations or other buried features are encountered.

10.2 Shallow Open-Cut Excavations

Temporary excavations will be required for construction of shallow foundations, pile caps, and utility trenches. The excavations should be opened using an excavator with a smooth (not toothed) bucket. The excavation should be "dry" and free of loose, soft or disturbed soil. Poor or soft soils exposed at the bottom excavations should be over-excavated and replaced with lean concrete.

Excavation safety systems should be in accordance with federal Occupational Safety and Health Administration (OSHA) Standards, 29 CFR Part 1926 (Revised July 1992), Subpart P, Excavations. Details for open-cut slopes and excavation shoring based on soil type and groundwater conditions are provided in the latest amended OSHA federal regulations. Excavations 4 feet deep or less are not generally required to be sloped back or braced following federal OSHA requirements for excavations. Based on our interpretation of the regulations and anticipated soil conditions, the cohesive fill soils and shallow granular soils would be classified as Type C soils and the natural cohesive soils would be Type B. Sides of temporary vertical excavations less than 4 feet deep may stay open for short periods of time. However, if sides of slopes begin to slough, the sides should be either braced or sloped back to a stable condition. Excavations deeper than about 4 feet should be either braced or sloped back no steeper than 1.5-horizontal to 1-vertical for Type C soils and 1-horizontal to 1-vertical for Type B soils.

The contractor should be prepared to handle any seepage into shallow excavations. This seepage is expected to be minor, and the contractor should be able to handle any seepage by pumping from sumps.

Soils exposed by the excavations for foundations should be protected from disturbance due to construction activities and exposure to free water. A seal slab should be placed to help maintain existing moisture conditions. We also recommend that good surface drainage away from the excavations should be established to prevent surface runoff from flooding excavations and ponding around the completed foundation.

10.3 Fill Selection and Placement

The following sections present our recommendations for fill selection and placement.

10.3.1 Structural Clay Fill. Structural clay fill should be used to bring over excavated areas to design rough grade, as backfill against pile caps and foundations. We recommend using low plasticity cohesive soils for structural clay fill. Structural clay fill should have a liquid limit of less than 40, a plasticity index between 8 and 20, and at least 60 percent of the material finer than the No. 200 sieve. Structural clay fill should be free of deleterious matter and should have an effective clod diameter less than 3 inches. We do ***not*** recommend mixing sand with high plasticity clay to develop structural clay fill. Some of the onsite soils meet these requirements. However, their physical characteristics should be confirmed by the Geotechnical Engineer-of-Record prior to their usage.

Structural clay fill should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted to 95 percent of the maximum dry density at a moisture content of 1 percent “dry” to 3 percent “wet” of optimum as determined by ASTM D698 (Standard Proctor). Structural clay fill should be compacted by a sheepsfoot or padfoot type roller, or by alternative methods that provide a “kneading” compaction equivalent to the sheepsfoot or padfoot roller. We recommend using hand-operated compaction equipment and 4-inch thick loose lifts adjacent to foundations, utilities, walls, or other structural features to avoid damage and inducing excessive lateral loads on the structural elements.

10.3.2 Lime-Stabilization. Lime-stabilization may be used to modify onsite cohesive materials. Laboratory tests should be conducted at the time of construction to determine the optimum lime content. The optimum lime content is the amount of lime necessary to achieve a pH of 12.4 (which represents lime fixation), while trying to achieve a plasticity (PI) of less than 20. For estimation purposes, about 8 to 10 percent lime, by dry weight, may be required to stabilize the onsite highly plastic clays. Organics, chemical fertilizers, and some clay minerals can modify the amount of lime necessary for lime fixation.

We recommend that a lime series be performed using the specific soil samples and proposed lime additive. Lime-stabilization should be done in accordance with the Lime Association recommendations. Key items for lime-stabilizing the clay soils include placing the proper percentage of lime, thoroughly mixing the lime into the clay soils, bringing the stabilized soil to the proper moisture content, allowing the stabilized soil to cure for at least 48 hours, adjusting the moisture content from 1 percent “dry” to 3 percent “wet” of optimum moisture content, pulverizing

the soils again until the lime is thoroughly blended, then placing the stabilized soil in accordance with the recommendations discussed herein.

Lime-stabilized clay fill should be placed in 6- to 8-inch-thick loose lifts and uniformly compacted to 95 percent of the maximum dry density as determined by ASTM D698 (Standard Proctor). We recommend using 4-inch-thick lifts in confined spaces. The moisture-density relationship should be established based on a material sample obtained on-site after stabilization with lime. A combination of sheepsfoot or padfoot rollers and pneumatic rollers is recommended to compact the lime-stabilized clay fill.

10.3.3 Flowable Fill. Flowable fill (soil-cement slurry) may be used as backfill adjacent to pile caps and to replace over-excavated areas during the foundation excavation. Flowable fill to be used for backfill should meet the following recommendations:

- sandy soils used for flowable fill contain no more than 30 percent fines,
- the soil and cement for the flowable fill should be thoroughly mixed,
- proportioning of the sandy soil and cement is sufficient to produce a minimum unconfined compressive strength of 50 psi at 7 days while remaining workable for placement,
- placement of the flowable fill should not be performed when the ambient air temperature is below 40°F,
- placement of the flowable fill should completely fill the space to be backfilled,
- flowable fill is properly compacted using standard ASTM procedures for soil-cement materials; if a fluid consistency is used for the flowable fill, an “elephant trunk” vibrator may be used to consolidate and remove air voids form the mixture,
- the consistency of the mix shall be tested by filling an open-ended 3-inch diameter cylinder 6 inches high, to the tope with flowable fill; the cylinder shall be immediately pulled straight up and the correct consistency should produce a minimum 8-inch diameter circular spread with no segregation and maintain the consistency when placed; and
- construction over the flowable fill will not be performed until after initial set (4 to 6 hours) has been achieved for the flowable fill.

10.4 Installation of Augered Cast-in-Place Piles

The proper installation of ACIP piles is dependent upon the Contractor’s experience, construction procedures, and equipment. Installation factors have a greater affect on the performance of ACIP piles compared to any other type of deep foundation. Detailed pile installation specifications specifically addressing the special concerns of ACIP piles should be prepared as part of the construction package. The pile specifications should be in general accordance with the *Augered*

Cast-in-Place Pile Model Specification[3], prepared by the Deep Foundations Institute. We should also be given an opportunity to review the proposed specifications prior to construction. Potential contractors should be made aware of the anticipated site and subgrade (*i.e.* soil and groundwater) conditions. We recommend consideration be given to utilizing a contractor with experience installing ACIP piles in the Medical Center.

10.4.1 Contractor Experience and Equipment. The selected contractor should have relevant project experience with augering and pumping equipment, installation of similar sized ACIP piles in similar subsurface soil conditions, placement of reinforcing steel, as well as experience handling the special grout mixes and admixtures. The contractor's crane should generally be equipped with a torque converter to allow the auger to be withdrawn at a slow continuous rate without sudden jerking. ***Our experience indicates that not all contractors have the specialized equipment and experience required to successfully install ACIP piles and particularly for the installation of piles with diameters of 24 inches and larger.*** We strongly recommend that the contractor's qualifications and experience records be reviewed as part of the selection process, especially if pile diameters greater than 24-inches are planned for this site.

10.4.2 Grout Mix. The ACIP pile contractor should be able to demonstrate that a grout mix can be furnished to meet this project's requirements. The results of a test mix design are generally required to be submitted prior to approval of the grout mix for construction. During construction, the fluidity of the grout mix should be frequently tested using a 3/4-inch diameter flow cone. Flow rates of 10 to 25 seconds are typically specified. Compressive strength of the grout mix should be checked by making at least six, 2-inch square cubes for each day of pile installation. Test cubes should be cured and tested in accordance with ASTM C109 and may be restrained from expansion as described in ASTM C942. The Deep Foundation Institute recommends testing two grout cubes at 7 days, two at 28 days, and keeping two on hold.

10.4.3 ACIP Pile Installation and Monitoring. The installation process makes it inherently difficult to verify the integrity of the installed ACIP piles, yet the integrity is essential to the load carrying capacity of ACIP piles. A comprehensive construction monitoring and record keeping program is recommended to help reduce the risks associated with improper construction techniques. Construction monitoring should be performed in accordance with the *Inspector's Guide to Augered Cast-in-Place Piles*[4] prepared by the Deep Foundations Institute. We recommend that qualified personnel independent of the contractor perform construction monitoring in conjunction with the Geotechnical Engineer-of-Record.

There are several aspects of the installation procedure that can be monitored to aid in assessing whether the ACIP pile is being installed properly. These aspects are: 1) viscosity of the grout mixture to be pumped, 2) initial grout placement prior to raising the augers and resulting grout head observed upon completion of pile installation, 3) incremental grout factor (volume of grout pumped

[3] *Augered Cast-in-Place Pile Model Specification*, Deep Foundation Institute, (1990).
[4] *Inspector's Guide to Augered Cast-in-Place Piles*, Deep Foundation Institute, (1994).

relative to the theoretical volume) computed over intervals of 5 feet or less as the auger is withdrawn, 4) uniformity of grout placement and computed grout factor along the entire length of the completed pile, 5) continuous grout placement and auger withdrawal without delays or grout pressure fluctuations, and 6) unobstructed and successful reinforcing steel placement in a timely manner following completion of grout placement. Records kept during construction monitoring should include these aspects of the pile installation.

The incremental volume of grout pumped as the auger is withdrawn is one of the more important installation controls. Common practice is to have the contractor calibrate the grout pump at the start of the work. The number of pump strokes is then computed to provide an initial grout head of 5 to 10 feet and at least 15 percent greater than the theoretical volume for each 5-foot increment of auger withdrawal. Typically, the actual volume of grout pumped is in the range of 15 to 60 percent greater than the theoretical volume. The required grout volume to obtain a uniform pile will vary depending on the soil type and consistency. The Geotechnical Engineer-of-Record or their qualified representative should continually monitor and document the number of pump strokes pumped over each 5-foot interval.

10.4.4 Reinforcing Steel Placement. The placement of reinforcing steel cages and tension steel can be improved significantly by using a second crane or cherry picker. The benefits of using a second crane to place the reinforcing steel are: 1) the ACIP pile installation crane can be more productive drilling and grouting piles, 2) the second crane has a better reach to pick up reinforcing steel away from the congested pile location, 3) there is better control of cage alignment within the grouted pile shaft, and 4) placement time is reduced by allowing the reinforcing steel to be lowered into the pile as soon as the auger is removed and the grouted pile top is cleaned.

10.4.5 Construction Sequence. The Contractor's installation plan should be reviewed at least one week prior to the start of construction. We recommend a preconstruction meeting to discuss the proposed installation procedures. The installation plan should address the allowable spacing between piles to be grouted the same day. In general, previously cast piles should achieve their initial set (at least 6 hours and preferably 1 day) prior to augering adjacent piles. We strongly recommend installing ACIP piles spaced closer than about 6 feet (clear spacing) on alternate days to reduce the risk of interference between adjacent piles. Piles installed at this site will likely penetrate into water-bearing granular soil layers. A drop in the grout level of a completed pile when installing an adjacent pile is an indication of a possible "blow-out" condition and should be addressed. We have observed, in the past, significant drops in the grout level of completed piles when installing adjacent piles through granular soil layers. As such, providing sufficient time at this site for individual piles to achieve their initial set before installing adjacent piles is essential.

10.5 Installation of Drilled Shafts

The proper installation of drilled shafts is dependent upon the Contractor's experience, construction procedures, and equipment. Detailed shaft installation specifications should be

prepared in accordance with the *Drilled Shaft Construction Procedures and Design Methods*[5] Manual. We should also be given an opportunity to review the proposed specifications prior to construction.

10.5.1 Contractor Equipment and Experience. Potential contractors should be made aware of the onsite soil and groundwater conditions. Surface casing may be necessary to install drilled shafts at this site. The casing should have at least 6 inches of stick-up above the ground surface to limit the amount of surficial debris from entering the excavation. We recommend consideration be given to utilizing a contractor with experience installing drilled shafts in the Medical Center.

Drilled shafts installed to depths greater than approximately 38 feet at this site will penetrate through water-bearing granular soil layers. Experience shows that the walls of the shafts excavated in similar soils, especially those below the depth-to-water will collapse unless supported by casing and/or heavyweight slurry. Therefore, use of a temporary casing and/or slurry is recommended to install drilled shafts at this site. The use of temporary casing requires that the Contractor be careful when pulling the auger to remove cuttings from the excavation. A slightly oversized casing is sometimes used to reduce chances for snagging and lifting the casing during cleanout operations.

In addition, the following installation techniques will aid in the successful construction of drilled shafts.

- The clear spacing between rebar or behind the rebar cage should be at least three times the maximum size of the coarse aggregate.
- Centralizers on the rebar cage should be installed to keep the cage properly positioned.
- Cross bracing of a reinforcing cage may be used when fabricating, transporting, and lifting. However, experience has shown that cross bracing can contribute to the development of voids in a concrete shaft. Therefore, we recommend removing the cross bracing prior to lowering the reinforcing cage into the open shaft.
- Concrete should have a slump of 6 inches ±1 inch, and it should be placed from the bottom of the shaft using a tremie pipe or chute, using proper tremie concrete placement procedures. The concrete should be designed to achieve the required strength at the higher slumps indicated.

10.5.2 Use of Slurry. Concrete tremied into a shaft with slurry should always maintain a hydrostatic pressure greater than either the surrounding water level or slurry in the excavation. The slurry should have a marsh funnel viscosity of between 30 to 40 seconds for a 500 milliliter funnel. The slurry should have a specific gravity between 1.02 (8.5 lb/gal.) and 1.15 (9.6 lb/gal.) at the time of concrete placement. In addition, the sand content in the slurry should be less than

[5] Reese, L.C. and O'Neill, M.W. (1988), Drilled Shafts: Construction Procedures and Design Methods, Federal Highway Administration, Publication No. FHWA-HI-88-042.

10 percent. If casing is used, a sufficient head of concrete that fills the casing is required before pulling the casing.

10.5.3 Scheduling and Sequencing. The successful completion of drilled shaft excavations will depend largely on the suitably of the drilling equipment and skill of the operator. The drilled foundation Contractor should try to reduce the time the excavation is open. The Contractor should schedule the sequence of operations so each excavation can be completed, reinforcing steel placed, and the concrete poured in a continuous, rapid, and orderly manner. The Contractor should not place shafts adjacent to each other until the first shafts sets. Drilled shafts spaced closer than about 6 feet (clear spacing) should be placed at least 6 hours apart and preferably on alternate days. A drilled shaft excavation should ***not*** be left open overnight.

The drilled shaft installation should be continually observed by the Geotechnical Engineer-of-Record or their qualified representative to verify that, among other things: 1) subsurface conditions are as anticipated from the soil borings; 2) the shafts are constructed to the proper diameter and penetration; 3) drilled shafts are within allowable tolerances for plumbness; 4) reinforcing is properly placed; and 5) a tremie is used for proper concrete placement. These critical items are fundamental to the proper installation and behavior of the drilled shafts in accordance with our recommendations.

10.6 Sonic Integrity Logging

A non-destructive technique currently available to observe completed ACIP piles and drilled shafts is the Sonic Integrity Logger. Consideration should be given to including sonic integrity logging as part of the installation procedures for ACIP piles and drilled shafts. We believe it is essential that a qualified Geotechnical Engineer observe the installation of ACIP piles and drilled shafts using sonic integrity logging to provide a higher degree of confidence that the piles/shafts are properly installed. Defects such as caving, necking, or soil inclusions in the pile/shaft can be detected using sonic integrity logging. Use of the Sonic Integrity Logger at the start of construction to evaluate alternate installation procedures and to help establish an acceptable production pile installation procedure is particularly beneficial. We can provide literature and/or a demonstration of the sonic integrity logging equipment if requested.

11.0 GEOTECHNICAL DESIGN REVIEW AND CONSTRUCTION MONITORING

We should be given the opportunity to review the drawings and specifications to verify that our design recommendations were adhered to and interpreted as we had intended. We recommend that the Geotechnical Engineer-of-Record, or their qualified representative, be present on-site during site preparation, earthwork, and construction to observe and monitor construction activities. Construction monitoring performed by qualified personnel independent of the Contractor is recommended because the performance of foundations and other structures constructed during this project will be directly related to the Contractor's adherence to the recommendations presented in this report and to the specifications prepared by the Designer.

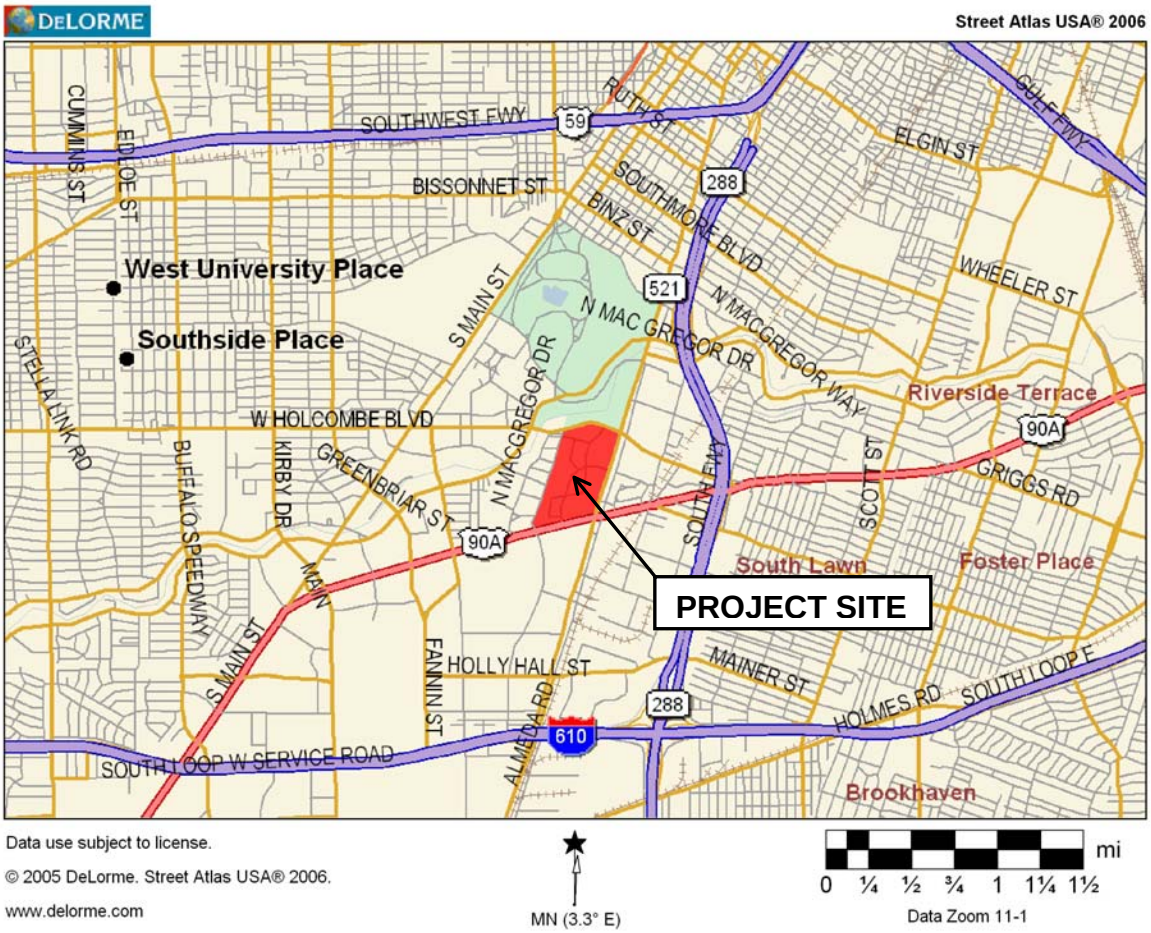
Additionally, the geotechnical aspects of construction contain inherent uncertainties that can result in unanticipated soil and/or groundwater conditions being encountered during construction, **especially with respect to the presence, depth, consistency, and extent of fill material across the site and the presence of buried structures from previous site usage.** Qualified geotechnical personnel observing onsite construction can monitor construction activities and may aid in recognizing unanticipated subsurface conditions and reconciling these conditions with design recommendations.

Construction monitoring should be performed during:

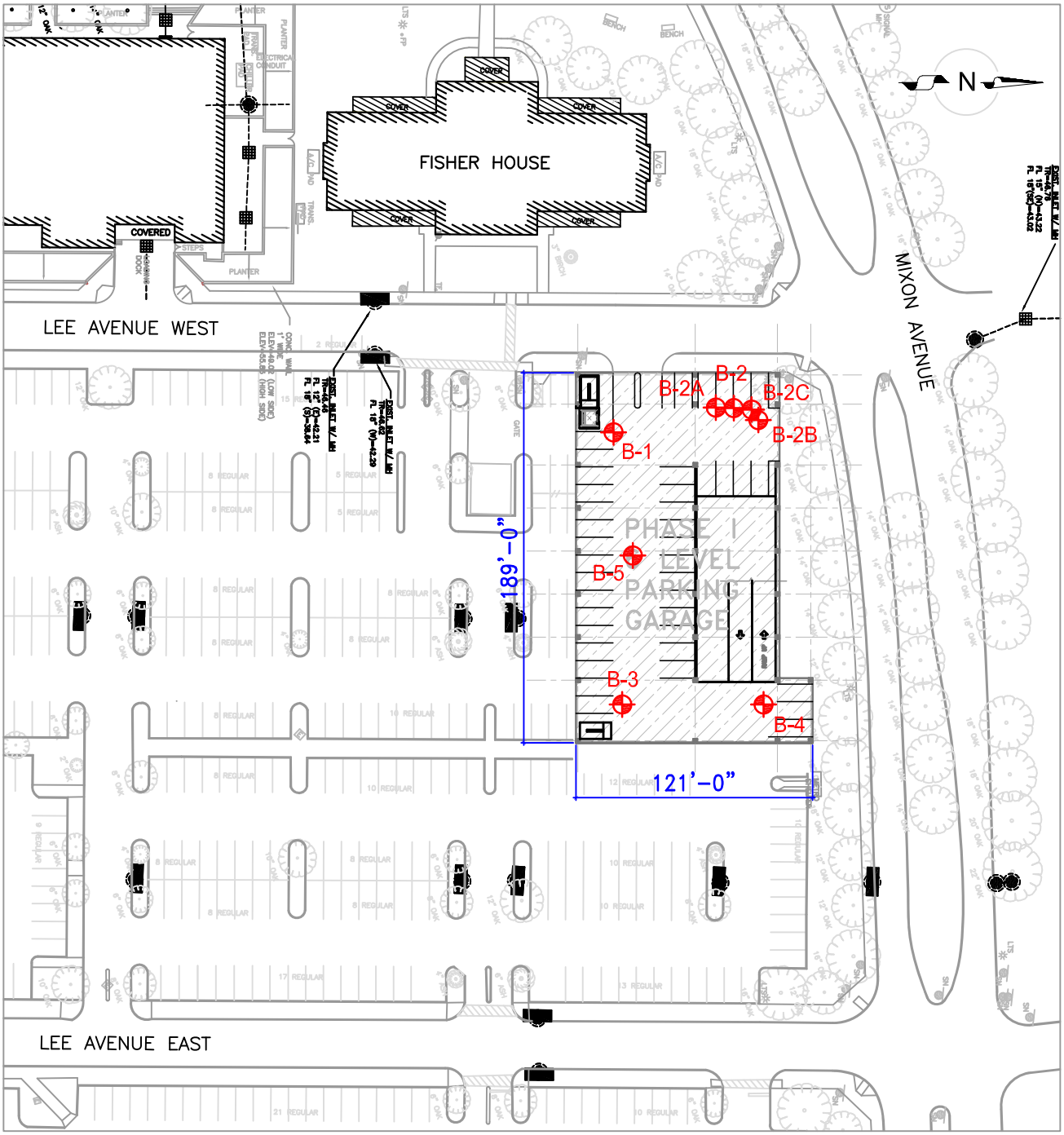
- Site Preparation and Site Excavation: The site preparation activities, including site excavations, should be monitored and the suitability of the exposed subgrade should be observed.
- Foundation Installation: The installation of foundations should be monitored to verify the suitability of the exposed foundation soils, the stability of the foundation excavation, and the placement of foundation concrete. This includes observing load tests of deep foundations.
- Placement of Fill: Activities regarding the placement of fill should be monitored to verify that the material being used as fill complies with the recommendations in this report and with the project specifications. This includes performing compaction tests, classification tests, and field density tests.

We recommend that we be retained during the site preparation, earthwork, and foundation excavation and construction phases to provide material testing and construction surveillance to: (1) observe compliance with the design concepts, specifications, and recommendations; (2) observe subsurface conditions during construction; and (3) perform quality control tests.

ILLUSTRATIONS



VICINITY MAP
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS



LEGEND

B-1 GEOTECHNICAL BORING

0 80 160 FEET

NOTES:

1. BORING LOCATIONS ARE APPROXIMATE.
2. BASE DRAWING PROVIDED BY HOK.

PLAN OF BORINGS

PARKING GARAGE

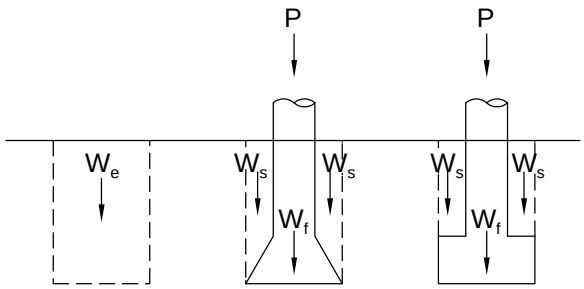
DEBAKEY VA MEDICAL CENTER

HOUSTON, TEXAS

FOUNDATION DESIGN CRITERIA

A properly-sized foundation must satisfy the two following criteria with respect to the supporting soil.

1. For soil strength. The bearing pressure created on the base of the foundation by the maximum design load must be less than that which would cause shear failure in the soil. A factor of safety of 2 or more with respect to the soil shear strength is generally used.
2. For soil compressibility. The bearing pressure created on the base of the foundation by the sustained load must not produce sufficient consolidation in the underlying soil to result in foundation settlement that is detrimental to the safety or utility of the structure.



SYMBOLS

TERMS AND SYMBOLS

P = Column load (subscript can be used to denote character of load: P_s = sustained load, P_n = normal operating load, P_m = maximum design load).

W_e = Weight of soil located above base of foundation excavation and lowest adjacent grade.*

W_s = Weight of soil located above foundation.*

W_f = Weight of foundation.*

A = Area of base of foundation.

p = Average bearing pressure acting on soil (subsript can be used to correspond to column load: P_s, P_n, P_m).

* Position of groundwater level must be considered in determining weights. Effective, or buoyant, unit weights should be used below the highest expected groundwater level.

BEARING PRESSURES

Gross Bearing Pressure, p, for any column load is the total pressure acting on the base of the foundation.

$$p = 1/A\ (P + W_s + W_f)$$

Net Bearing Pressure, p', for any column load is the difference between the gross bearing pressure acting on the base of the foundation and the soil pressure existing at that elevation from the lowest overlying or adjacent soils.

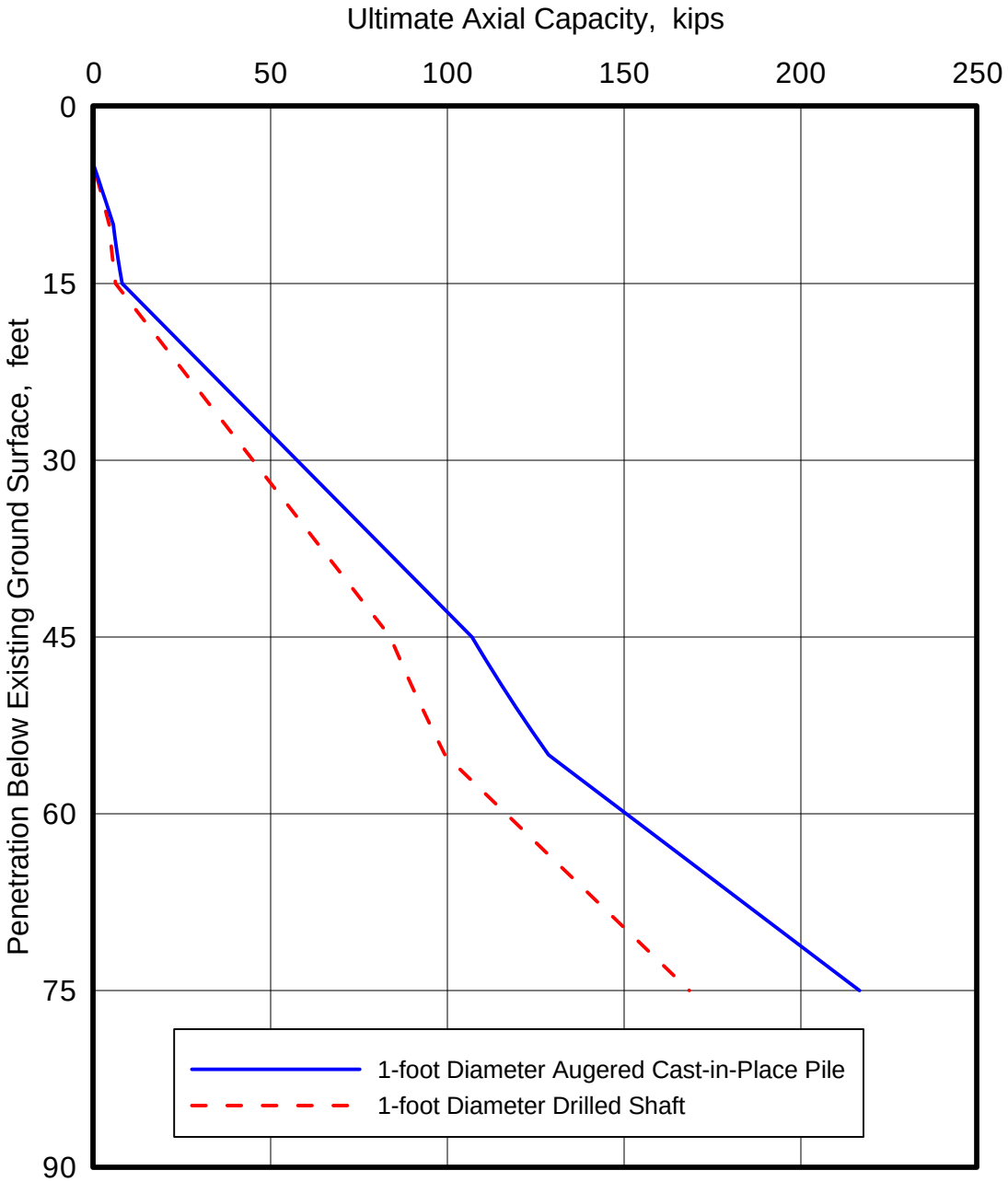
$$p' = 1/A\ (P + W_s + W_f - W_e)$$

For analysis with regard to the first design criterion, soil strength, the column load in the above equations should usually be the maximum design load, P_m. Occasionally, the normal operating load, P_n, may also be used. If footing is loaded eccentrically, the increase in edge bearing pressure due to the eccentricity should be computed in the usual manner.

For analysis with regard to the second design criterion, soil compressibility, the column load in the above equations should be the sustained load, P_s. This load is the dead load plus the sustain live load.

For further references, see pp. 506 - 512, "Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck (2nd edition); and pp. 564 - 565, "Fundamentals of Soil Mechanics" by Donald W. Taylor.

COMPUTATION OF BEARING PRESSURES
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS



Notes:

1. These curves represent ***ultimate*** values of an augered cast-in-place (ACIP) pile and drilled shaft for compression without end bearing and tension. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile/shaft. Group effects are discussed in the report text.
3. To obtain capacity for a given size pile/shaft, multiply values from the curve by the pile/shaft diameter, in feet.
4. Tension capacity should be limited to depth of steel reinforcement that can develop proposed tensile load.

ULTIMATE AXIAL CAPACITY CURVES
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

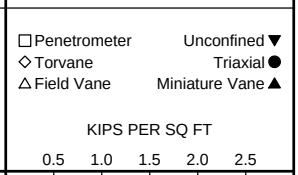
APPENDIX A – BORING LOGS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					(6") CONCRETE	0.5						
					FILL: SAND, gray, possibly stablized	2.0						
					FILL: CLAY, stiff, dark gray, with sand pockets	4.0						
					CLAY, stiff, dark gray		89		31	73	17	56
					- olive gray, 7' to 8'							
					- greenish gray and red, 8' to 11'							
10					- with calcareous and ferrous nodules below 9'		107		18	49	14	35
					- red and gray below 11'	12.0						
				16	SILTY SAND, fine-grained, medium dense, tan and gray, with clay seams and calcareous nodules			47				
				22	- brown and gray below 15'	18.0						
					CLAY, stiff to very stiff, red							
20												
					- brown and gray, 23' to 28'							
					- with calcareous nodules, 23' to 35'		106		21			
					- reddish brown, 28' to 33'							
30												
					- gray and brown, 33' to 38'				27			
					- red, 38' to 43'							
40												
					- firm to stiff, 42' to 58'							
					- brown, 43' to 48'							
					- with fine sand pockets, 34' to 53'		108	81	20			
					SANDY CLAY, firm, greenish gray	48.0						
50												
					- greenish gray and red, with sand seams, 53' to 58'							
					CLAY, very stiff, red, with siltstone nodules	58.0			26			
60						60.0						

CLASSIFICATION

SHEAR STRENGTH



NOTES:
1. Free water not encountered during drilling.
2. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 15, 2011
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 25'
WET ROTARY: 25' to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

R:\04120\2010 PROJECTS\0001-0099\0412-10-0060\DRAFTING\04.12100060.GPJ FUGRO_SO (LAB DATA)_INDUSTRIAL GROUP 2/11/2011

LOG OF BORING NO. B-1
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					(5 1/2") CONCRETE	0.5							
					FILL: SAND, gray, possibly stabilized	1.0							
					FILL: CLAY, very stiff, brown and gray - dark gray below 2' - stiff below 3' - with shell fragments below 4'				16				4.5 □
					(REFUSAL at 6')	6.0							
10													
20													
30													
40													
50													
60													

CLASSIFICATION

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Field Vane — Unconfined ▼ Triaxial ● Miniature Vane ▲

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5

NOTES:
1. Free water not encountered during drilling.
2. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 15, 2011
TOTAL DEPTH: 6'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 6'
WET ROTARY: Not Applicable
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

R:\04120\2010 PROJECTS\0001-0099\0412-10-0060\DRAFTING\04.12100060.GPJ FUGRO_SO (LAB DATA)_INDUSTRIAL GROUP 2/10/2011

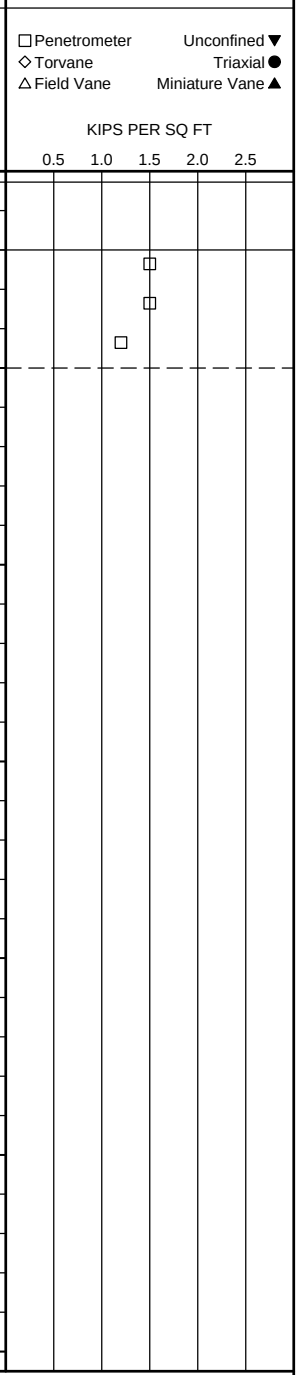
LOG OF BORING NO. B-2
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					(6 5/8") CONCRETE	0.6						
					FILL: SAND, fine-grained, gray, with clay, possibly stabilized	4.0			15			
					FILL: CLAY, stiff, dark gray, gray and tan				18			
10					(REFUSAL at 10')	10.0						
20												
30												
40												
50												
60												

CLASSIFICATION

SHEAR STRENGTH



NOTES:
1. Free water not encountered during drilling.
2. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 15, 2011
TOTAL DEPTH: 10'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 10'
WET ROTARY: Not Applicable
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

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LOG OF BORING NO. B-2A
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					(6 1/2") CONCRETE	0.5						
					FILL: SAND, gray, possibly stabilized	2.0						
					FILL: CLAY, stiff to very stiff, dark gray	4.0	94		28	76	17	59
					CLAY, firm to stiff, olive gray, with silt pockets							
							85	90	36	94	20	74
					- tan, 8' to 10'							
10					- with ferrous nodules below 8'							
					- brown and gray, with calcareous nodules below 10'	12.0						
					CLAYEY SAND, stiff to very stiff, brown and gray	15.0		39				
					CLAY, stiff, reddish brown, with sand pockets							
					- red, slickensided, 18' to 23'							
20												
					- gray and tan, 23' to 28'							
					- with silt pockets, 23' to 38'							
					- stiff to very stiff, 28' to 40'							
30							111		19	49	12	37
					- red, 38' to 43'							
40					- firm to stiff below 40'							
					- greenish gray below 43'							
					CLAYEY SAND, fine-grained, very loose, gray	49.0		38				
50				3	- with clay seams below 54'			25				
					CLAY, very stiff, red	58.0						
60						60.0						

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Field Vane — Unconfined ▼ Triaxial ● Miniature Vane ▲

KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

NOTES:
1. Free water not encountered during drilling.
2. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 17, 2011
TOTAL DEPTH: 60'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 25'
WET ROTARY: 25' to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

R:\04120\2010 PROJECTS\0001-0099\0412-10-0060\DRAFTING\04.12100060.GPJ FUGRO_SO (LAB DATA)_INDUSTRIAL GROUP 2/10/2011

LOG OF BORING NO. B-3
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					(7.5") CONCRETE	0.6						
					FILL: SAND, gray, possibly stabilized	2.0						
					FILL: CLAY, stiff, dark gray	4.0		52	35	91	22	69
					CLAY, stiff, greenish gray - firm to stiff, 6' to 10' - tan, with calcareous nodules and ferrous nodules, 6' to 8' - reddish brown below 8' - stiff to very stiff below 10'		83 96		39 29			
				15	SANDY CLAY, stiff, tan	13.0		53				
				7	CLAY, firm, reddish brown - stiff, tan and gray, with sand pockets below 23'	18.5						
					SANDY CLAY, stiff, gray and tan	28.0			25			
					CLAY, stiff, brown - with silt pockets and silt seams below 39'	34.0			28	58	15	43
				19	SANDY CLAY, very stiff, gray and tan - gray below 49'	43.0		60				
				18	CLAY, very stiff, gray and brown, with silt pockets and siltstone nodules	53.5						
60						60.0	101		26			

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Field Vane — Unconfined ▼ Triaxial ● Miniature Vane ▲

KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

NOTES:
1. Free water not encountered during drilling.
2. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 16, 2011
TOTAL DEPTH: 60'
CAVED DEPTH: 16'
DRY AUGER: Surface to 16'
WET ROTARY: 16' to 60'
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

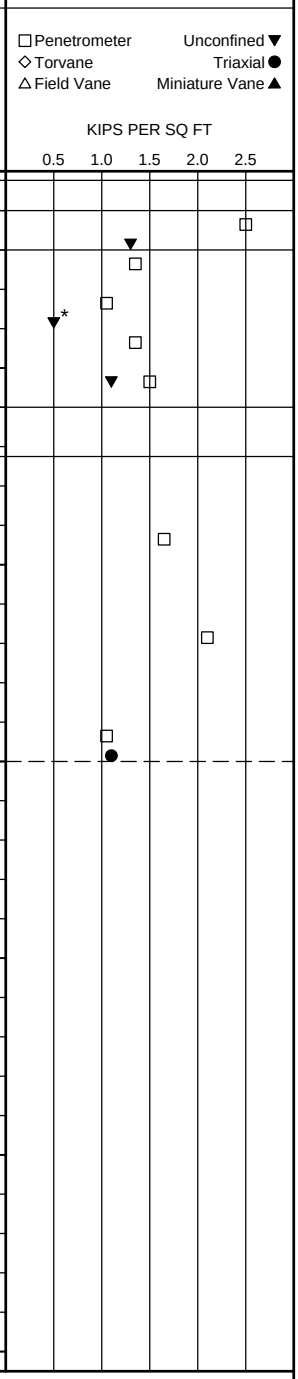
R:\04120\2010 PROJECTS\0001-0099\0412-10-0060\DRAFTING\04.12100060.GPJ FUGRO_SO (LAB DATA)_INDUSTRIAL GROUP 2/11/2011

LOG OF BORING NO. B-4
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

LOCATION: See Plate 2
COORDINATES: Not Available
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					(5 1/2") CONCRETE	0.5						
					FILL: CLAYEY SAND, fine-grained, dark gray, possibly stabilized	2.0						
					FILL: CLAY, stiff to very stiff, dark gray	4.0	89		33			
					CLAY, stiff, dark gray, with organic material, slickensided							
					- tan and gray, with ferrous nodules, 6' to 8'		85	88	36	83	20	63
					- brown, with calcareous nodules, 8' to 10'							
10					- red and gray below 10'		96		26			
				8	SILT, loose, brown, with sand pockets	12.0		83				
				7	CLAY, stiff to very stiff, red, with silt pockets and seams	14.5						
					- brown and gray, 18' to 23'				21			
20												
					- red and gray, with calcareous nodules, 23' to 28'							
					- stiff, brown and gray, with sand pockets below 28'							
30						30.0	105		21			
40												
50												
60												

SHEAR STRENGTH



NOTES:
1. Free water not encountered during drilling.
2. * Test was influenced by secondary structure.
3. Terms and symbols defined on Plates A-7a and A-7b.

DATE: January 16, 2011
TOTAL DEPTH: 30'
CAVED DEPTH: Not Applicable
DRY AUGER: Surface to 30'
WET ROTARY: Not Applicable
BACKFILL: Cement-Bentonite Grout
LOGGER: E. Schulak

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LOG OF BORING NO. B-5
PARKING GARAGE
DEBAKEY VA MEDICAL CENTER
HOUSTON, TEXAS

SOIL TYPES

Sand
Silt
Clay
Gravel
Silty Sand
Sandy Silt
Sandy Clay
Peat or Highly Organic
Clayey Sand
Clayey Silt
Silty Clay
Debris or Mixed Fill
Asphalt
Concrete

SAMPLER TYPES

Thin-walled Tube
Partial Recovery w/ Tube
Auger
Split-barrel
No Recovery
Pitcher
Piston
Geoprobe
Rock Core

SOIL GRAIN SIZE

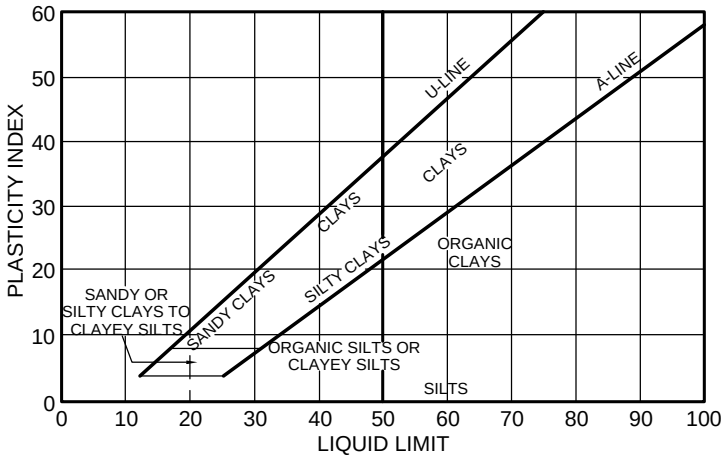
U.S. Standard Sieve

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

6" 3" 3/4" 4 10 40 200

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002 (mm)

PLASTICITY CHART



SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (1 of 2)

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

DENSITY OF GRANULAR SOILS

Descriptive Term	*Relative Density, %	**Blows Per Foot (SPT)
Very Loose	< 15	0 to 4
Loose	15 to 35	5 to 10
Medium Dense	35 to 65	11 to 30
Dense	65 to 85	31 to 50
Very Dense	> 85	> 50

*Estimated from sampler driving record.
**Requires correction for depth, groundwater level, and grain size.

STRENGTH OF COHESIVE SOILS

Term	Undrained Shear Strength, ksf	Blows Per Foot (SPT) (approximate)
Very Soft	< 0.25	0 to 2
Soft	0.25 to 0.50	2 to 4
Firm	0.50 to 1.00	4 to 8
Stiff	1.00 to 2.00	8 to 16
Very Stiff	2.00 to 4.00	16 to 32
Hard	> 4.00	> 32

SHEAR STRENGTH TEST METHOD

U - Unconfined Q = Unconsolidated - Undrained Triaxial

P = Pocket Penetrometer T = Torvane V = Miniature Vane F = Field Vane

HAND PENETROMETER CORRECTION

Our experience has shown that the hand penetrometer generally overestimates the in-situ undrained shear strength of over consolidated Pleistocene Gulf Coast clays. These strengths are partially controlled by the presence of macroscopic soil defects such as slickensides, which generally do not influence smaller scale tests like the hand penetrometer. Based on our experience, we have adjusted these field estimates of the undrained shear strength of natural, overconsolidated Pleistocene Gulf Coast soils by multiplying the measured penetrometer reading by a factor of 0.6. These adjusted strength estimates are recorded in the "Shear Strength" column on the boring logs. Except as described in the text, we have not adjusted estimates of the undrained shear strength for projects located outside of the Pleistocene Gulf Coast formations.

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

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (2 of 2)