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GEOTECHNICAL EXPLORATION

Proposed SixTen Franklin Building
610 Franklin Street
Tampa, Florida

UES Project No. 80732-003-01

Prepared For:

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May 31, 2006

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Consultants in: Geotechnical Engineering • Environmental Sciences
Construction Materials Testing • Threshold Inspection • Private Provider Inspection

May 31, 2006

David E. Thompson
Wood Partners
701 South Howard Avenue, Suite 106
Tampa, Florida, 33606

Reference: Geotechnical Exploration
Proposed SixTen Franklin Building
610 Franklin Street
Tampa, Florida
UES Project No. 80732-003-01

Dear Mr. Thompson:

Universal Engineering Sciences, Inc. (UES) has completed a geotechnical exploration on the above-referenced site in Tampa, Florida. Our scope of services was in general accordance with UES Proposal #06-075, dated February 23, 2006, and authorized by you on February 28, 2006.

This report contains the results of our study, an engineering interpretation of the subsurface data obtained with respect to the project characteristics described to us, recommendations for foundation and pavement design, and general construction and site preparation considerations.

We appreciate the opportunity to have worked with you on this project and look forward to a continued association with Wood Partners. Please do not hesitate to contact us if you should have any questions, or if we may further assist you as your plans proceed.

Respectfully submitted,

UNIVERSAL ENGINEERING SCIENCES, INC.
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1.0 INTRODUCTION

1.1 GENERAL

In this report we present the results of our geotechnical exploration on the site of the proposed SixTen Franklin Building, located at 610 Franklin Street, in Tampa, Hillsborough County, Florida. Our scope of services was in general accordance with UES Proposal #06-075, dated February 23, 2006, and authorized by you on February 28, 2006. However, the proposed H-pile testing was not completed since the demolition contractor removed all H-piles prior to our mobilization to the site. Also, as the limestone underlying the site was found to be more poorly indurated with increasing depth, our borings were extended to greater depths. This change was authorized by you via email on April 28, 2006.

1.2 PROJECT DESCRIPTION

We understand that the project involves redeveloping the site with a new 33-story high-rise, multi-use building that will encompass the entire block. A site plan was provided by WilsonMiller, and structural load estimates were provided by Echelon Engineering.

Based on information provided by Rafael Machado, P.E., of Echelon Engineering, the maximum column loads on the tower footprint will be approximately 4000 kips. Typical interior column loads will be on the order of 2500 kips, and typical exterior column loads will be about 1500 kips. Column loads for the parking deck area will range from 500 to 1200 kips.

As no grading plan was available at this time, we have assumed construction will proceed on existing grade (estimated to be on the order of +15 feet NGVD).

Our geotechnical recommendations are based upon the above assumptions and considerations. If any of this information is incorrect or if you anticipate any changes, please inform Universal Engineering Sciences so that we may review our recommendations, and make revisions as needed.

A general location map of the project area appears in Appendix A: Site Location Map. Also included in Appendix A for your reference are a Site Aerial Photograph, Site Topographic Map and SCS Soil Survey Map.

2.0 PURPOSE AND METHODOLOGIES

2.1 PURPOSE

The purpose of our services was:

- to explore the general subsurface conditions at the site using SPT borings and rock coring;
- to interpret and review the subsurface conditions with respect to the proposed construction as described to us; and

- to provide geotechnical engineering information for foundation and pavement design, and to provide general recommendations for site preparation.

This report presents an evaluation of site conditions on the basis of traditional geotechnical procedures for site characterization. The recovered samples were not examined, either visually or analytically, for chemical composition or environmental hazards.

2.2 FIELD EXPLORATION

The subsurface conditions across the site were explored with a total of eight (8) borings advanced to depths of 100 to 200 feet below the existing ground surface (bgs). These borings were advanced using the rotary wash method, and samples were collected while performing the Standard Penetration Test (SPT) at regular intervals.

We performed the SPT test in general accordance with ASTM D-1586 guidelines. In general, a standard split-barrel sampler (split-spoon) is driven into the soil using a 140-pound hammer free-falling 30 inches. The number of hammer blows required to drive the sampler 12 inches, after first seating it 6 inches, is designated the penetration resistance, or SPT-N value. This value is used as an index to soil strength and consistency.

In addition to the SPT sampling, we also collected core samples of the limestone bedrock underlying the site. Rock coring was completed using a 3-inch I.D., double wall core barrel having a length of 5 feet. The penetration rate (in minutes per foot), down pressure (in psi), percent recovery (REC) and rock quality designation (RQD) were recorded for each 5 foot core run.

Consider the indicated locations, elevations and depths to be approximate. Our drilling crew located the borings based upon estimated distances and taped measurements from existing site features. Top-of-boring elevations, if listed, were estimated from published USGS topographic contours. If more precise location and elevation data are desired, a registered professional land surveyor should be retained to locate the borings and determine their ground surface elevations. The Boring Location Plan is presented in Appendix B.

Soil, rock, water, and/or other samples obtained from the project site are the property of UES. Unless other arrangements are agreed upon in writing, UES will store such samples for no more than 60 calendar days from the date UES issued the first document that includes the data obtained from these samples. After that date, UES will dispose of all samples.

2.3 LABORATORY TESTING

The soil samples recovered from the test borings were returned to our laboratory and visually classified by our geotechnical staff under the direct supervision of our Senior Geotechnical Engineer. Our engineer then selected representative soil samples for index properties testing to aid in the engineering classification of these materials. The results of this testing are presented on the Report of Soil Borings in Appendix B. Grain size distribution curves and Atterberg limits test results are also included on separate summary sheets in Appendix B.

All of the rock cores that were collected were visually inspected by our engineer and photo-documented (refer to Appendix C). REC and RQD were verified, and samples were then chosen for testing to determine the unconfined compressive strength (UC) (ASTM D-2938) and splitting tensile strength (ST) (ASTM D-3967) of the specimens. The results of this testing were then statistically analyzed according to accepted practice and used to calculate the unit skin friction (f_{su}) and unit end bearing (q_{tu}) of the limestone stratum.

2.3.1 ROCK CORE PARAMETERS

All rock core runs utilized a core barrel having a length of 60 inches. Percent recovery (REC) refers to the fraction of the core barrel, expressed as a percent, that contained sample after a run. Rock quality designation (RQD) is a modified recovery parameter which only considers in the recovery calculation the length of in-tact rock pieces 4 inches or more in length (ASTM D-6032). RQD generally gives a better indicator of rock quality as it discounts very clayey or very brittle rock which cannot maintain its integrity during the sampling process.

Mathematically, these are shown in Equations (1) and (2) below.

$$REC = \frac{Sample\ Length}{60\ inches} \times 100\% \tag{1}$$

$$RQD = \frac{\sum\ Sample\ Lengths \geq 4\ inches}{60\ inches} \times 100\% \tag{2}$$

The RQD results can then be used to qualitatively describe the limestone as shown in Table 1.

TABLE 1
ROCK QUALITY DESIGNATION

RQD (%)	Classification of Rock Quality
0 to 25	Very Poor
25 to 50	Poor
50 to 75	Fair
75 to 90	Good
90 to 100	Excellent

Unit skin friction (f_{su}) and unit end bearing (q_{tu}) refer to the ultimate stresses that can develop between the foundation element and the soil/rock stratum in side shear and point bearing, respectively. These values are computed from correlations to SPT-N values and also from data analysis of rock core testing for unconfined compressive and splitting tensile strengths. As these are ultimate values, an appropriate safety factor must be used for design. Also, full-scale, instrumented foundation load tests are usually employed to verify these values for a given site. Adjustments are then made to the final design values as dictated by the results of the load testing.

3.0 FINDINGS

3.1 SURFACE CONDITIONS

At the start of our geotechnical exploration, we reviewed aerial photographs available from the Hillsborough County Property Appraiser's office and TerraServer USA, USGS topographic quadrangle maps, and the USDA Soil Conservation Service (SCS) Soil Survey of Hillsborough County for relevant information about the site. According to USGS topographic information, the elevation across the approximately 1.9 acre parcel is on the order of +15 feet NGVD. Although several structures appear in the Aerial Photograph, these had been razed and the site rough graded by the time our field services began. The Hillsborough River is located within 0.25 miles of the site, to the west.

3.2 SUBSURFACE CONDITIONS

3.2.1 SOIL SURVEY

According to SCS, the entire 1.9 acre site is classified as “Urban Land (#56).” Soils in this group have been reworked or covered with pavement and/or structures such that native soil conditions are no longer present or attainable. Uniquely mapped soil properties are therefore unavailable, and subgrade conditions on sites so classified are often highly variable.

3.2.2 SOIL BORINGS

The boring locations and detailed subsurface conditions are illustrated in Appendix B: Boring Location Plan and Report of Soil Borings (Logs). The classifications and descriptions shown on the logs are based upon visual characterizations of the recovered soil samples. The general subsurface soil profile on the site, based on the soil boring information, is described below. For more detailed information, please refer to the logs.

The subsurface stratigraphy generally contains four primary strata, although the sequence of these strata is somewhat variable. These are sand and/or fill, very hard limestone, medium stiff to hard clay and soft to moderately hard limestone, or “intermediate geomaterial” (IGM*).

> **As referenced herein, IGM refers to poorly indurated, clayey limestone. SPT-N values are typically between 10 and 75. REC can be high for IGM but RQD is normally very low, therefore reduced strength parameters (f_{su} & q_{tu}) are used.*

Beginning at the ground surface, there is a surficial layer of mixed sand and silt (SP, SP-SM) that extends to an approximate average depth of 25 feet. The shallow portion of this material contains miscellaneous construction debris left from the razing of the structures previously occupying the site (brick, concrete, metal, wood). In some locations, imported sand fill was encountered in the borings, most likely the result of backfilling operations during demolition.

This upper sandy soil zone, which is typically in a very loose to medium dense state, grades to a very hard, shallow limestone layer at five of the eight boring locations at depths of 20 to 30 feet. This shallow limestone layer was very hard to drill and resulted in the highest REC and RQD values of any of the rock recovered on the site. However, this zone is typically only 5 to 10 feet thick and is underlain by a medium stiff to hard, cemented marine clay CL/CH. In three of the eight borings the cemented marine clay was contacted directly beneath the surficial sands.

The cemented clay layer is also hard to drill, and is often calcareous. In several borings, this material offered “refusal” conditions to our SPT testing, and little to no sample was recovered. In some borings, however, the consistency of this material decreases with increasing depth until “weight of hammer” conditions are encountered just above the limestone surface.

The clay layer generally grades back into the hard to very hard limestone stratum at depths of approximately 45 feet to 70 feet (average ~50 feet). This portion of the limestone, however, was found to contain intermittent pockets/lenses of cemented clay (CH). Drilling resistance within this portion of the stratum varies from easy to hard, and drilling fluid circulation was lost and recovered sporadically. Portions of this stratum offered very little resistance to SPT testing, while at other depths refusal conditions were recorded.

Generally, the quality of the limestone decreases with increasing depth. The material below depths of approximately 100 feet to 140 feet are more appropriately classified as intermediate geomaterial (IGM), the fourth stratum identified.

The shallow water table was encountered approximately 7 feet below existing grade at Boring B-02, which is the only boring where the water table was measured. Daily and seasonal fluctuation is possible.

The boring logs and related information included in this report are indicators of subsurface conditions only at the specific locations and times noted. Subsurface conditions at other locations on the site, including groundwater levels and the presence of deleterious materials, may differ significantly from conditions which in the opinion of UES exist at the sampling locations. Note, too, that the passage of time may affect conditions at the sampling locations.

3.2.3 ROCK CORE TESTING

Rock cores were collected from borings/depths where material was able to be cored. The recovered rock core samples were tested for unconfined compressive (UC) strength (13 tests total) and splitting tensile (ST) strength (32 tests total), in order to better ascertain the ultimate unit side shear (skin friction) and end bearing values necessary for rock-socketed deep foundation design.

Based on a statistical analysis of the rock core test results, we have calculated the average ultimate unit side shear (f_{su}) of the limestone to be 12 tons per square foot (tsf). Within the IGM stratum, the average f_{su} reduces to 5 tsf. Ultimate unit end bearing (q_{tu}), on average, is estimated at 30 tsf for the limestone and 20 tsf for the IGM. Since end bearing will not fully mobilize concurrently with skin friction within structurally tolerable vertical displacements, a reduction of these values is used for design purposes.

4.0 RECOMMENDATIONS

4.1 GENERAL

In this section of the report we present our recommendations for building foundation and pavement design, general site preparation, and construction related services. These recommendations are made based upon a review of the attached soil test data, our understanding of the proposed construction as it was described to us, and our stated assumptions. If the grading plans or the site layout differ from those assumed or described to us, we should be retained to review the new or updated information and amend our recommendations with respect to those changes. Additionally, if subsurface conditions are encountered during construction that were not encountered in the test borings, report those conditions immediately to us for observation and recommendations.

We identified the following two (2) primary geotechnical considerations as part of our study:

- The thin, shallow stratum of very hard limestone typically encountered at a depth of 25 feet across the site (at 5 of 8 boring locations) will limit the available foundation options for this project. Driven piles and/or augercast piles will likely hit refusal before significant axial or lateral capacities are developed.

- The variable, very loose to loose nature of the overburden materials atop the shallow limestone stratum will require that the chosen deep foundation alternative be designed to develop the majority of the necessary capacity within the limestone, primarily in side shear. Foundation elements not extended sufficiently into the limestone stratum to develop a substantial side shear interface will not be geotechnically sound given the reported magnitude of the structural loads.

4.2 GROUNDWATER

Based upon our visual inspection of the recovered soil samples, review of information obtained from SWFWMD and the USDA Soil Survey of Hillsborough County, and our knowledge of local and regional hydrogeology, our best estimate is that the seasonal high groundwater level could be on the order of 5 feet below the existing grade at the test boring locations. Artificial drainage around this urban site should greatly influence the water table depth.

Several factors influence the determination of the seasonal high water table (SHWT). When soils are subjected to alternating cycles of saturation and drying, discoloration or staining that is not part of the dominant soil color occurs. This is called mottling, and manifests itself in various shades of gray, brown, red or yellow. There are numerous processes that lead to this discoloration, including mineral accretions, oxidation, and bacteria growth within the soil. The presence of this discoloration indicates that groundwater has, at some point in time, reached that elevation and remained there long enough to cause any or all of these processes to occur. The SHWT elevation is assumed to be the highest point at which mottling is observed, regardless of whether water is present at the time of observation. This estimate is independent of the actual location of the groundwater table.

It should be noted that the estimated SHWT does not provide any assurance that groundwater levels will not exceed this level in the future. Should impediments to surface water drainage exist on the site, or should rainfall intensity and duration exceed the normally anticipated amounts, groundwater levels may exceed our seasonal high estimate. Also, future development around the site could alter surface runoff and drainage characteristics, and cause our seasonal high estimate to be exceeded. We therefore recommend positive drainage be established and maintained on the site during construction. Further, we recommend permanent measures be constructed to maintain positive drainage from the site throughout the life of the project.

Temporary dewatering may be required on this site if construction proceeds during the wet season, particularly if deep excavations are necessary (>5 feet).

4.3 BUILDING FOUNDATIONS

4.3.1 EVALUATED OPTIONS

We evaluated the following deep foundation alternatives for use on this project:

- Driven piles, both precast concrete and steel
- Augered cast-in-place piles (augercasts)
- Conventional drilled shafts (drilled caissons)

It is our opinion that driven piles are not appropriate for use on this project primarily because they may not be able to deeply penetrate the limestone stratum. This will result in mainly end bearing-only foundation elements. Although axial capacity could be large, undesirably low lateral and uplift pile capacities will be realized. Further complication arises from areas of the site were precast piles may penetrate a stiff/hard layer and encounter a soft layer below. Tension stresses during driving will become difficult to control, and damaged piles are likely. Finally, the noise and vibration associated with driven pile installation may be objectionable to surrounding property owners in this densely populated urban area. ***For these reasons, we do not recommend the use of driven piles on this project.***

Augercast piles are not a viable option on this site for several reasons. From a geotechnical perspective, augercast piles will likely hit early refusal atop the shallow hard to very hard limestone on this site. No practical embedment into this limestone will be attainable with an augercast drill rig. This will result in end-bearing only augercast piles, which is an undesirable design for augercast piles. Since there is no control over the conditions at the toe of augercast pile excavations (diameter, thickness of sediment, firmness of bearing level soils, etc.), and also because there is always a high degree of variability in the quality of the finished piles, proper use of these foundation elements necessitates skin-friction only axial design.

The feasibility of using augercast piles is further reduced on this site as the anticipated magnitude of the column loads compared to the allowable capacity of even the most well-constructed augercast pile will require a very large quantity of piles to be installed. This problem is again amplified by the high variability of augercasts and the difficulty of quality assurance during pile construction, both of which necessitate the use of a larger factor of safety as compared to drilled shafts or driven piles. Typically, a minimum factor of safety of 3 is required for augercast piles, resulting in a highly inefficient design for the magnitude of loads anticipated. Finally, lateral pile capacity is very low for augercast piles as compared to other deep foundation alternatives. ***For these reasons we do not recommend the use of augercast piles on this project.***

Conventional drilled shafts (a.k.a. “drilled caissons”, “drilled piers”), in our opinion, are ideal for use on this site for several reasons. First, the skin friction resistance that can be developed within the hard to very hard limestone and cemented clay underlying this site will be substantial, therefore fewer foundation elements will be required as compared to other options. Second, significant end bearing can develop provided good shaft construction and quality control measures are enforced. This will further increase the efficiency of this system. Finally, the lateral and uplift resistance of these elements will be substantially larger than any of the other foundation options. ***For these reasons, drilled shaft foundations are, in our opinion, the most technically correct and economically viable foundation alternative for this project.***

4.3.2 RECOMMENDED FOUNDATION ALTERNATIVE

Based on a review of all of our data collected to date, we recommend supporting the proposed structure using a drilled shaft foundation system. Shaft diameters of 36 inches to 48 inches should be adequate to cover the range of structural loads anticipated for this project, using a average embedment depth of 60 feet. Assuming a current ground surface elevation of +15 feet, the minimum tip elevation on this site is estimated to be -45 feet. This will need to be adjusted pending the results of load testing and pilot borings prior to construction of production shafts.

4.3.3 DRILLED SHAFT CAPACITY

Although we recommend the use of shaft diameters of 36 inches to 48 inches for this project, we have tabulated the estimated axial and lateral capacity of drilled shaft diameters of 24 inches through 60 inches, in 6 inch increments. For the axial analyses (compression and uplift), capacity curves were generated for each of these diameters at each of the eight borings, using various AASHTO and FDOT published methods. Ultimately, Borings B-01 and B-07 were selected as representative borings that bracketed the range of soil conditions encountered across the site. These borings were used for the final axial models. For the lateral analyses, Borings B-01 and B-07 were again used as representative borings (with and without a shallow rock lense, respectively), and each diameter was then modeled using L-PILE 5. P-delta effects were considered by applying the allowable axial load simultaneously with each lateral load modeled.

The average results of our analyses, shown below in Table 2, indicate that the allowable axial compressive capacity of drilled shafts embedded into the limestone on this site will be limited by the compressive strength of the concrete, not by the strength of the limestone. *Florida Building Code requires that the 28-day compressive strength of drilled shaft concrete is at least 4 times greater than the working stress that will be applied to the shaft.*

The capacity curves that were generated for each axial and lateral run are included in Appendix D for your reference. Although these curves show increasing capacity with increasing embedment depth, a code-mandated usable limit will be reached as previously mentioned. The allowable values listed in Table 2 are based on the assumption that 7000 psi concrete will be used to construct the shafts, and assume an average embedment depth of 60 feet below existing grade. These code-mandated limits are also shown on the figures in Appendix D. If a lesser mix strength will be used, these allowable values should be reduced accordingly.

TABLE 2
SUMMARY OF DRILLED SHAFT CAPACITIES - 60 FEET DEEP

Diameter (inches)	Minimum Toe Elevation (feet)	Allowable Compression (kips)	Allowable Tension (kips)	Allowable Lateral (kips)	Average Depth to Lateral Point of Fixity (feet)
24	-45	790	510	30	9
30	-45	1230	790	40	11
36	-45	1780	1150	70	12
42	-45	2420	1570	80	14
48	-45	3160	2050	90	16
54	-45	4000	2600	120	19
60	-45	4940	3210	140	22

The center to center spacing of adjacent shafts must not be less than three shaft diameters, or group capacity reduction may be necessary.

The allowable tensile (uplift) loads shown in Table 2 are based on the dead weight of each shaft and a portion of the fully mobilized skin friction. These capacities translate to approximately 65% of the allowable compressive capacities.

The allowable lateral loads shown in Table 2 are the loads for which a lateral shaft head displacement of approximately 0.5 inches is predicted, including bending caused by p-delta effects. These capacities translate to approximately 4% of the allowable compressive capacities. The predicted point of fixity below the existing ground surface (assuming no shaft stick-up) is also listed in Table 2 for each shaft size.

4.3.4 CAPS AND GRADE BEAMS

Drilled shafts may not require caps in all instances, but grade beams may still be needed to provide lateral bracing between pier locations. The project Structural Engineer should determine whether or not caps and grade beams are needed, and should provide the necessary design and construction guidelines.

4.3.5 PILOT BORINGS

A single pilot hole boring shall be drilled at the center of each proposed drilled shaft location to a depth of at least 3 times the shaft diameter below the planned tip elevation. Drilling should be completed using a standard drill-rig (i.e., CME-45/55 or equivalent) with an appropriate tri-cone bit, using the rotary wash method with limited SPT testing. The RPM, time required to advance the bit through the limestone/bearing stratum, in minutes per foot, and down pressure applied should all be monitored and recorded. Soil cuttings should also be monitored and logged, if possible. This will allow the tip elevation at each location to be adjusted by the UES Geotechnical Engineer based on changes in subsurface stratigraphy across the site.

UES shall be retained to provide the drilling and monitoring services during the drilled shaft construction phase of this project. This will allow the UES Geotechnical Engineer to provide input to field personnel in a timely manner during construction.

4.3.6 PILOT SHAFT LOAD TESTING

At least one strain-instrumented pilot shaft shall be constructed using the same materials and methodologies as production shafts. The pilot shaft shall be no smaller than 36 inches in diameter. This shaft may be reused as a production shaft, as testing will be non-destructive. A single pilot boring shall be advanced at the center of this shaft location prior to construction. The results of the pilot boring will be used to set the toe elevation and select instrumentation elevations.

This shaft shall be instrumented with either resistance or vibrating wire strain gages installed at elevations to be specified by the UES Geotechnical Engineer. A full scale static load test (ASTM D-1143) shall then be executed to at least 200% of the design allowable axial load in compression. Osterberg Cells (O-cells) shall be the method used to execute the load test, as reliable end bearing and side shear design values can be independently measured with this method. The UES Geotechnical Engineer shall be retained to direct the load test, review the collected data and results, and modify the drilled shaft design parameters as necessary. For additional information on O-cell testing, the reader is referred to http://www.loadtest.com.

Alternately, rapid load testing using Statnamic (http://www.testpile.com) is acceptable, provided at least two (2) separate shafts are tested on this site. This is due to the regression algorithm that is necessary to interpret Statnamic data, and also due to the possibility of not fully mobilizing skin friction in order to obtain reliable unit end bearing measurements, particularly in substantial rock sockets.

If rapid load testing is used, we recommend testing a typical highly loaded shaft (interior tower area) and a typical lighter loaded shaft (exterior tower or parking area). Again, the UES Geotechnical Engineer shall be retained to direct the load test, review the collected data and results, and modify the drilled shaft design parameters as necessary.

4.3.7 DRILLED SHAFT INSTALLATION

The axial and lateral capacity estimates presented herein assume no permanent casing will be used on the constructed shafts. Therefore, the installation of the shafts shall be accomplished using either the wet method (using water or drilling slurry) or the temporary casing method, as approved by the UES Geotechnical Engineer. When using wet method construction, a positive drilling fluid head shall be maintained above ground level throughout the excavation and concreting process.

Shaft concrete shall be tremie-placed from the bottom up, maintaining a positive concrete head above the bottom of the tremie throughout the pour at all times. Insertion of the tremie into the excavation below water/drilling slurry will require the installation of a tremie pig that is dislodged at the start of the pour. The use of the pig is not optional.

The Structural Engineer shall design the reinforcing cages and specify concrete mixes such that the CSD (the ratio of the smallest clear spacing in the cage, inclusive of spliced areas, instrumentation, CSL and grout tubes, etc., to the largest diameter coarse aggregate in the mix) is greater than 6. The use of a pea-gravel (FDOT #7 stone) mix is recommended, provided the required compressive strength (7000 psi for the Table 2 capacities) can be achieved.

Concrete slump from <u>each truck</u> shall be checked immediately before pouring, and slump shall also be measured immediately prior to temporary casing extraction. A slump of 8-inches is recommended during concrete placement, and an allowable range of 7-inches to 9-inches shall be used. Concrete must be placed within 90 minutes of being batched (unless the Structural Engineer approves the use of a retarder), and before the internal temperature of the mix reaches 100 °F. Concrete which does not meet all of these criteria shall be rejected.

Temporary casing, if used, shall be extracted immediately after concrete placement using a continuous, uniform rate. Casing extraction must be completed before the slump of the freshly poured concrete falls below 5-inches. Do not start and stop casing extraction. If concrete slump falls to 5 inches prior to casing extraction, leave the casing in place and contact the UES Geotechnical Engineer for design modifications.

Should the shafts be constructed using drill slurry, we recommend that the sand content of the slurry at the time of concrete placement not be allowed to exceed 1%. This will reduce the amount of sediment fallout during the pour, decreasing the likelihood for anomalies in the cross sections of the shafts due to agglomeration. This can be accomplished either by de-sanding or by maintenance of two separate slurry tanks - excavation slurry and concreting slurry.

<u>4.3.8 CSL TESTING</u>

We recommend that every drilled shaft contains one CSL access tube per foot of shaft diameter, but no fewer than 2 tubes per shaft, installed in the rebar cage along the entire axis of the shaft, or as directed by the UES Geotechnical Engineer. CSL tubes must be evenly spaced around the circumference of each shaft, and shall be attached with tie-wire to the inside of the rebar cage. Plastic CSL tubes shall not be used.

CSL testing of at least 10% of the production shafts, and the pilot shaft, should be completed to ensure proper cross section, embedment depth and concrete quality. Because of the extensive rock sockets that will be used, pile integrity testing (PIT) may not be effective, so CSL is the most feasible option to check shaft integrity and provide quality assurance.

The UES Geotechnical Engineer will specify which shafts are to be tested, and CSL shall not be completed on any shaft until at least 72 hours of curing has elapsed.

4.3.9 ESTIMATED STRUCTURAL SETTLEMENT

For foundations designed and installed as recommended we estimate total foundation settlement (vertical) of one inch or less, and differential settlement of less than one half inch. These estimates are highly dependant on the quality of the constructed shafts. If our recommendations concerning installation and quality assurance testing are not followed, these values may be exceeded during the service life of the structure.

4.4 PAVEMENT SECTIONS

4.4.1 RIGID PAVEMENT

We anticipate that a rigid concrete pavement system will be used on this project. Listed in Tables 3 and 4 below are general guidelines for concrete pavement given the following construction considerations:

1. Subgrade soils must be densified to at least 98% of the Modified Proctor Maximum Dry Density (MPMDD) (ASTM D-1557) to a depth of at least 1-foot directly below the bottom of concrete slab.

2. The surface of the subgrade soils must be smooth, and any disturbances or wheel rutting corrected prior to placement of concrete.

3. The subgrade soils must be moistened prior to placement of concrete.

4. Concrete pavement thickness should be uniform throughout, with exception to the thickened edges (curb or footing).

5. The bottom of the pavement should be separated from the estimated seasonal high groundwater level by at least 12 inches.

Our recommendations on slab thickness for standard duty concrete pavements are based on (1) the subgrade soils densified to at least 98% MPMDD, (2) modulus of subgrade reaction (k) equal to 100 psi/in, (3) a 20-year design life, and (4) total equivalent 18 kip single axle loads (ESAL) of 45,000. We recommend using the design shown in the following table for standard duty concrete pavements.

TABLE 3
RIGID PAVEMENT COMPONENT RECOMMENDATIONS - LIGHT DUTY

Minimum Pavement Thickness	Maximum Control Joint Spacing	Minimum Sawcut Depth
4 Inches	10 Feet x 10 Feet	1.0 Inches

Our recommendations on slab thickness for heavy duty concrete pavements are based on the same factors as above with the exception of the total ESAL increased to 300,000. Our recommended design for heavy duty concrete pavement is shown in Table 4 below.

TABLE 4 **RIGID PAVEMENT COMPONENT RECOMMENDATIONS - HEAVY DUTY**		
Minimum Pavement Thickness	Maximum Control Joint Spacing	Minimum Sawcut Depth
6 Inches	14 Feet x 14 Feet	1.5 Inches

For both standard duty and heavy duty rigid pavement sections, we recommend using normal weight concrete having a 28 day compressive strength (f'_c) of 4,000 psi, and a minimum 28-day flexural strength (modulus of rupture) of at least 600 psi (based on the 3 point flexural test of concrete beam samples). Layout of the sawcut control joints should form square panels, and the depth of sawcut joints should be at least ¼ of the concrete slab thickness.

We recommend allowing Universal Engineering Sciences to review and comment on the final concrete pavement design, including section and joint details (type of joints, joint spacing, etc.), prior to the start of construction.

For further details on concrete pavement construction, please reference the "Guide to Jointing of Non-Reinforced Concrete Pavements" published by the Florida Concrete and Products Association, Inc., and "Building Quality Concrete Parking Areas," published by the Portland Cement Association.

4.4.2 EFFECTS OF GROUNDWATER

One of the most critical influences on pavement performance in Florida is the relationship between the pavement subgrade and the seasonal high groundwater level.

It has been our experience that many roadways and parking areas have been damaged as a result of deterioration of the base and the base/surface course bond due to moisture intrusion. Regardless of the type of base selected, we recommend that the seasonal high groundwater and the bottom of the base course be separated by at least 18-inches.

At this site, pavement constructed on or above existing grade should meet the minimum required separation.

4.4.3 CURBING

We recommend that curbing around any landscaped sections adjacent to the parking lots and driveways be constructed with full-depth curb sections. Using extruded curb sections which lie directly on top of the final asphalt level, or eliminating the curbing entirely, can allow migration of irrigation water from the landscape areas to the interface between the asphalt and the base. This migration often causes separation of the wearing surface from the base and subsequent rippling and pavement deterioration.

4.4.4 CONSTRUCTION TRAFFIC

Light duty roadways and incomplete pavement sections will not perform satisfactorily under construction traffic loadings. We recommend that construction traffic (construction equipment, concrete trucks, sod trucks, garbage trucks, dump trucks, etc.) be re-routed away from these roadways or that the pavement section be designed for these loadings.

4.5 SITE PREPARATION

We recommend normal, good-practice site preparation procedures. As the site has already been cleared and rough graded, additional preparation prior to mobilization of foundation installation equipment should be minimal. General guidelines for clearing, filling and grading are provided below.

1. Strip the proposed construction limits of any remaining deleterious materials within and 5 feet beyond the perimeter of the proposed building and in all paved areas. Some small buried debris may be encountered within the first several feet of excavation on this site, but we do not anticipate substantial occurrences of such debris.

2. Proof-roll the subgrade with a heavily loaded, rubber-tired vehicle under the observation of a Universal Engineering Sciences geotechnical engineer or his representative. Proof-rolling will help locate any zones of especially loose or soft soils not encountered in the soil test borings. Then undercut, or otherwise treat these zones as recommended by the engineer.

3. Prior to any filling of the site, compact the subgrade from the surface using a heavy vibratory drum roller, until you obtain a minimum density of 95% MPMDD to a depth of 2 feet below stripped grade. In order to achieve the required degree of compaction, the soils may need to be moisture conditioned until the in-situ water content is within +/- 2% of the optimum moisture content (OMC).

4. Place fill material as required. The fill should consist of fine to medium sand with less than 5 percent soil fines. You may use fill materials with soil fines between 5 and 12 percent, but strict moisture control may be required. Place fill in uniform 10 to 12 inch loose lifts and compact each lift to a minimum density of 95% MPMDD at a moisture content of +/- 2% of optimum (OMC).

5. Perform compliance tests within the fill at a frequency of not less than one test per 2,500 square feet per lift in the building areas, or at a minimum of two test locations, whichever is greater. In paved areas, perform compliance tests at a frequency of not less than one test per 10,000 square feet per lift, or at a minimum of two test locations, whichever is greater.

Using vibratory compaction equipment or vibratory casing installation hammers on this site may disturb adjacent structures. We recommend you monitor nearby structures before and during casing installation/extraction and all compaction operations.

4.6 CONSTRUCTION RELATED SERVICES

Universal Engineering Sciences (UES) operates and maintains an in-house, Florida Department of Transportation certified Construction Materials Testing laboratory. Our technicians are highly trained and experienced, and our engineering staff is already familiar with the details of your project. Therefore, we recommend the owner retain UES to perform construction materials testing and field observations on this project. This includes drilling all pilot borings, directing instrumentation and load testing of the pilot shaft(s), monitoring installation of all drilled shafts, materials testing and post-tension inspections during vertical construction, and threshold inspection services.

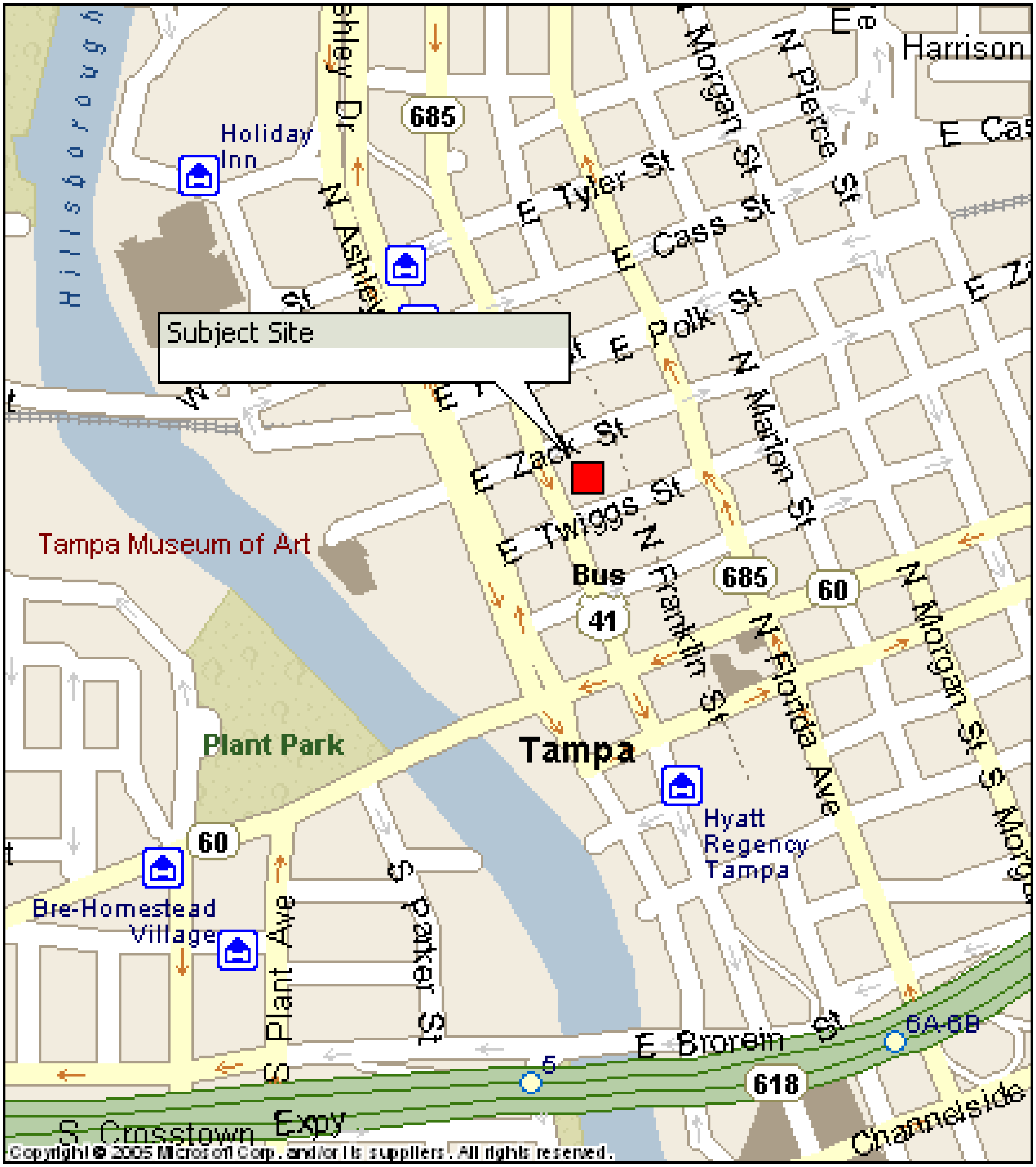
The geotechnical engineering design does not end with the advertisement of the construction documents. It is an on-going process throughout construction. Because of our familiarity with the site conditions and the intent of the engineering design, our engineers are the most qualified to address problems that might arise during construction in a timely and cost-effective manner.

5.0 LIMITATIONS

During the early stages of most construction projects, geotechnical issues not addressed in this report may arise. Because of the natural limitations inherent in working with the subsurface, it is not possible for a geotechnical engineer to predict and address all possible subsurface variations. An Association of Engineering Firms Practicing in the Geosciences (ASFE) publication, "Important Information About Your Geotechnical Engineering Report" appears in Appendix E, and will help explain the nature of geotechnical issues. Further, we present documents in Appendix E: Constraints and Restrictions, to bring to your attention the potential concerns and the basic limitations of a typical geotechnical report.

Do not apply any of this report's conclusions or recommendations if the nature, design, or location of the facilities is changed. If changes are contemplated, UES must review them to assess their impact on this report's applicability. Also, note that UES is not responsible for any claims, damages, or liability associated with any other party's interpretation of this report's subsurface data or reuse of this report's subsurface data or engineering analyses without the express written authorization of UES.

APPENDIX A



UNIVERSAL
ENGINEERING SCIENCES

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, HILLSBOROUGH COUNTY, FLORIDA

SITE LOCATION MAP

DRAWN BY: J.C.	DATE: MAY 31, 2006	REVIEWED BY: E.J.G.	DATE: MAY 31, 2006
SCALE: NOT TO SCALE	PROJECT NO: 80732-003-01	REPORT NO:	APPENDIX: A

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PROPOSED SIXTEN FRANKLIN BUILDING
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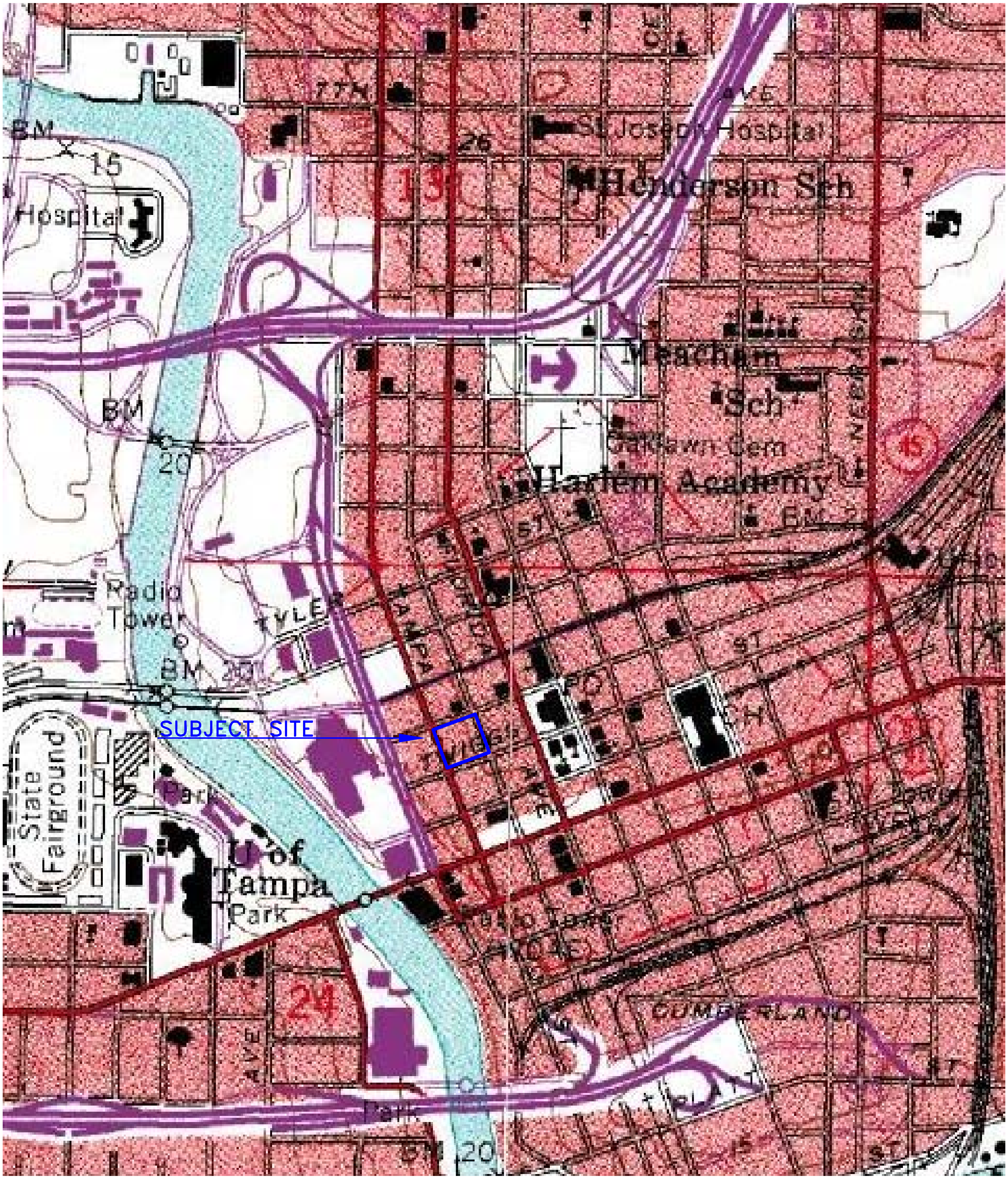
SITE AERIAL PHOTOGRAPH

DRAWN BY: J.C.	DATE: MAY 31, 2006	REVIEWED BY: E.J.G.	DATE: MAY 31, 2006
SCALE: NOT TO SCALE	PROJECT NO: 80732-003-01	REPORT NO:	APPENDIX: A



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PROPOSED SIXTEN FRANKLIN BUILDING
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TAMPA, HILLSBOROUGH COUNTY, FLORIDA

SITE TOPOGRAPHIC MAP

DRAWN BY: J.C.	DATE: MAY 31, 2006	REVIEWED BY: E.J.G.	DATE: MAY 31, 2006
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PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
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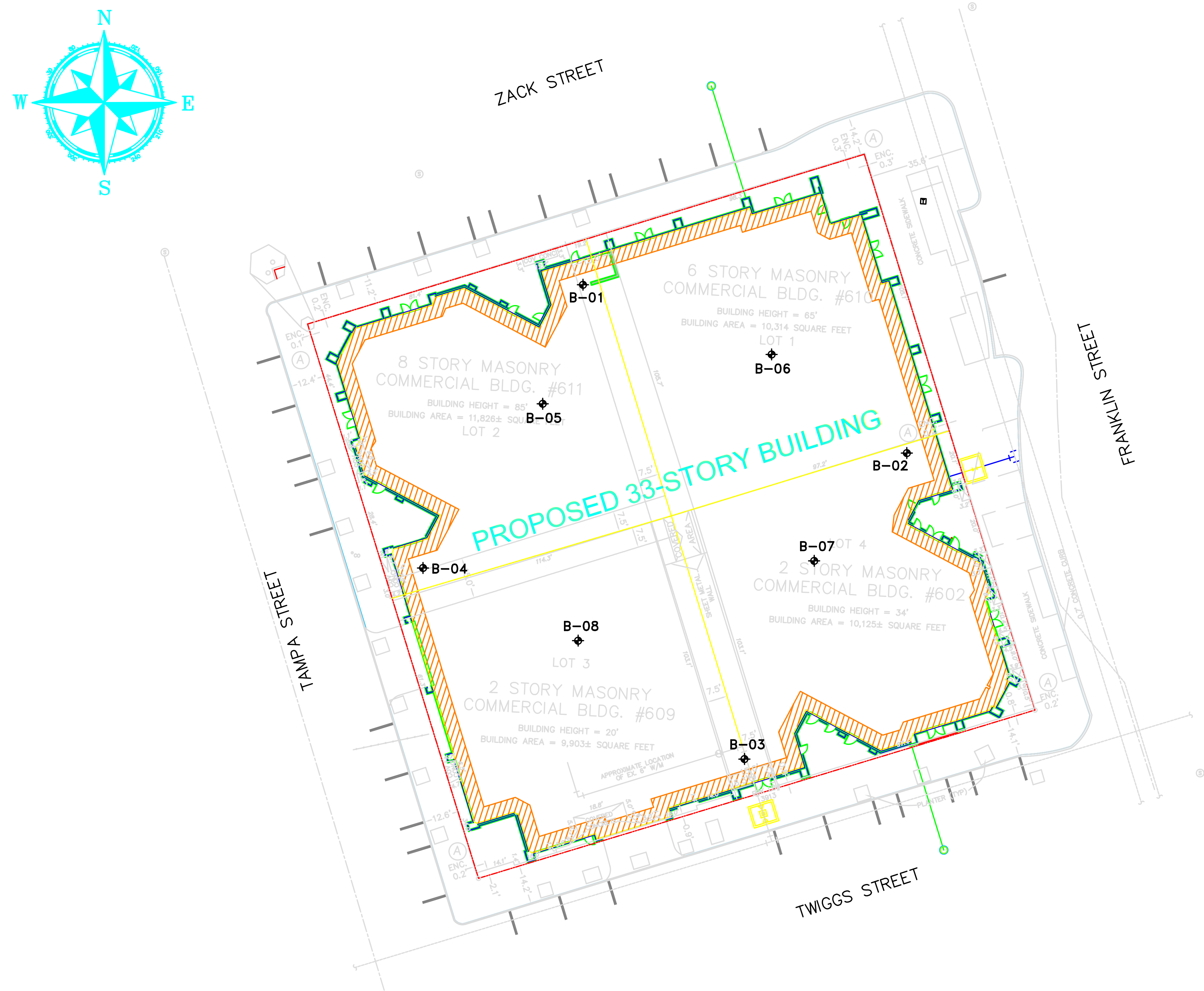
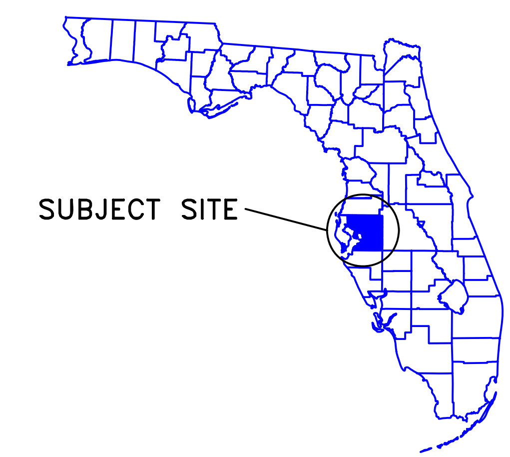
SCS SOIL SURVEY MAP

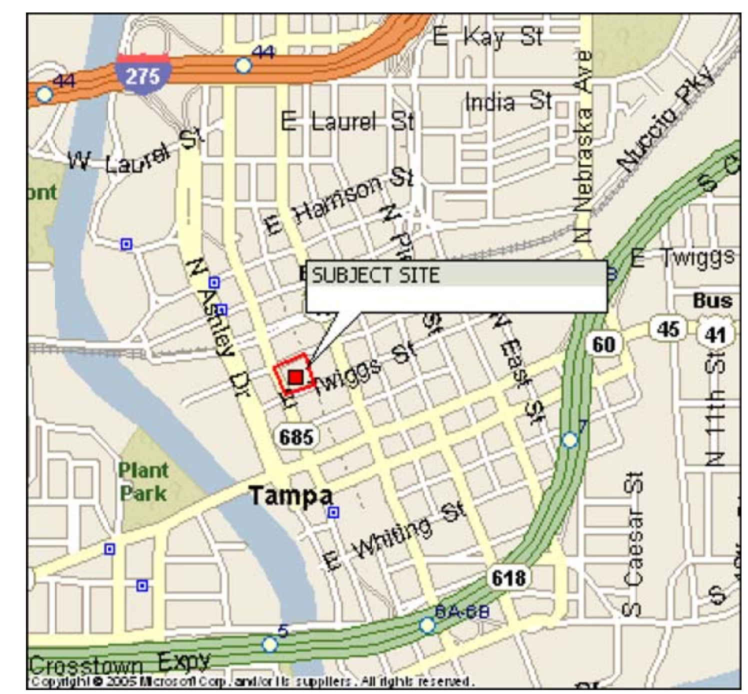
DRAWN BY: J.C.	DATE: MAY 31, 2006	REVIEWED BY: E.J.G.	DATE: MAY 31, 2006
SCALE: NOT TO SCALE	PROJECT NO: 80732-003-01	REPORT NO:	APPENDIX: A

APPENDIX B





SECTION: 24 TOWNSHIP: 29 S RANGE: 18 E



LEGEND:
⌖B-01 Approximate SPT boring location

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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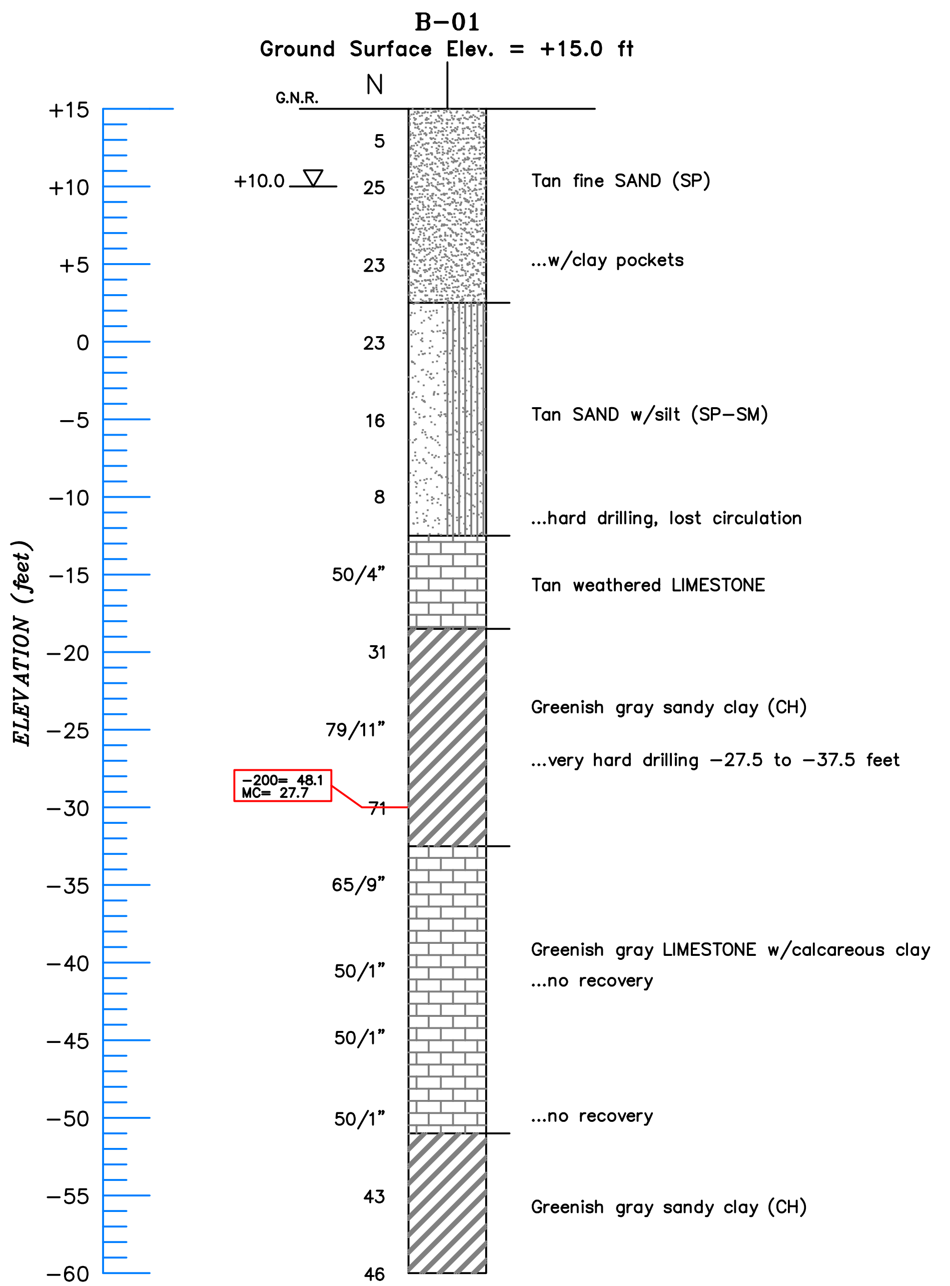
WOOD PARTNERS

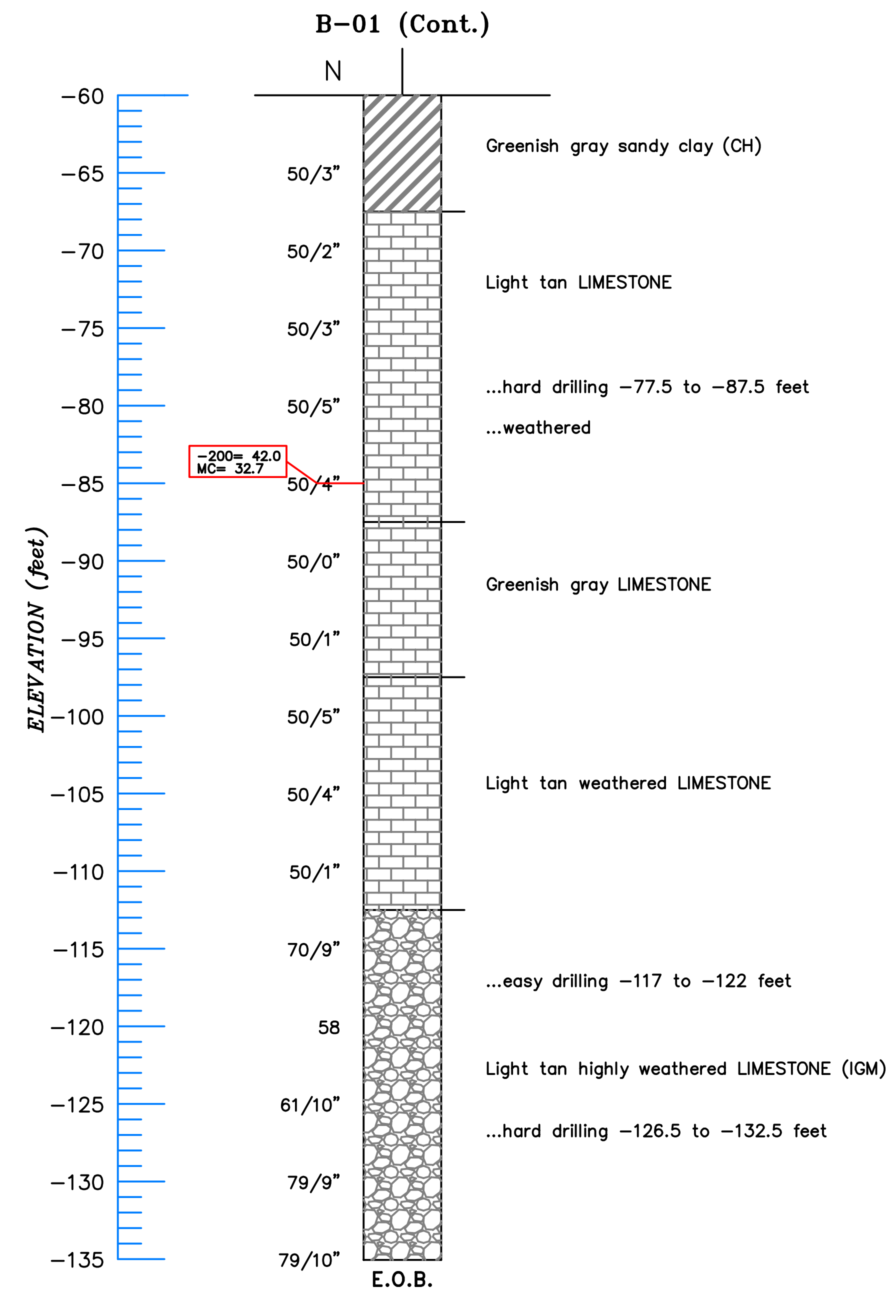
COUNTY	PROJECT NO.
HILLSBOROUGH	80732-003-01

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

BORING LOCATION PLAN

APPENDIX
B
SHEET NO.
1 OF 9





KEY TO BORING LOGS

FILL — Highly Weathered LIMESTONE — LIMESTONE
SAND — Clayey SAND — CLAY (CH)
SAND w/ Silt or Clay — Silty SAND — CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
RELATIVE DENSITY	SPT N (BLOWS/FT.)	CONSISTENCY	SPT N (BLOWS/FT.)
Very Loose	0–4	Very Soft	0–2
Loose	5–10	Soft	3–4
Medium Dense	11–30	Firm	5–8
Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS	LL	LIQUID LIMIT
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)	PI	PLASTICITY INDEX
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING	+10.0 ▽	EST. SEASONAL HIGH GROUNDWATER TABLE
-200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE	+8.2 ▼	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
WC	NATURAL MOISTURE CONTENT (%)	G.N.R.	GROUNDWATER TABLE NOT REPORTED
UC	UNCONFINED COMPRESSIVE STRENGTH	ST	SPLITTING TENSILE STRENGTH
		REC	PERCENT RECOVERY
		RQD	ROCK QUALITY DESIGNATION

BORING LOCATION PLAN

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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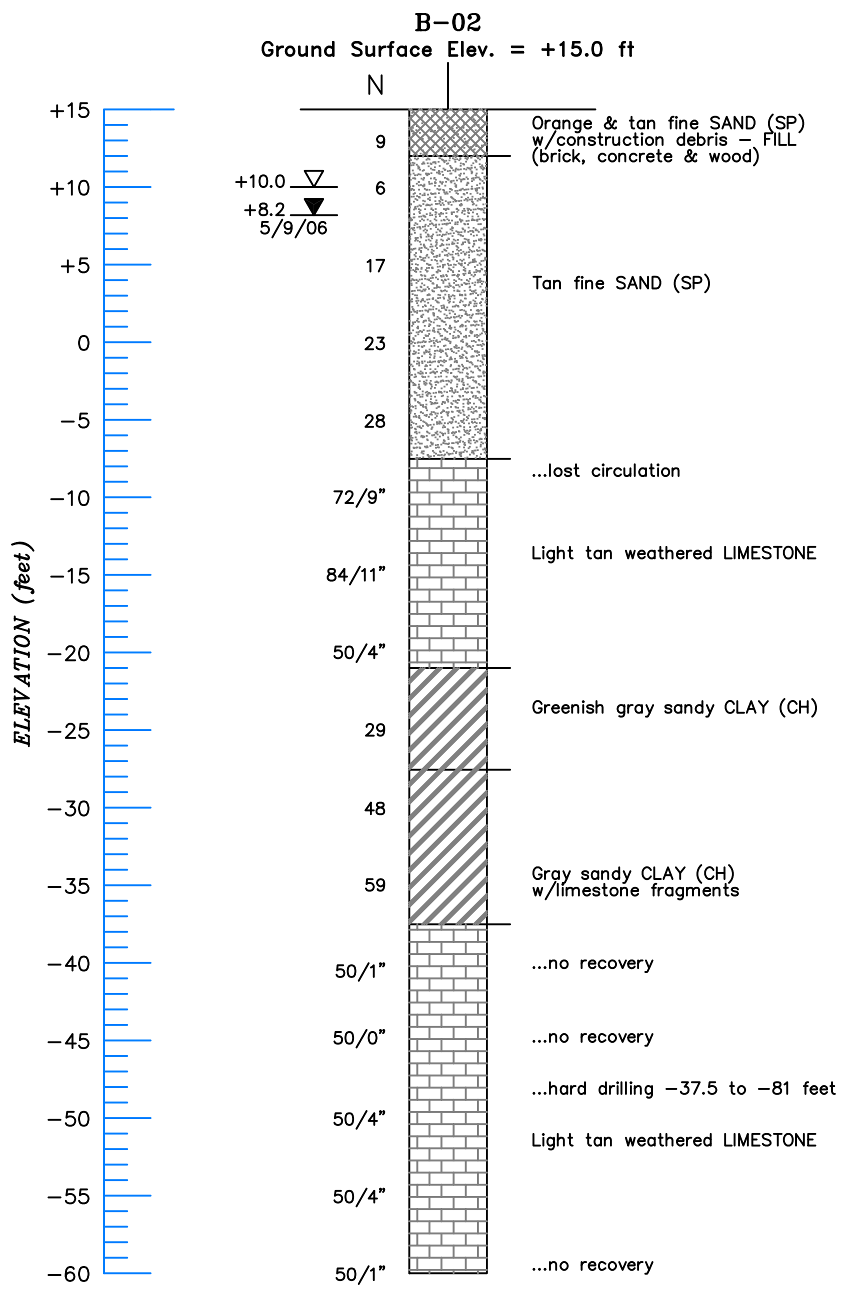
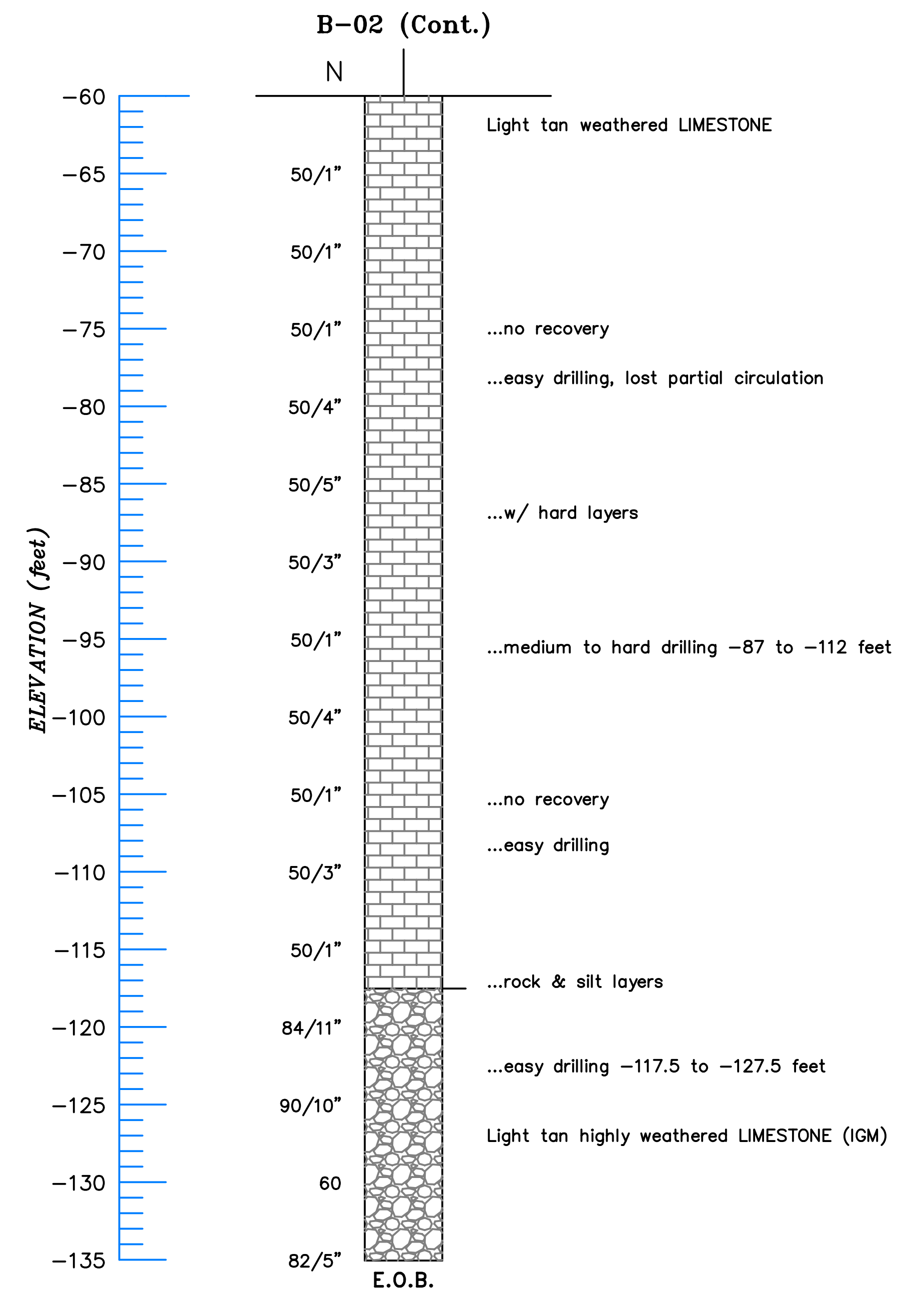
COUNTY: HILLSBOROUGH
PROJECT NO.: 80732-003-01

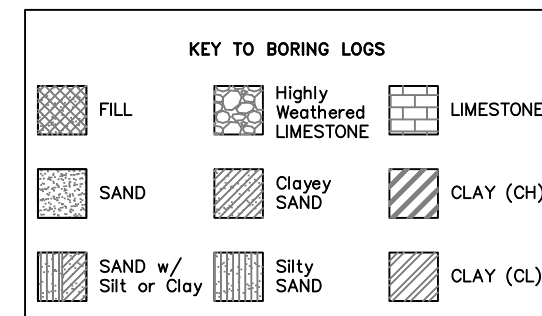
PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX B
SHEET NO. 2 OF 9







CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
RELATIVE DENSITY	SPT N (BLOWS/FT.)	CONSISTENCY	SPT N (BLOWS/FT.)
Very Loose	0–4	Very Soft	0–2
Loose	5–10	Soft	3–4
Medium Dense	11–30	Firm	5–8
Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

BORING LOCATION PLAN

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS	LL	LIQUID LIMIT
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)	PI	PLASTICITY INDEX
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING	+8.2 ▼	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
–200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE	G.N.R.	GROUNDWATER TABLE NOT REPORTED
WC	NATURAL MOISTURE CONTENT (%)	ST	SPLITTING TENSILE STRENGTH
UC	UNCONFINED COMPRESSIVE STRENGTH	REC	PERCENT RECOVERY
		RQD	ROCK QUALITY DESIGNATION

+10.0 ▽ EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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COUNTY: HILLSBOROUGH
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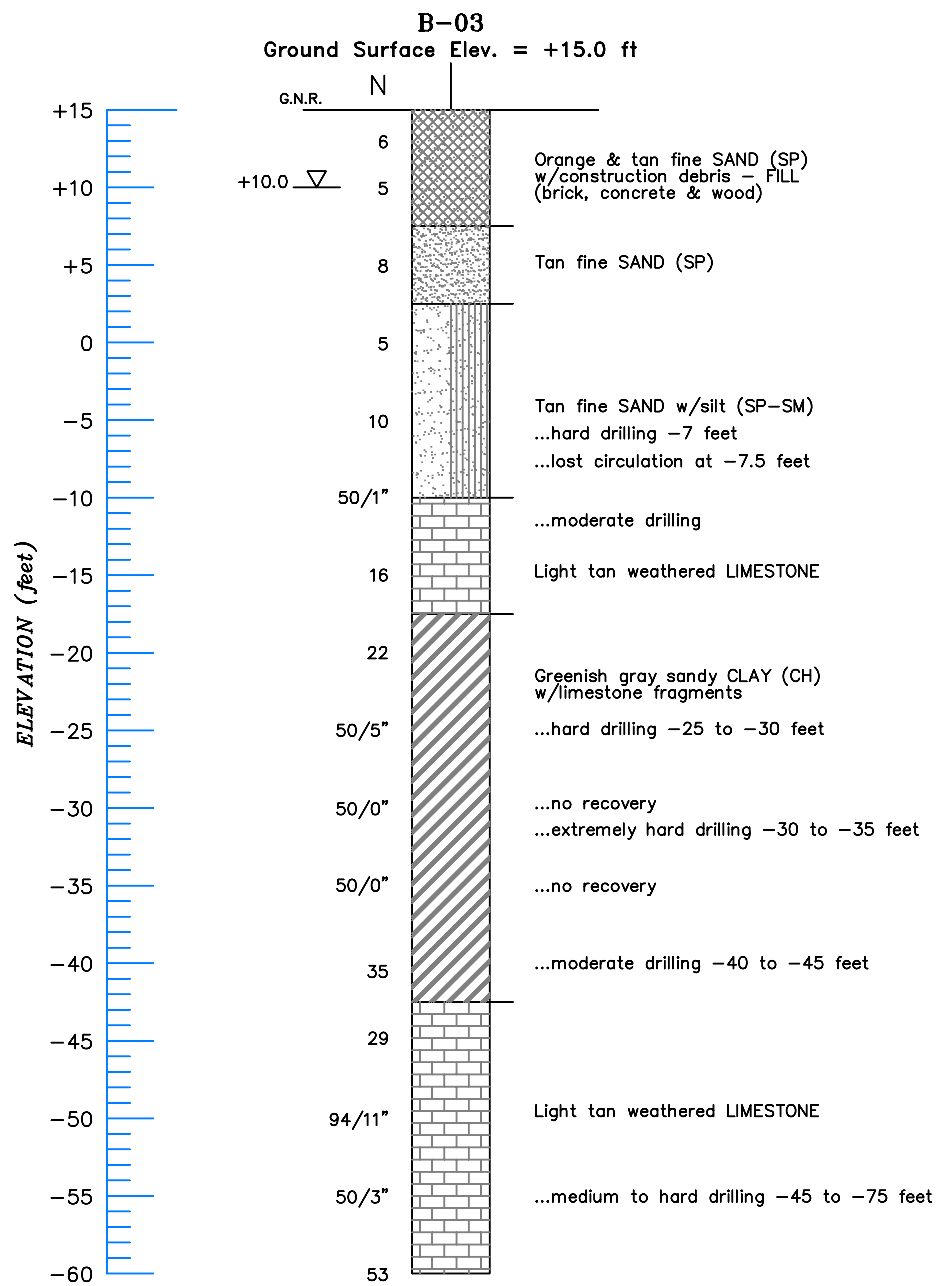
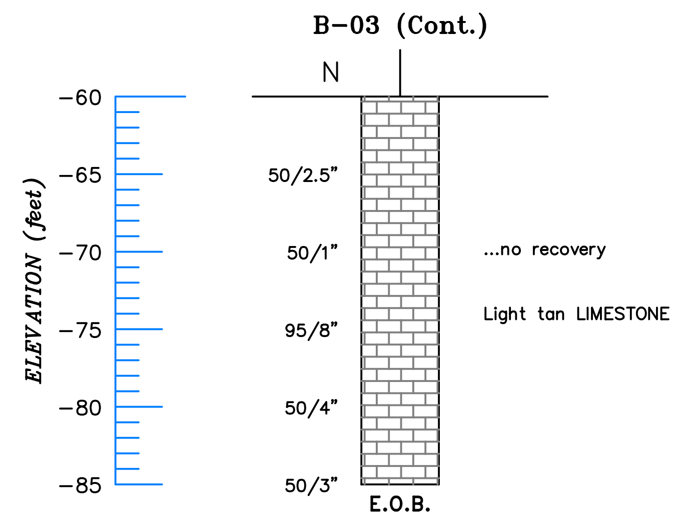
PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

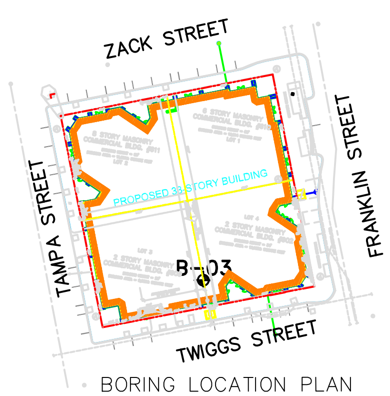
REPORT OF SOIL BORINGS

APPENDIX B

SHEET NO. 3 OF 9







BORING LOCATION PLAN

KEY TO BORING LOGS

FILL; Highly Weathered LIMESTONE; LIMESTONE; SAND; Clayey SAND; CLAY (CH); SAND w/ Silt or Clay; Silty SAND; CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
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Dense	31-50	Stiff	9-16
Very Dense	50+	Very Stiff	17-30
		Hard	30+

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING
-200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
WC	NATURAL MOISTURE CONTENT (%)
UC	UNCONFINED COMPRESSIVE STRENGTH
LL	LIQUID LIMIT
PI	PLASTICITY INDEX
+8.2 ▼	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
G.N.R.	GROUNDWATER TABLE NOT REPORTED
ST	SPLITTING TENSILE STRENGTH
REC	PERCENT RECOVERY
RQD	ROCK QUALITY DESIGNATION
+10.0 ▽	EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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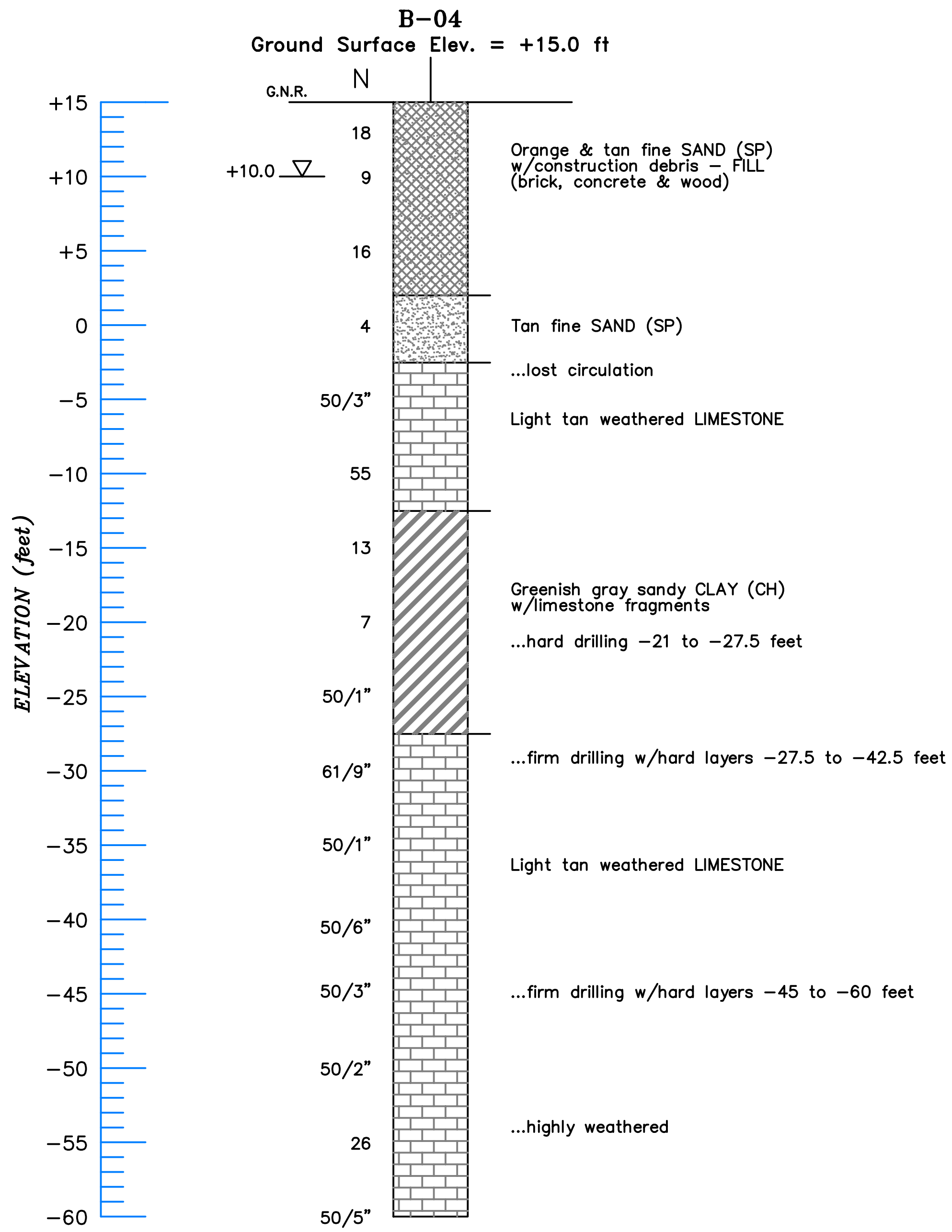
WOOD PARTNERS

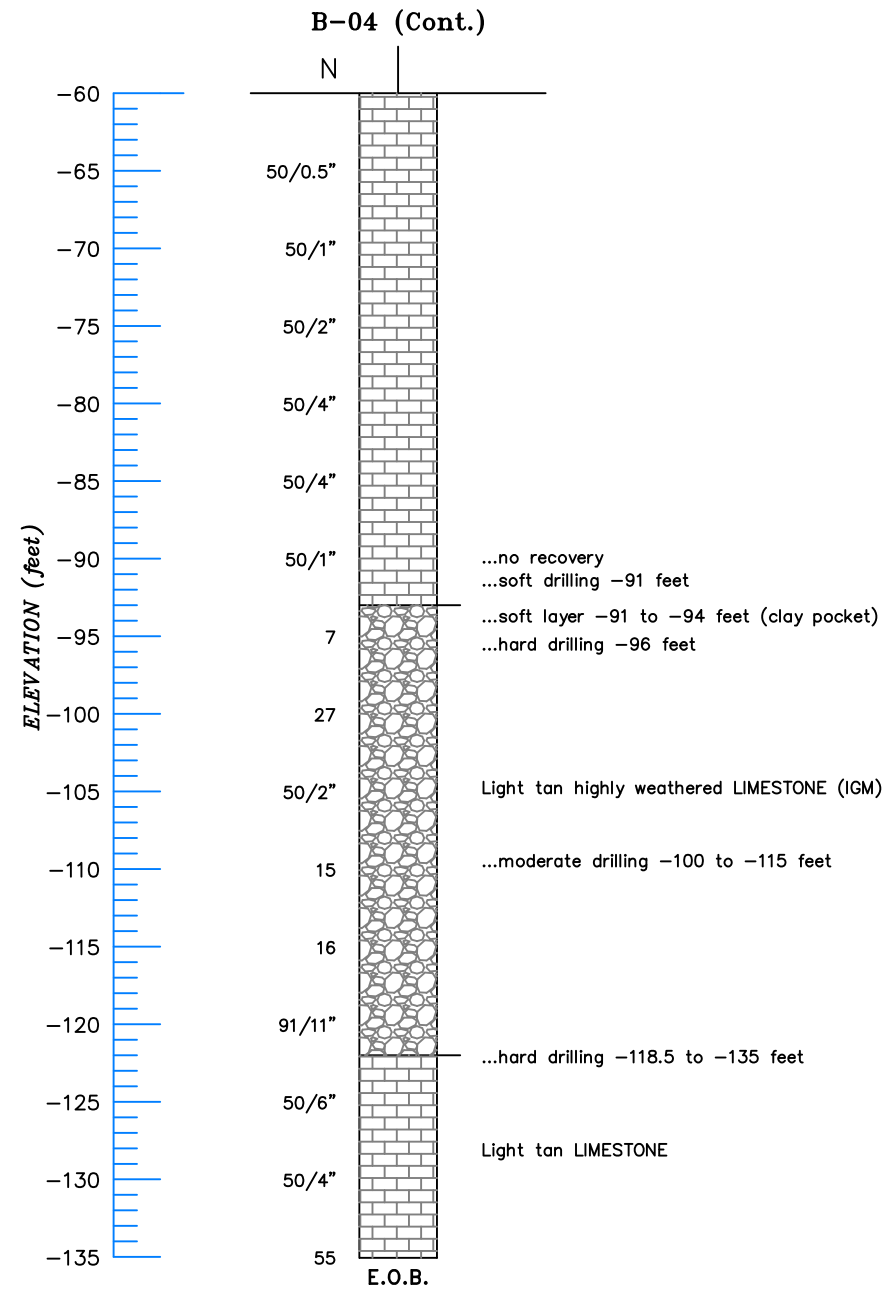
COUNTY	PROJECT NO.
HILLSBOROUGH	80732-003-01

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX
B
SHEET NO.
4 OF 9





KEY TO BORING LOGS

FILL — Highly Weathered LIMESTONE — LIMESTONE
SAND — Clayey SAND — CLAY (CH)
SAND w/ Silt or Clay — Silty SAND — CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
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Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

BORING LOCATION PLAN

NOTES TO BORING LOGS

- G.S.E. GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS
- N STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)
- (SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING
- –200 PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
- WC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH
- LL LIQUID LIMIT
- PI PLASTICITY INDEX
- +10.0 EST. SEASONAL HIGH GROUNDWATER TABLE
- +8.2 GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
- G.N.R. GROUNDWATER TABLE NOT REPORTED
- ST SPLITTING TENSILE STRENGTH
- REC PERCENT RECOVERY
- RQD ROCK QUALITY DESIGNATION

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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WOOD PARTNERS

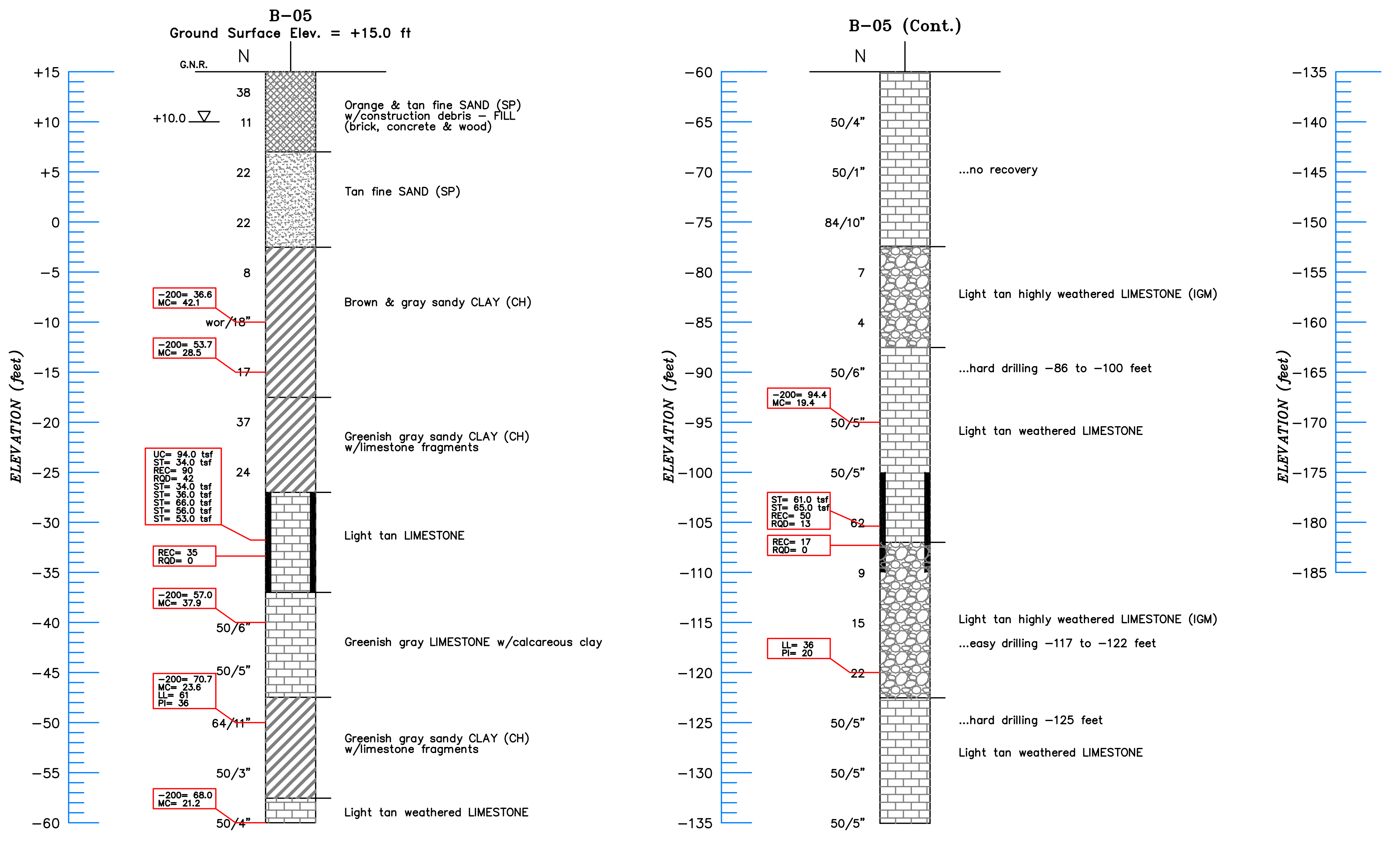
COUNTY: HILLSBOROUGH
PROJECT NO.: 80732-003-01

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX B

SHEET NO. 5 OF 9



KEY TO BORING LOGS

FILL	Highly Weathered LIMESTONE	LIMESTONE
SAND	Clayey SAND	CLAY (CH)
SAND w/ Silt or Clay	Silty SAND	CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

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BORING LOCATION PLAN

NOTES TO BORING LOGS

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G.N.R.	GROUNDWATER TABLE NOT REPORTED
ST	SPLITTING TENSILE STRENGTH
REC	PERCENT RECOVERY
RQD	ROCK QUALITY DESIGNATION
+10.0 ▽	EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
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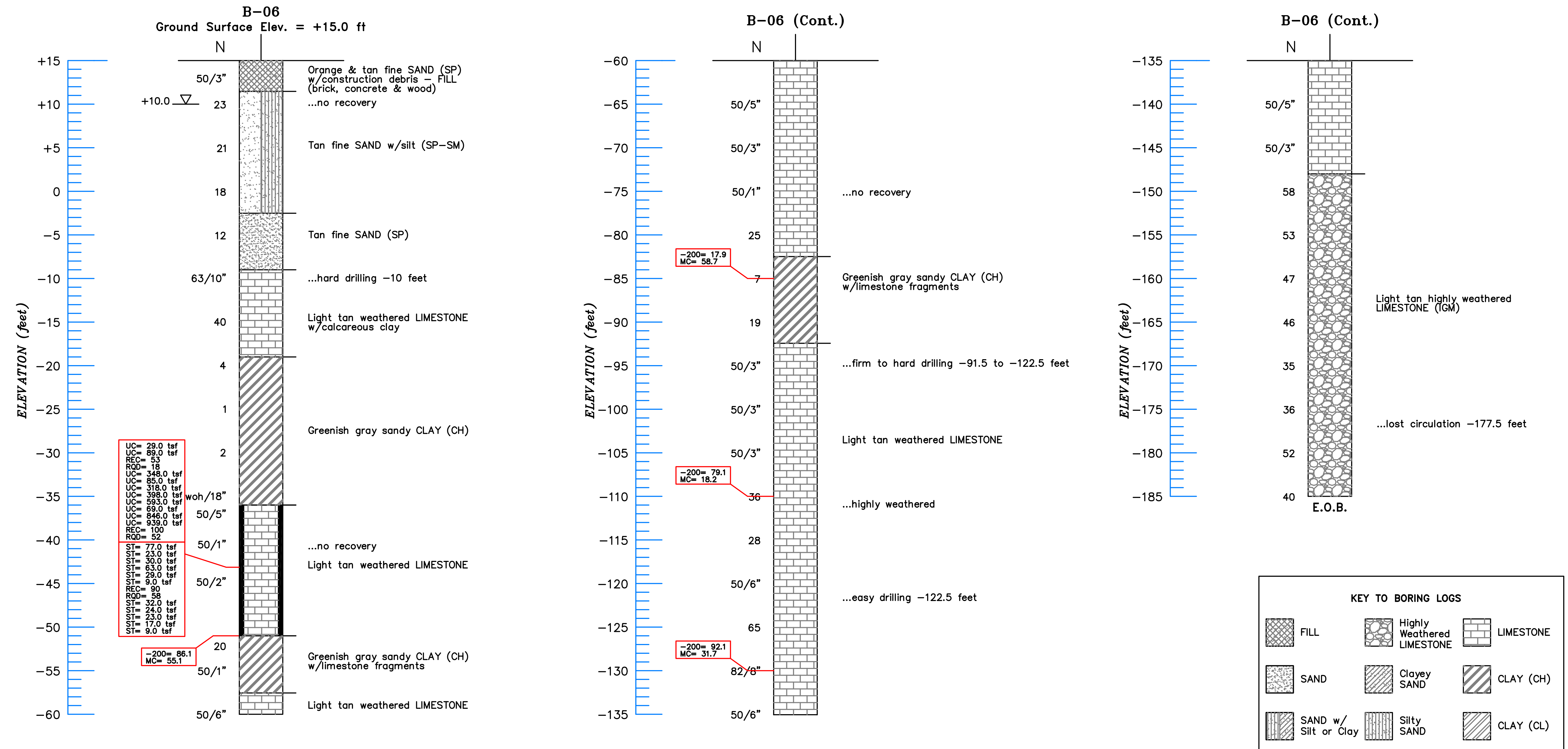
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX B

SHEET NO. 6 OF 9



CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
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Medium Dense	11–30	Firm	5–8
Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

BORING LOCATION PLAN

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING
-200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
WC	NATURAL MOISTURE CONTENT (%)
UC	UNCONFINED COMPRESSIVE STRENGTH
LL	LIQUID LIMIT
PI	PLASTICITY INDEX
+8.2 ▼	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
G.N.R.	GROUNDWATER TABLE NOT REPORTED
ST	SPLITTING TENSILE STRENGTH
REC	PERCENT RECOVERY
RQD	ROCK QUALITY DESIGNATION
+10.0 ▽	EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

Dr. Edward J. Garbin, Jr., P.E.
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ENGINEER OF RECORD:
Universal Engineering Sciences
9802 Palm River Road
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Phone: (813) 740-8506
Cert. of Authorization #549

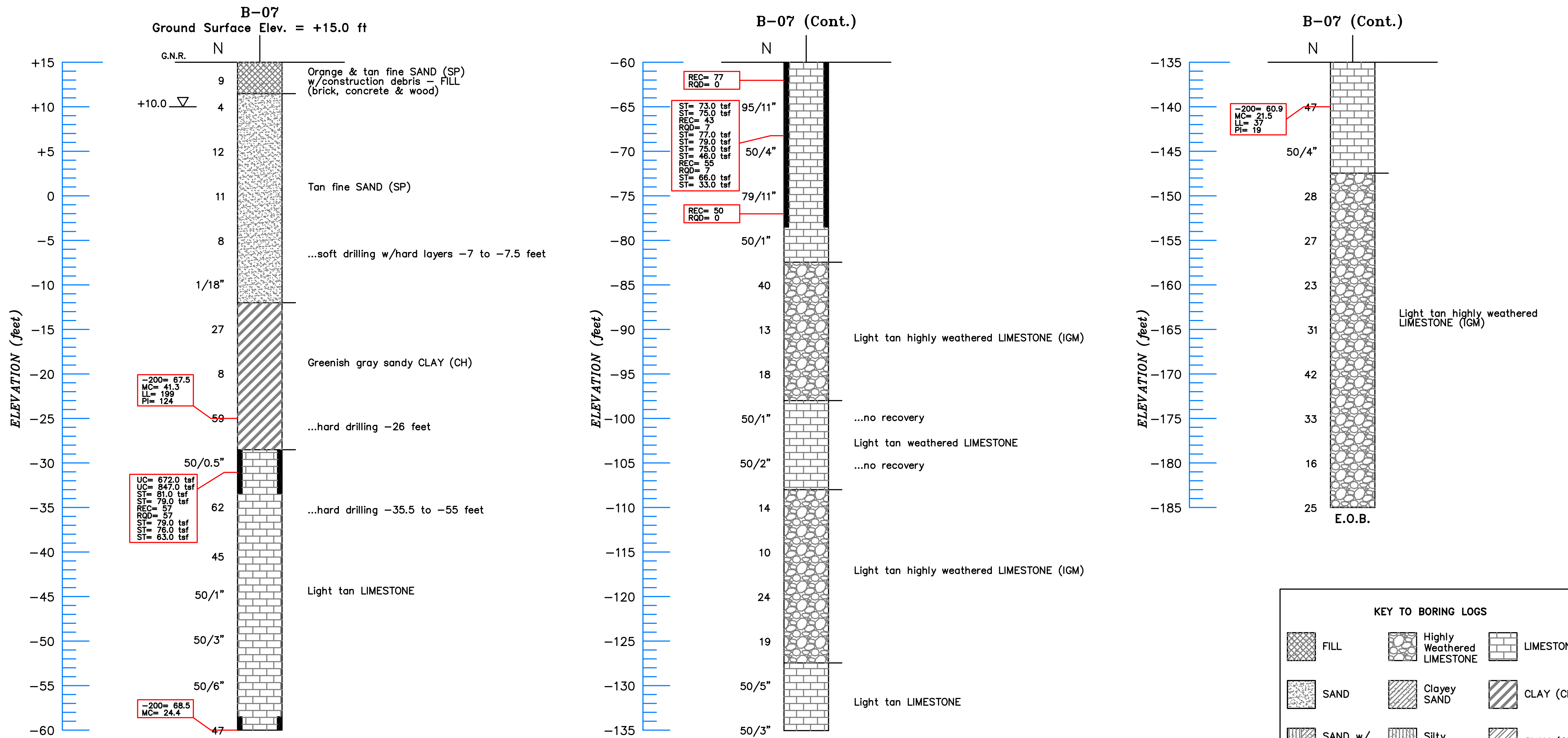
WOOD PARTNERS

COUNTY	PROJECT NO.
HILLSBOROUGH	80732-003-01

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX
B
SHEET NO.
7 OF 9



KEY TO BORING LOGS

FILL	Highly Weathered LIMESTONE	LIMESTONE
SAND	Clayey SAND	CLAY (CH)
SAND w/ Silt or Clay	Silty SAND	CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
RELATIVE DENSITY	SPT N (BLOWS/FT.)	CONSISTENCY	SPT N (BLOWS/FT.)
Very Loose	0–4	Very Soft	0–2
Loose	5–10	Soft	3–4
Medium Dense	11–30	Firm	5–8
Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

BORING LOCATION PLAN

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS	LL	LIQUID LIMIT
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)	PI	PLASTICITY INDEX
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING	+8.2	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
–200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE	G.N.R.	GROUNDWATER TABLE NOT REPORTED
WC	NATURAL MOISTURE CONTENT (%)	ST	SPLITTING TENSILE STRENGTH
UC	UNCONFINED COMPRESSIVE STRENGTH	REC	PERCENT RECOVERY
		RQD	ROCK QUALITY DESIGNATION
		+10.0	EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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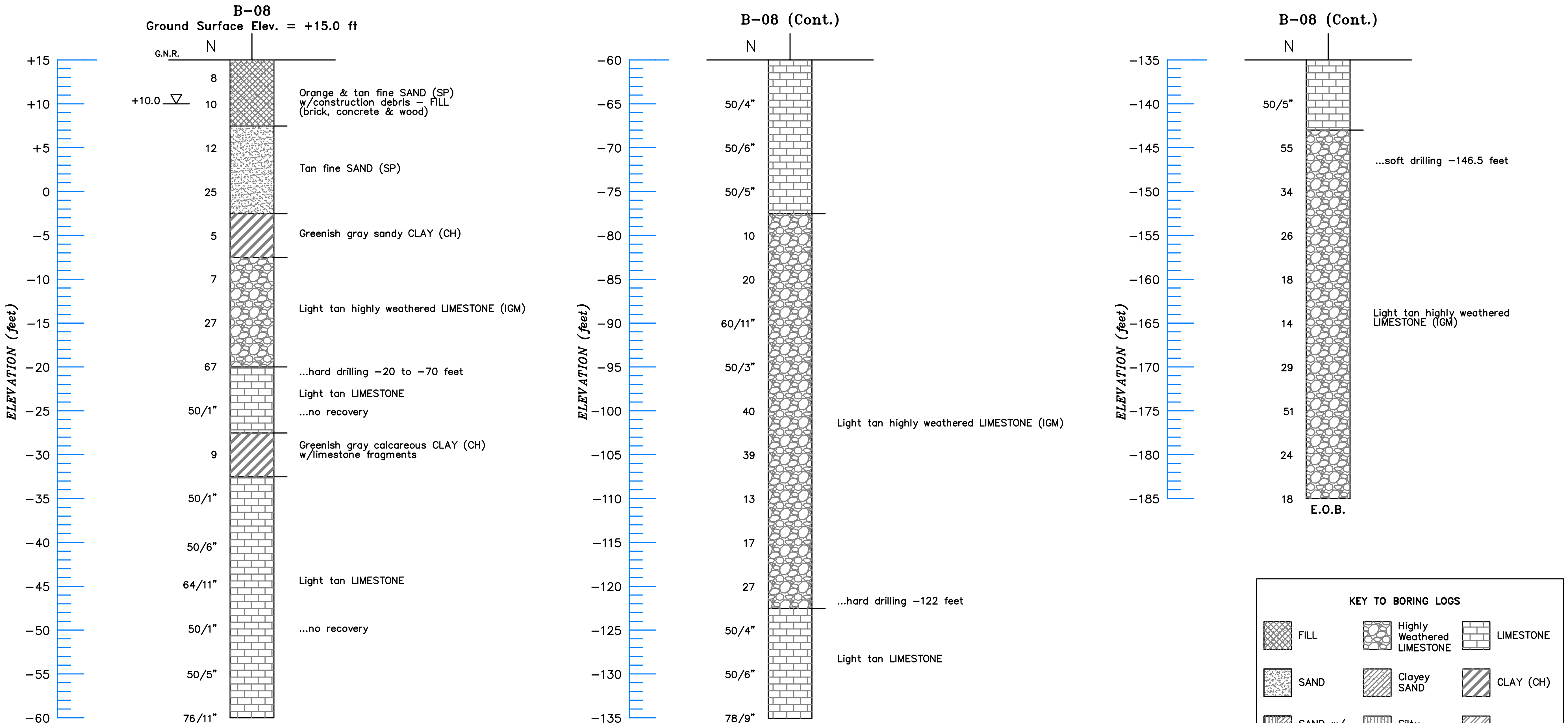
COUNTY	PROJECT NO.
HILLSBOROUGH	80732-003-01

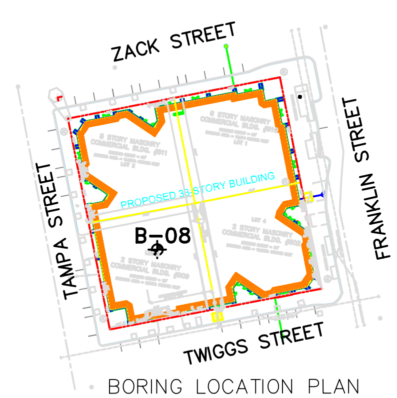
PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX B

SHEET NO. 8 OF 9





KEY TO BORING LOGS

FILL	Highly Weathered LIMESTONE	LIMESTONE
SAND	Clayey SAND	CLAY (CH)
SAND w/ Silt or Clay	Silty SAND	CLAY (CL)

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR MATERIALS (SANDS & GRAVELS)		COHESIVE MATERIALS (SILTS & CLAYS)	
RELATIVE DENSITY	SPT N (BLOWS/FT.)	CONSISTENCY	SPT N (BLOWS/FT.)
Very Loose	0–4	Very Soft	0–2
Loose	5–10	Soft	3–4
Medium Dense	11–30	Firm	5–8
Dense	31–50	Stiff	9–16
Very Dense	50+	Very Stiff	17–30
		Hard	30+

NOTES TO BORING LOGS

G.S.E.	GROUND SURFACE ELEVATION ESTIMATED FROM USGS TOPOGRAPHIC MAPS	LL	LIQUID LIMIT
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (24" SPOON – ASTM D 1586)	PI	PLASTICITY INDEX
(SP)	UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL BASED ON VISUAL OBSERVATION/LAB TESTING	+8.2	GROUNDWATER TABLE ELEVATION ENCOUNTERED DURING DRILLING
-200	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE	G.N.R.	GROUNDWATER TABLE NOT REPORTED
WC	NATURAL MOISTURE CONTENT (%)	ST	SPLITTING TENSILE STRENGTH
UC	UNCONFINED COMPRESSIVE STRENGTH	REC	PERCENT RECOVERY
		RQD	ROCK QUALITY DESIGNATION

+10.0 EST. SEASONAL HIGH GROUNDWATER TABLE

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/31/06	J.C.	FINAL REVISIONS			

	Names	Dates
Drawn by	J.C.	5/30/06
Reviewed by	E.J.G.	5/31/06

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COUNTY	PROJECT NO.
HILLSBOROUGH	80732-003-01

PROPOSED SIXTEN FRANKLIN BUILDING
610 FRANKLIN ST.
TAMPA, FLORIDA

REPORT OF SOIL BORINGS

APPENDIX B

SHEET NO. 9 OF 9

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% 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 OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				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 GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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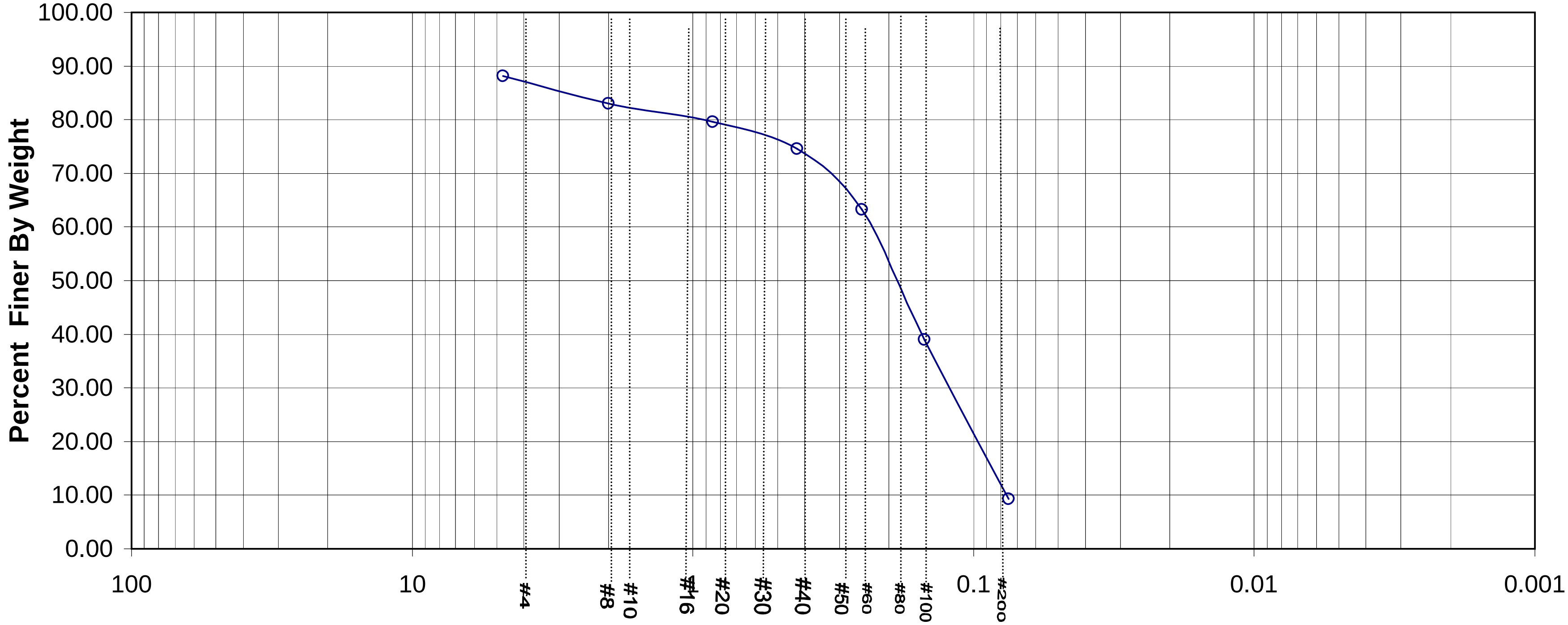
GRAIN SIZE DISTRIBUTION



GRAVEL		SAND			SILT OR CLAY
COURSE	FINE	COURSE	MEDIUM	FINE	

UNIVERSAL
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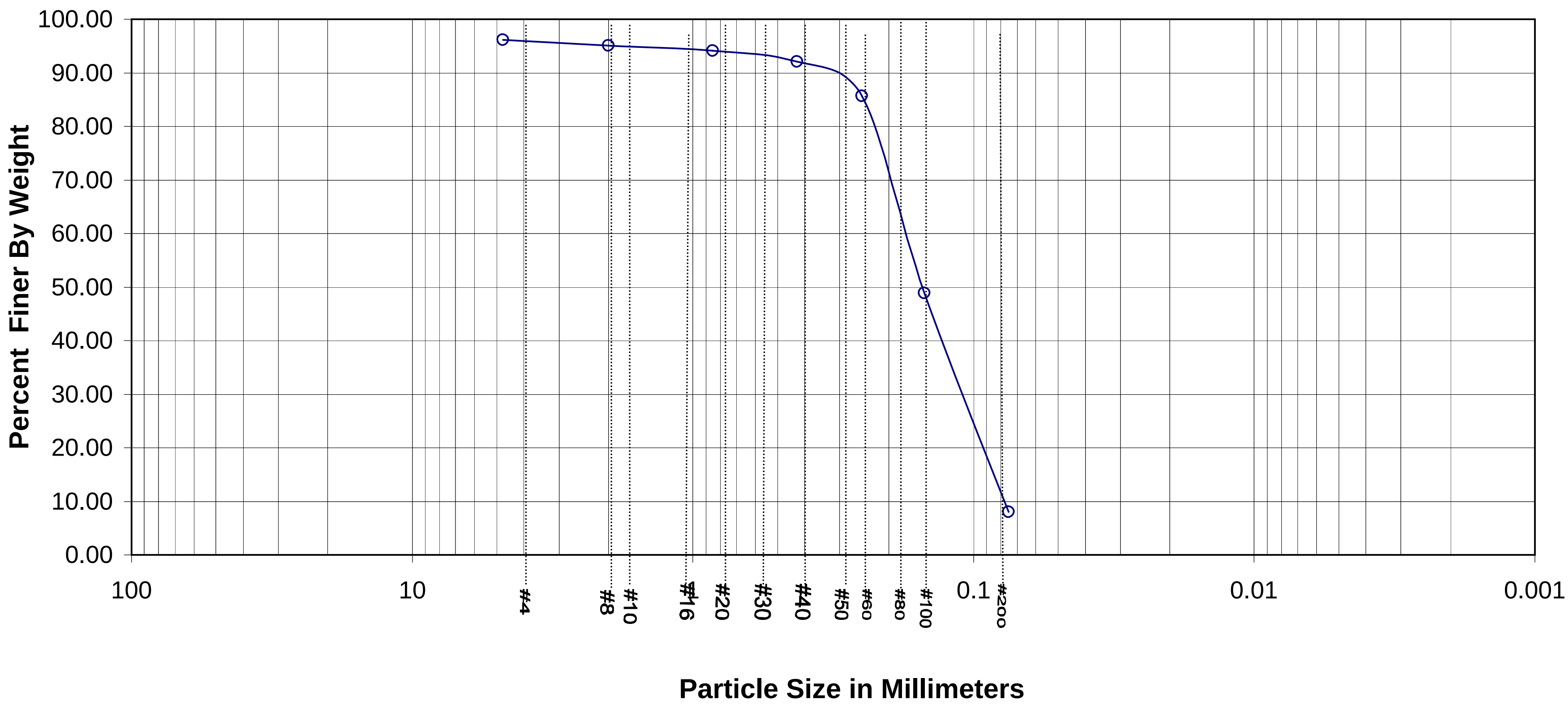
SixTen Franklin Building

SAMPLE: B-5 / 1-4

GRAIN SIZE DISTRIBUTION ANALYSIS

Project #:	80732-003-01	**Tested By:**	KS	**Date:**	5/20/2006

GRAIN SIZE DISTRIBUTION



GRAVEL		SAND			SILT OR CLAY
COURSE	FINE	COURSE	MEDIUM	FINE	

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ENGINEERING SCIENCES

SixTen Franklin Building

SAMPLE: B-7 / 1-6

GRAIN SIZE DISTRIBUTION ANALYSIS

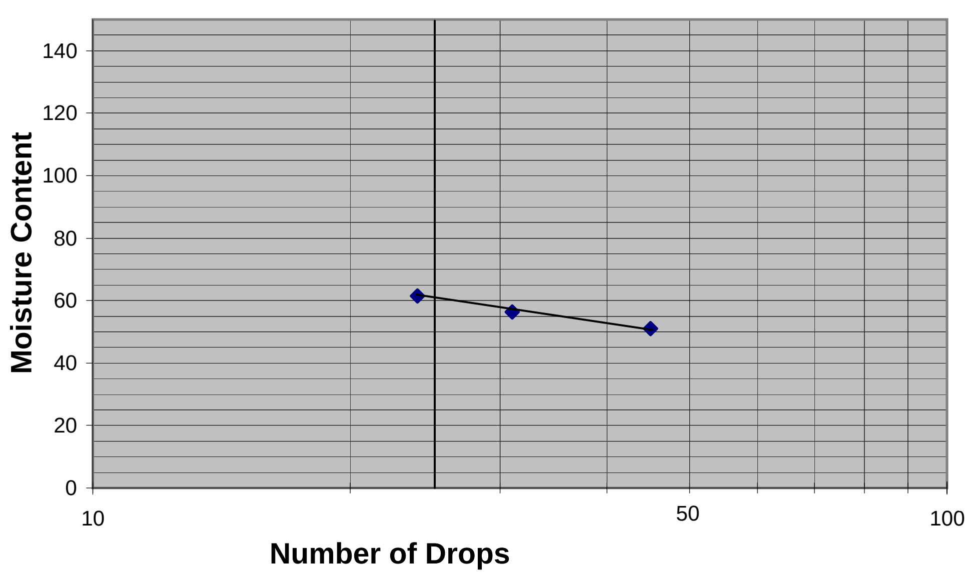
Project #:	80732-003-01	**Tested By:**	KS	**Date:**	5/20/2006

ATTERBURG LIMITS
LIQUID LIMIT / PLASTIC LIMIT / INDEX

Project Name: SixTen Franklin Building/Project # 80732-003-01 Date: 5/3/2006

Sample #: B-5 / 13 Tested By: KS

	LIQUID LIMIT			PLASTIC LIMIT	
No. of Blows	45	31	24		
Container No.	G-48	G-40	G-36	G-39	G-11
Container + wet sample	27.01	32.14	29.20	23.35	23.72
Container + dry sample	24.90	27.99	26.00	22.83	23.10
Wt. of water lost	2.11	4.15	3.20	0.52	0.62
Container weight	20.77	20.63	20.79	20.62	20.77
Weight of dry soil	4.13	7.36	5.21	2.21	2.33
Percent Moisture	51.1	56.4	61.4	23.5	26.6



Liquid Limit	61
Plastic Limit	25
Plasticity Index	36

ATTERBURG LIMITS
LIQUID LIMIT / PLASTIC LIMIT / INDEX

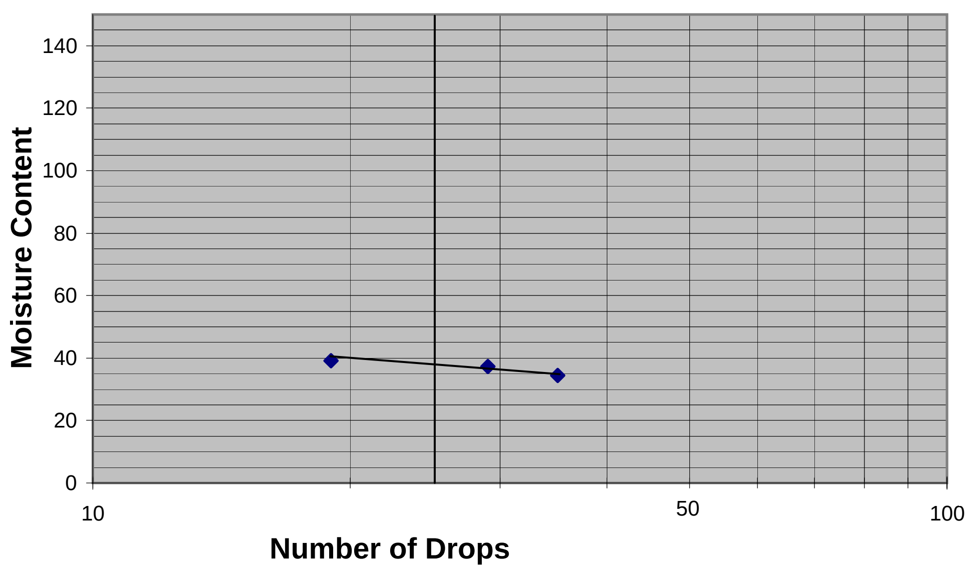
Project Name: SixTen Franklin Building/Project # 80732-003-01

Date: 5/3/2006

Sample #: B-7 / 30

Tested By: KS

	LIQUID LIMIT			PLASTIC LIMIT	
No. of Blows	35	29	19		
Container No.	G-22	G-46	G-44	G-37	G-34
Container + wet sample	26.60	27.18	28.45	26.93	27.31
Container + dry sample	25.14	25.46	26.27	26.00	26.31
Wt. of water lost	1.46	1.72	2.18	0.93	1.00
Container weight	20.90	20.85	20.71	20.96	20.85
Weight of dry soil	4.24	4.61	5.56	5.04	5.46
Percent Moisture	34.4	37.3	39.2	18.5	18.3



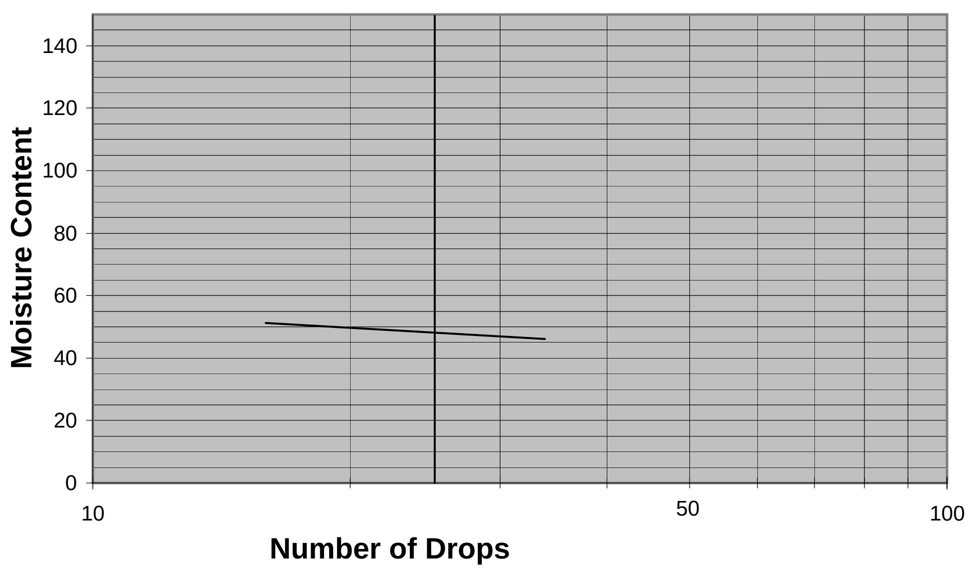
Liquid Limit	37
Plastic Limit	18
Plasticity Index	19

ATTERBURG LIMITS
LIQUID LIMIT / PLASTIC LIMIT / INDEX

Project Name: SixTen Franklin Building/Project # 80732-003-01 Date: 5/3/2006

Sample #: B-7 / 9 Tested By: KS

	LIQUID LIMIT			PLASTIC LIMIT	
No. of Blows	41	31	16		
Container No.	G-28	G-16	G-3	G-4	G-41
Container + wet sample	25.57	26.02	26.47	23.89	23.73
Container + dry sample	22.43	22.50	22.60	22.54	22.39
Wt. of water lost	3.14	3.52	3.87	1.35	1.34
Container weight	20.75	20.71	20.73	20.79	20.58
Weight of dry soil	1.68	1.79	1.87	1.75	1.81
Percent Moisture	186.9	196.6	207.0	77.1	74.0



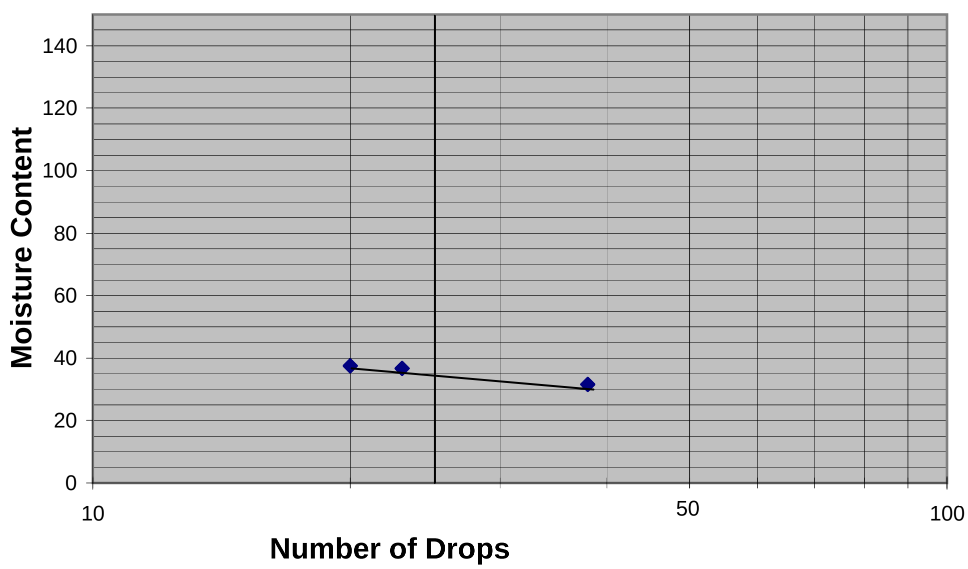
Liquid Limit	199
Plastic Limit	76
Plasticity Index	124

ATTERBURG LIMITS
LIQUID LIMIT / PLASTIC LIMIT / INDEX

Project Name: SixTen Franklin Building/Project # 80732-003-01 Date: 5/3/2006

Sample #: B-5 / 22 Tested By: KS

	LIQUID LIMIT			PLASTIC LIMIT	
No. of Blows	38	23	20		
Container No.	G-34	G-11	G-37	G-36	G-26
Container + wet sample	26.68	29.53	27.92	24.94	24.10
Container + dry sample	25.28	27.18	26.02	24.37	23.63
Wt. of water lost	1.40	2.35	1.90	0.57	0.47
Container weight	20.85	20.77	20.95	20.79	20.76
Weight of dry soil	4.43	6.41	5.07	3.58	2.87
Percent Moisture	31.6	36.7	37.5	15.9	16.4



Liquid Limit	36
Plastic Limit	16
Plasticity Index	20

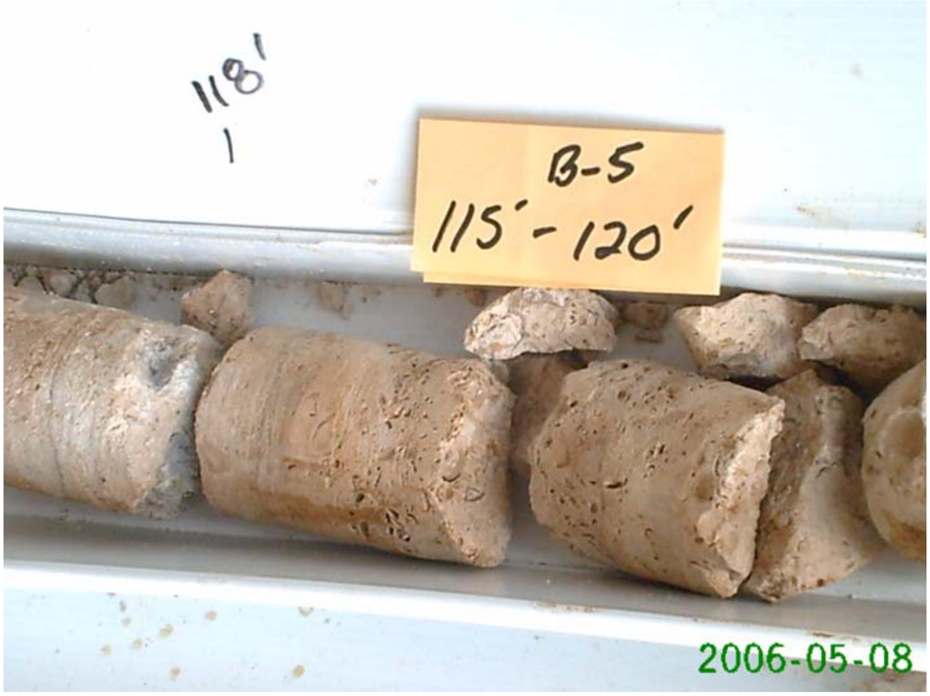
APPENDIX C

B-5 CORE RUN @ 44 FEET

B-5 CORE RUN @ 44 TO 45 FEET

B-5 CORE RUN @ 51 FEET

B-5 CORE RUN @ 51 FEET



B-5 CORE RUN @ 118 FEET

B-5 CORE RUN @ 119 FEET

B-5 CORE RUN @ 120 FEET

B-5 CORE RUN @ 120 FEET

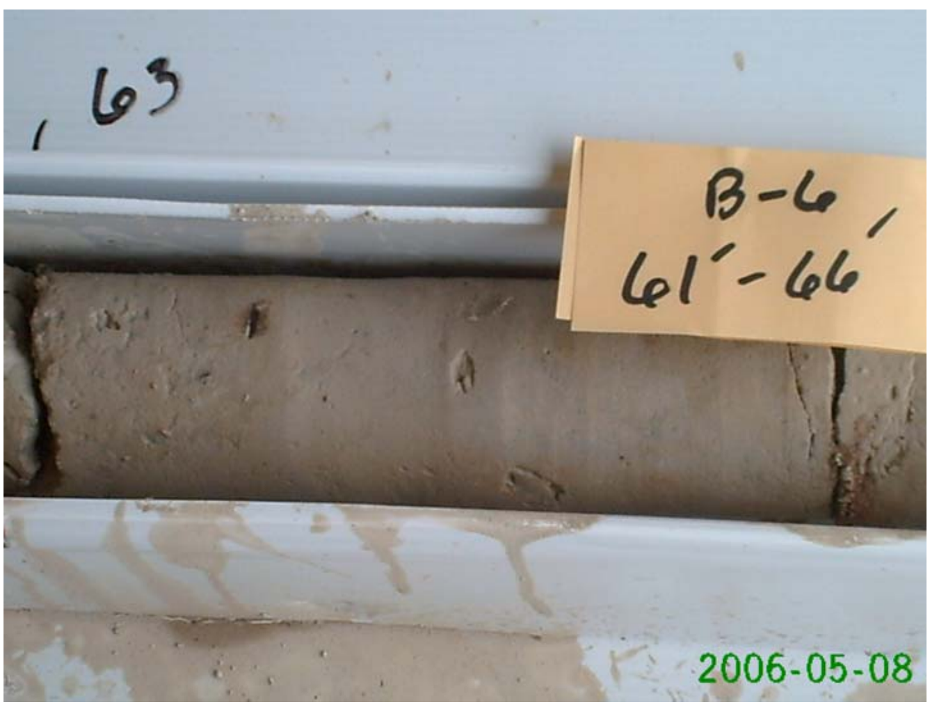
B-6 CORE RUN @ 55 FEET

B-6 CORE RUN @ 53 TO 54 FEET

B-6 CORE RUN @ 58 FEET

B-6 CORE RUN @ 58 FEET

B-6 CORE RUN @ 63 TO 64 FEET



B-6 CORE RUN @ 63 FEET

B-7 CORE RUN @ 48 FEET

B-7 CORE RUN @ 48 FEET (TOP) & 75 FEET (BOTTOM)

B-7 CORE RUN @ 75 FEET

B-7 CORE RUN @ 75 FEET

B-7 CORE RUN @ 82 FEET (TOP) & 87 FEET (BOTTOM)

B-7 CORE RUN @ 82 FEET (TOP) & 87 FEET (BOTTOM)

B-7 CORE RUN @ 92 FEET

B-7 CORE RUN @ 92 TO 93.5 FEET

APPENDIX D

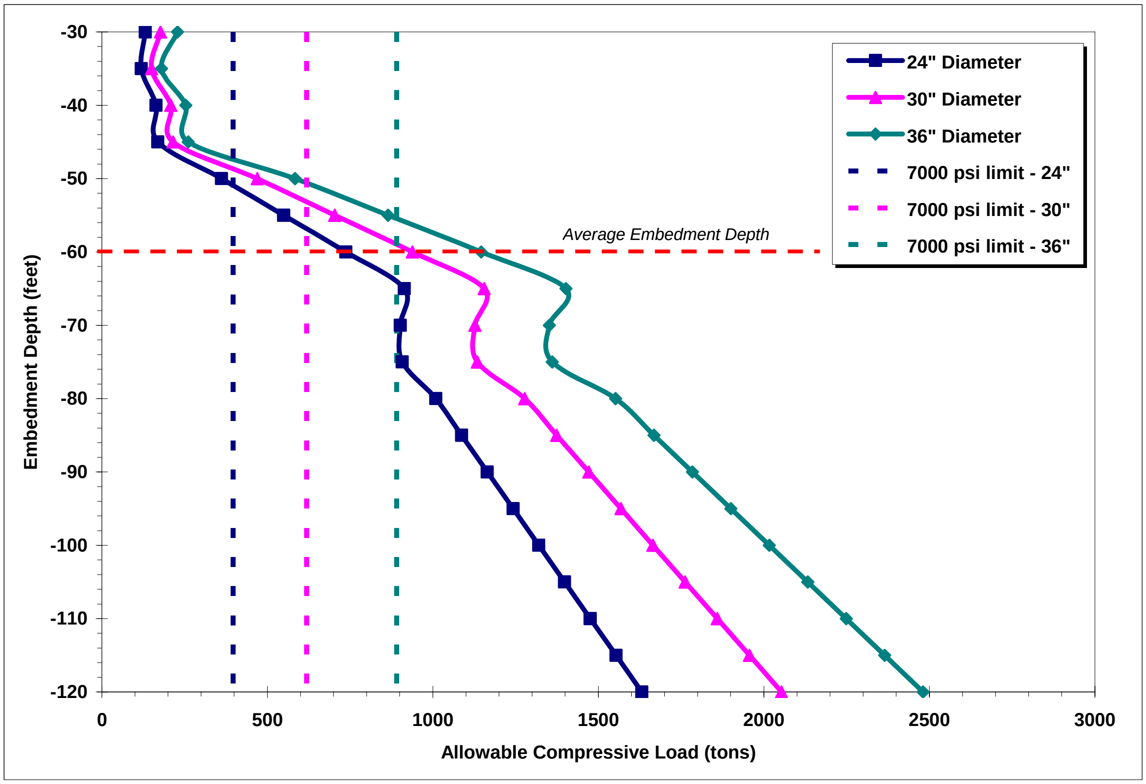


Figure 1 Axial Capacities @ Boring B-01

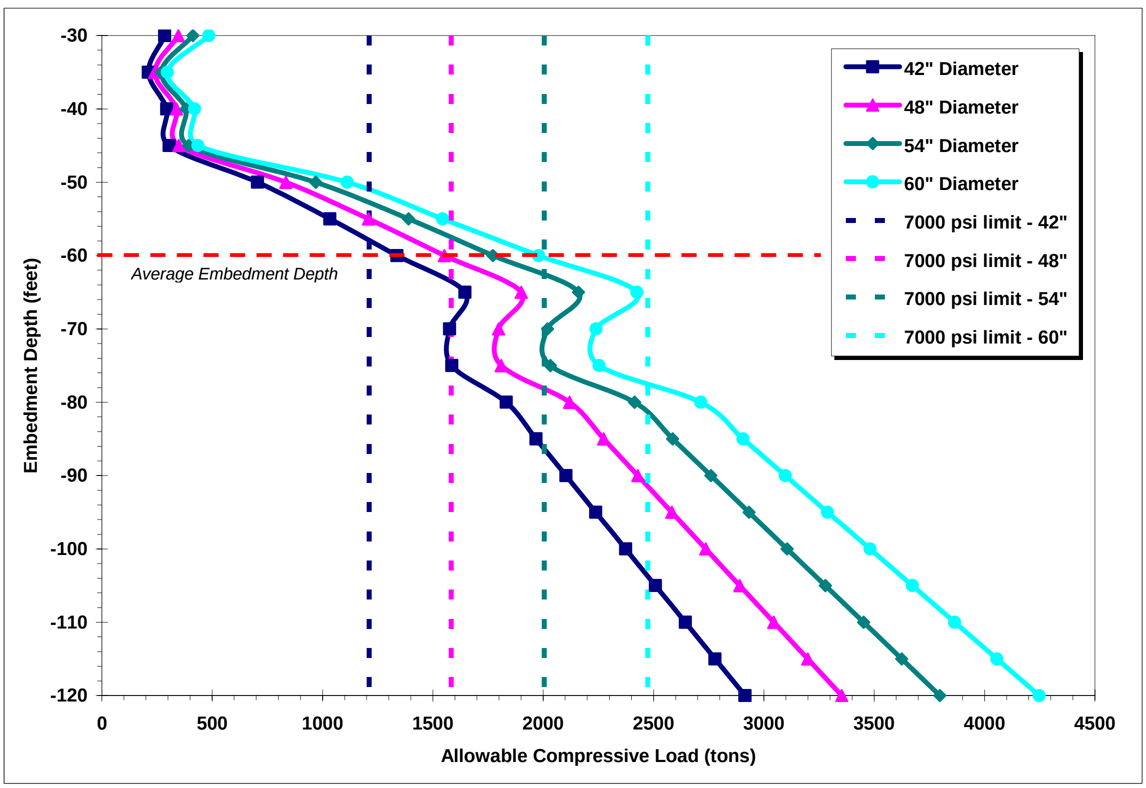


Figure 2 Axial Capacities @ Boring B-01

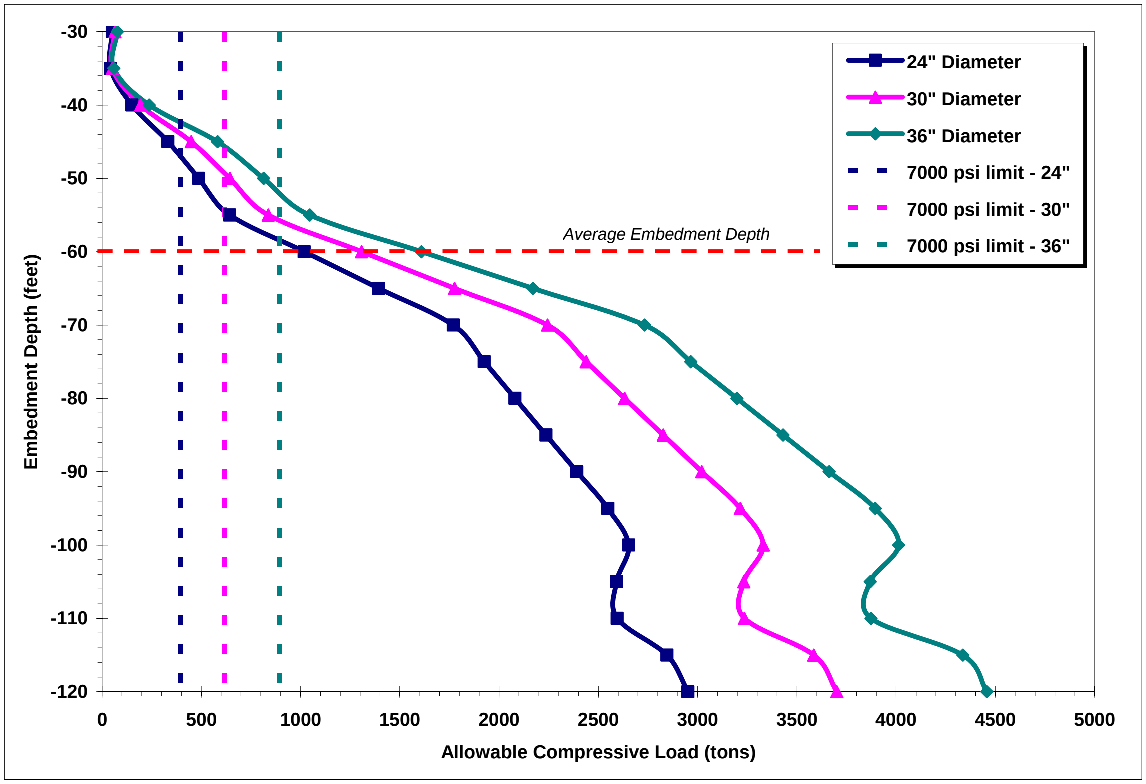


Figure 3 Axial Capacities @ Boring B-07

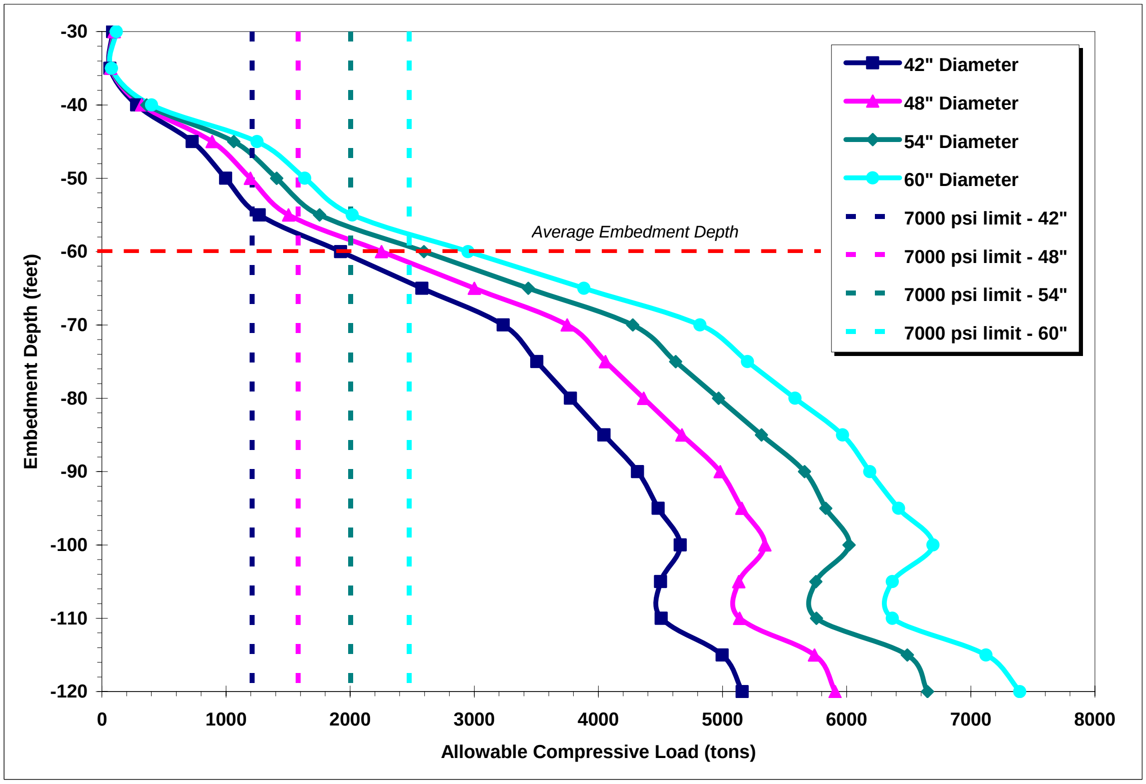
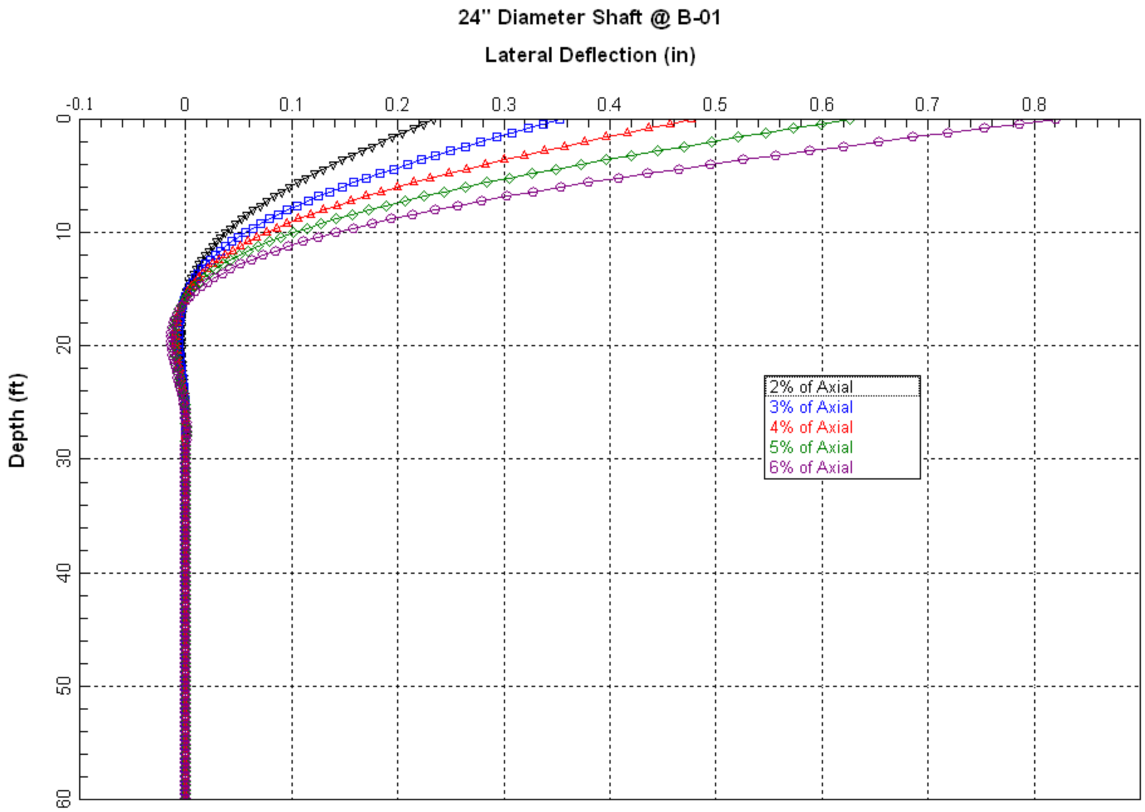
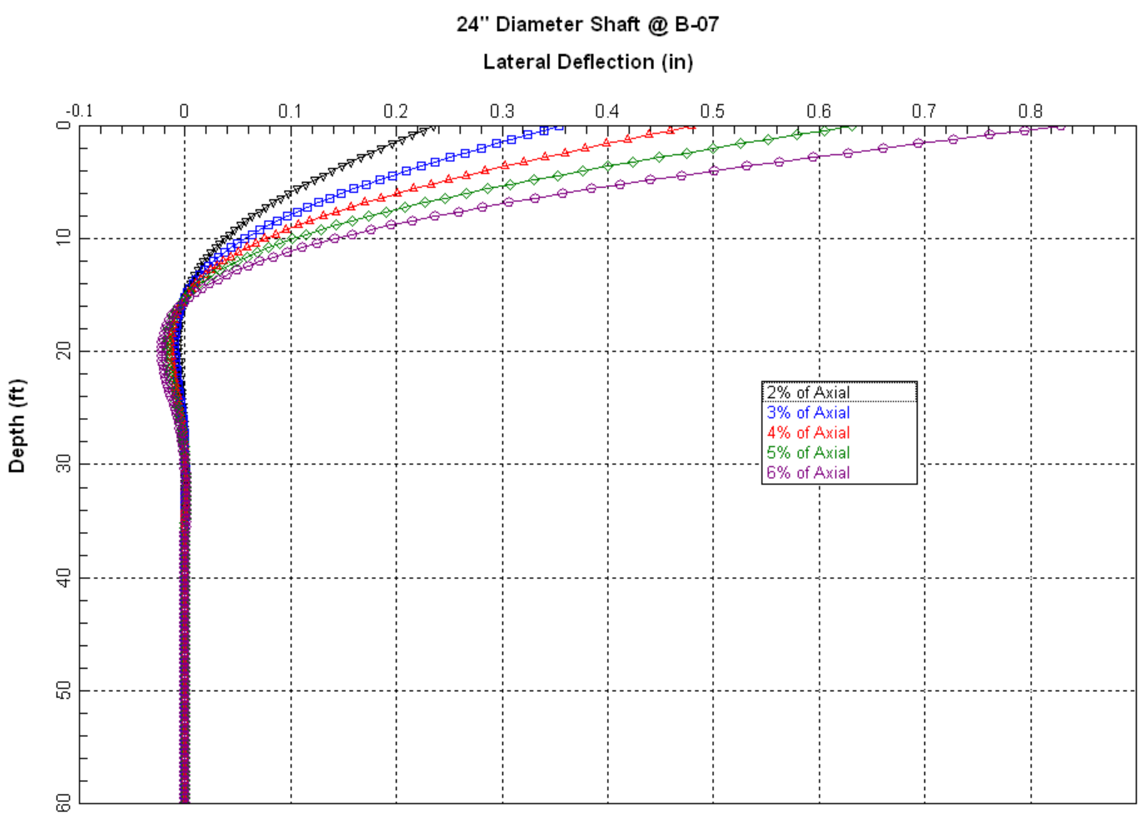
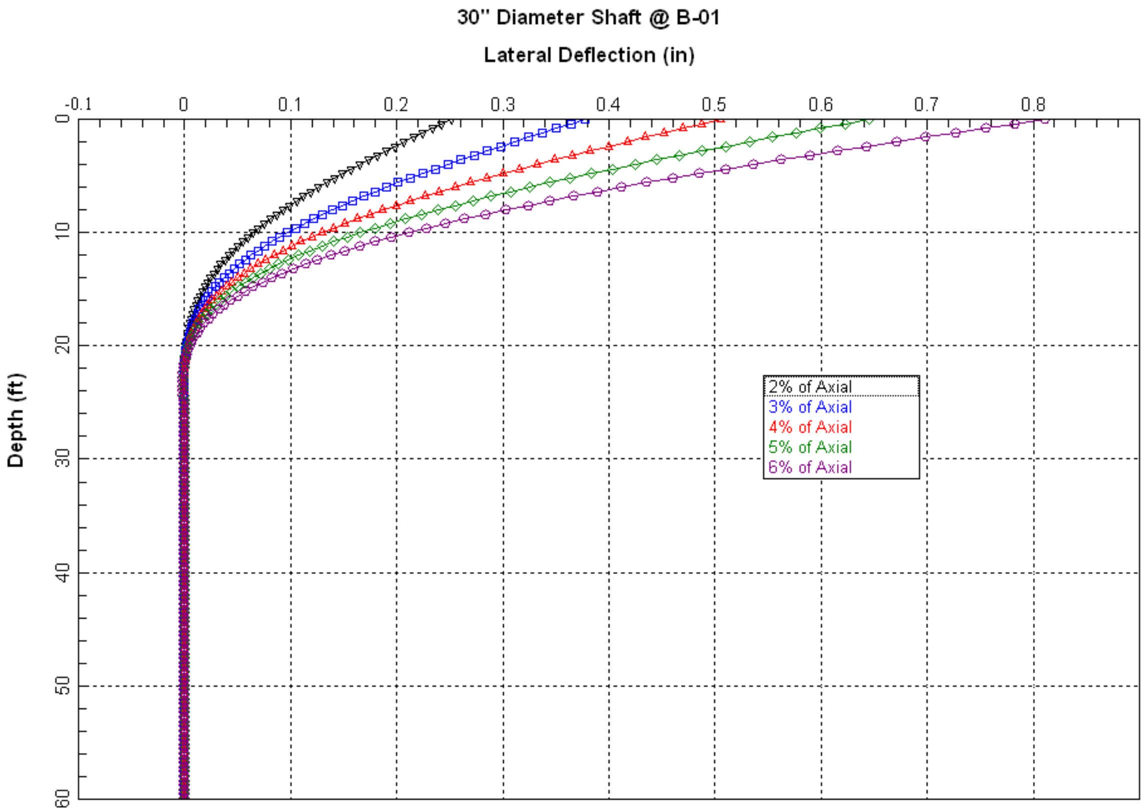
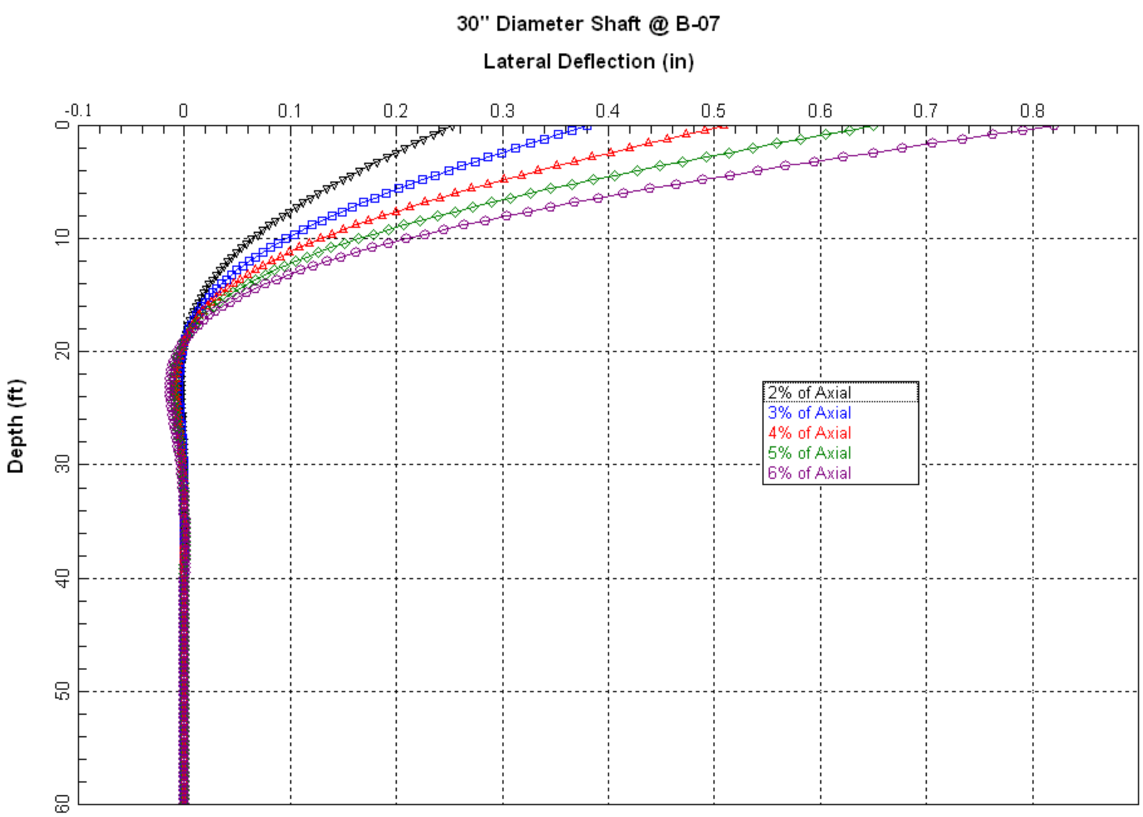


Figure 4 Axial Capacities @ Boring B-07

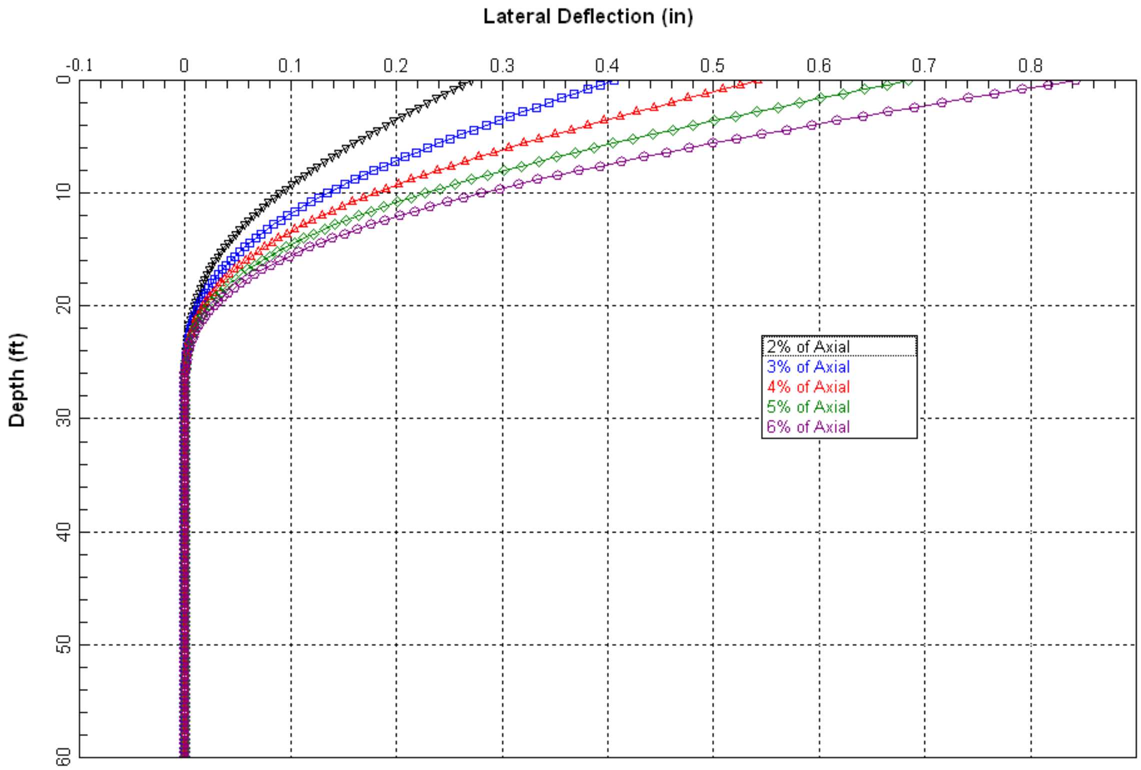

24" Diameter Shaft @ B-01
Lateral Deflection (in)
Depth (ft)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial


24" Diameter Shaft @ B-07
Lateral Deflection (in)
Depth (ft)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

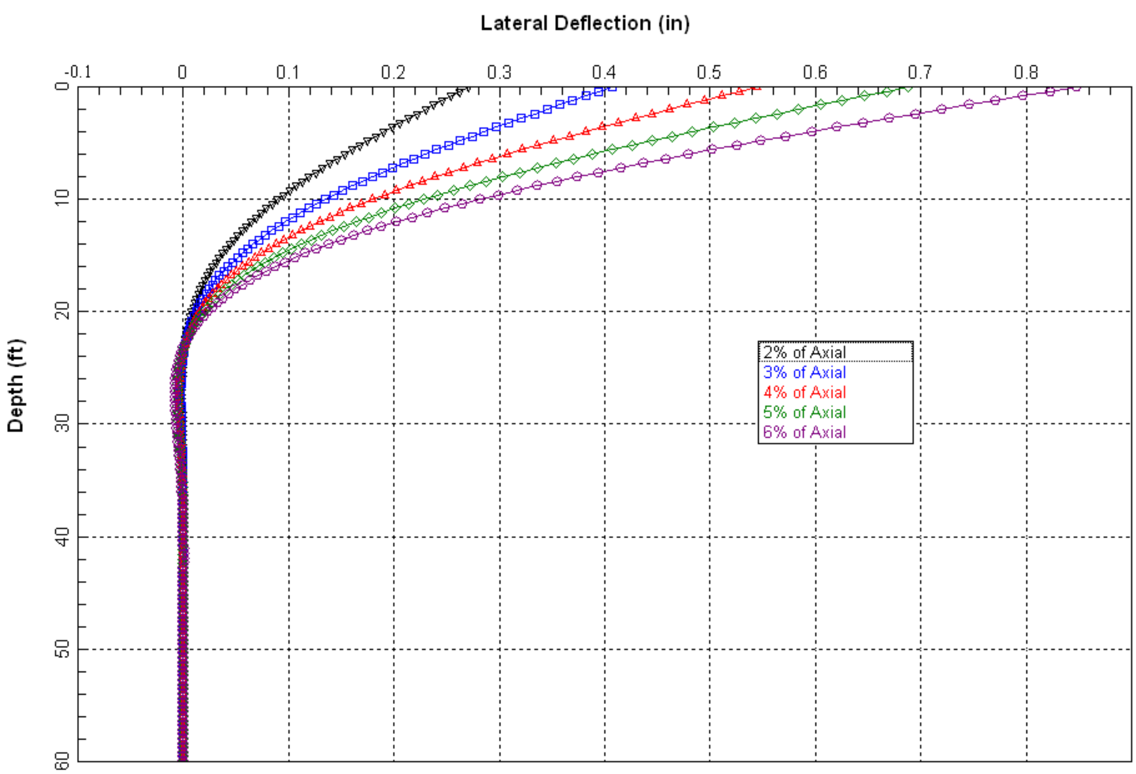

30" Diameter Shaft @ B-01
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial


30" Diameter Shaft @ B-07
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

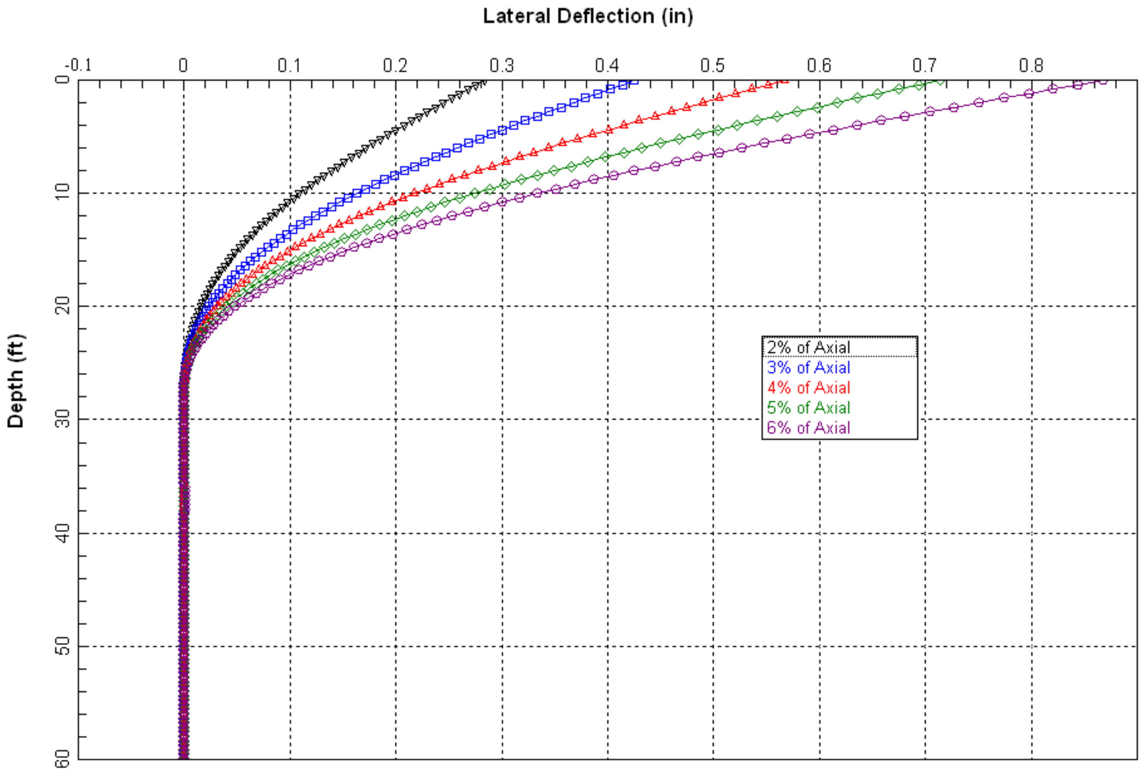
36" Diameter Shaft @ B-01



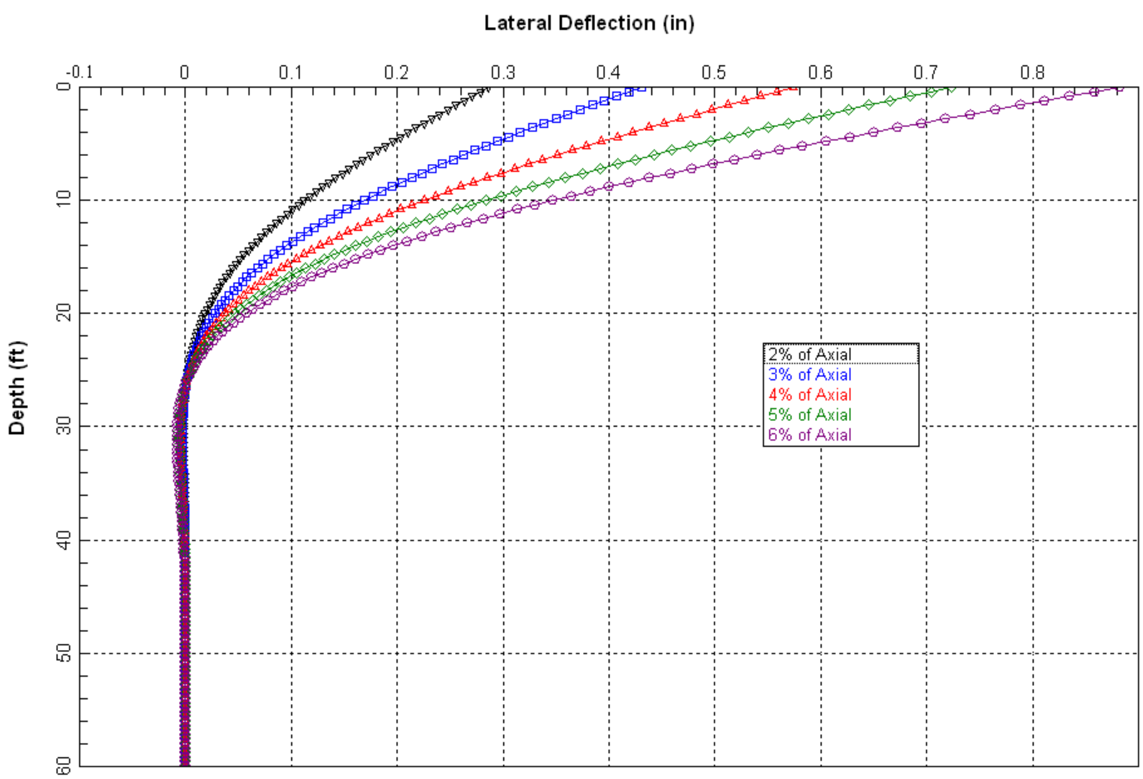
36" Diameter Shaft @ B-07

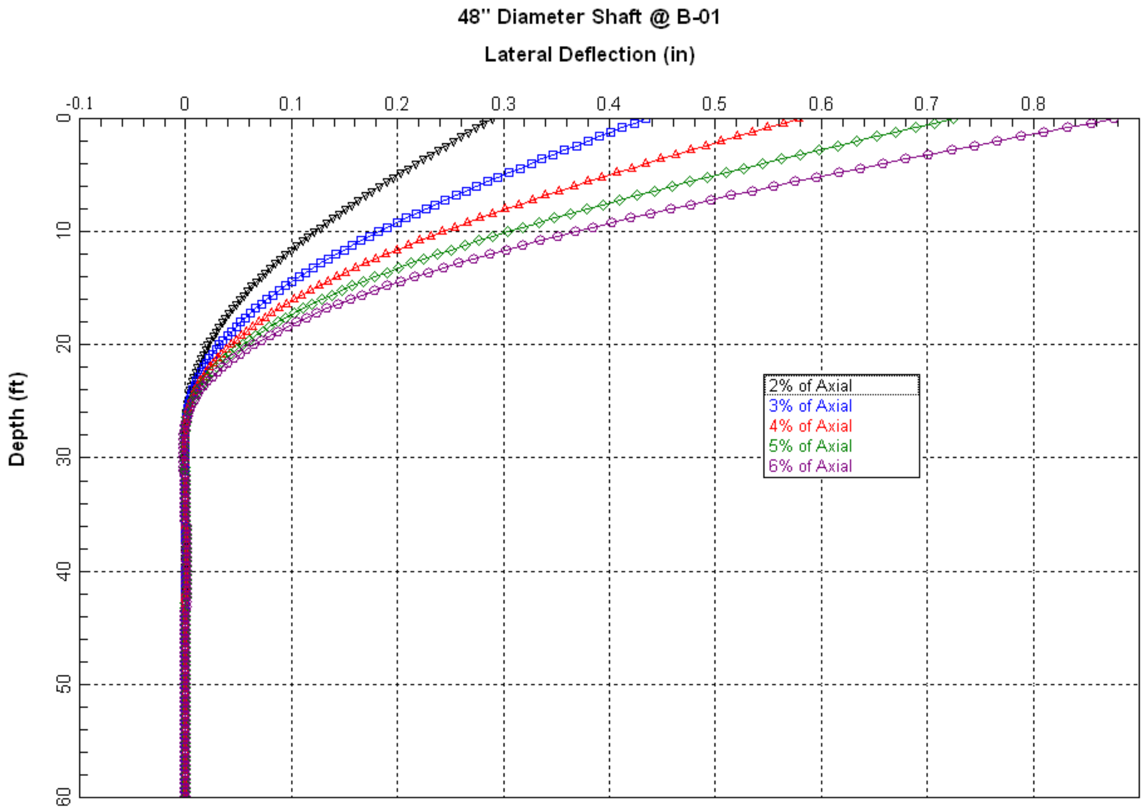


42" Diameter Shaft @ B-01

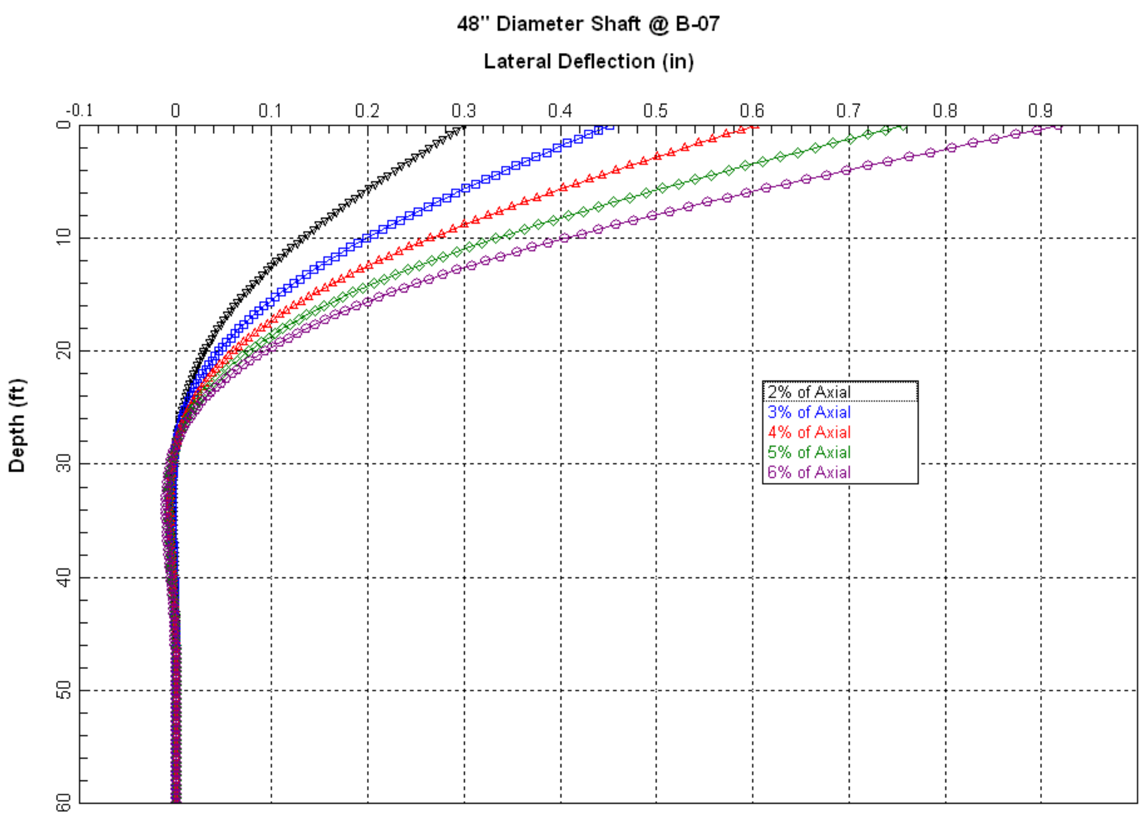


42" Diameter Shaft @ B-07

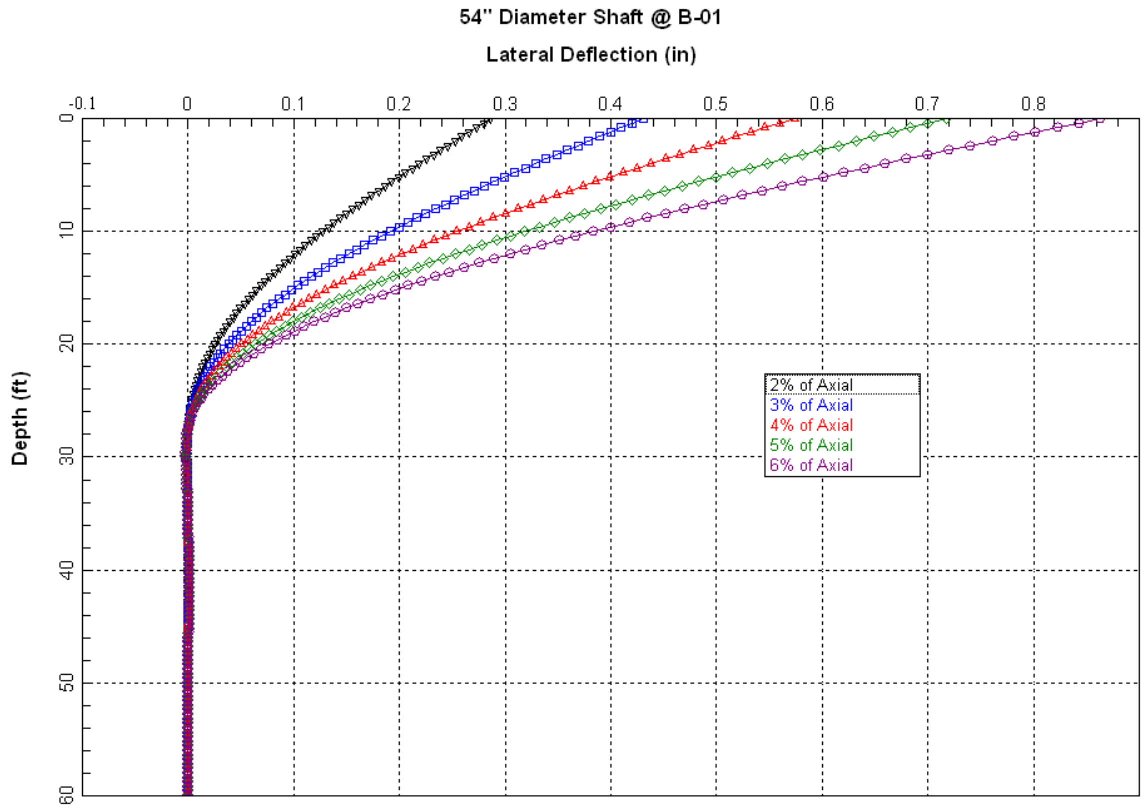


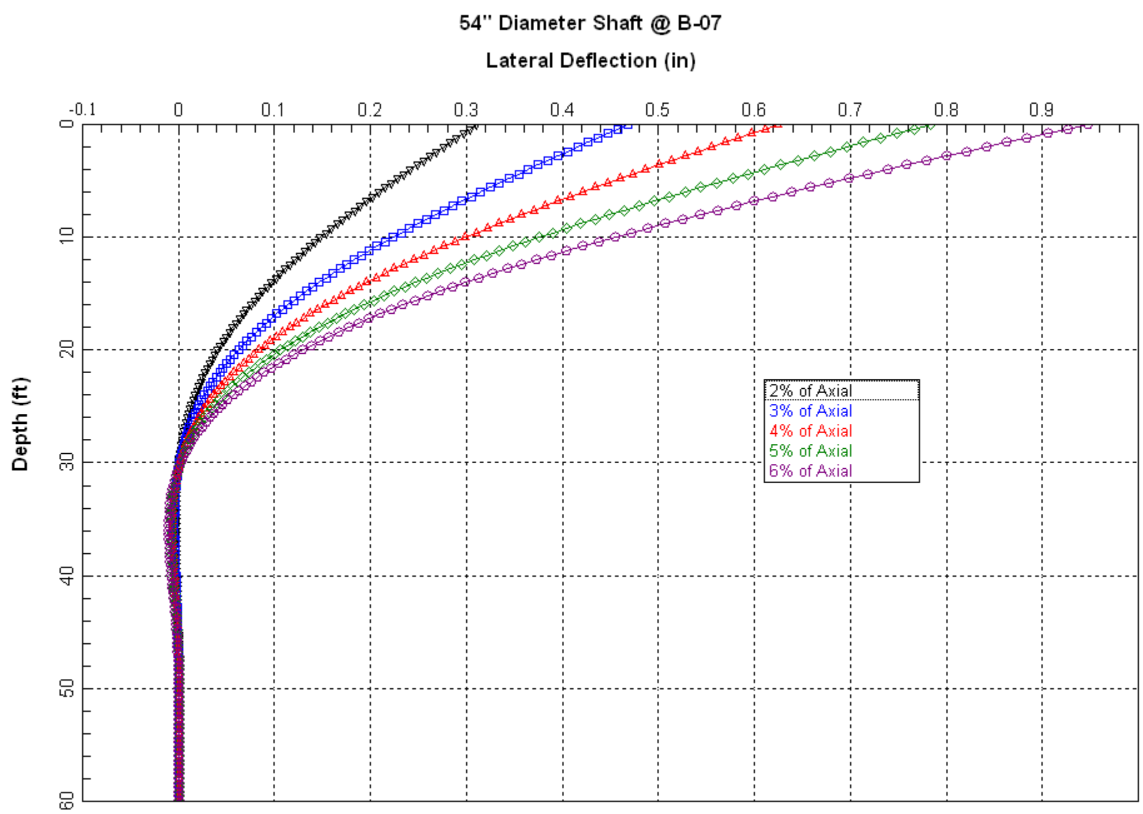


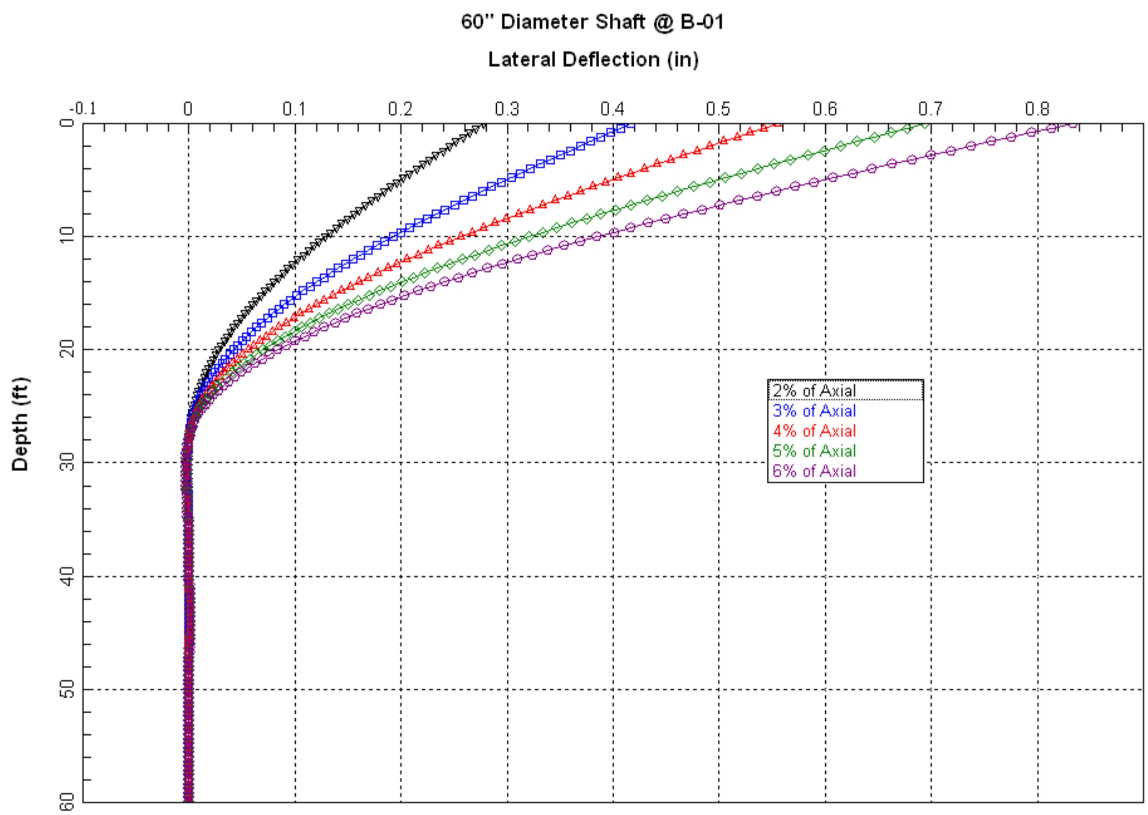
48" Diameter Shaft @ B-01
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

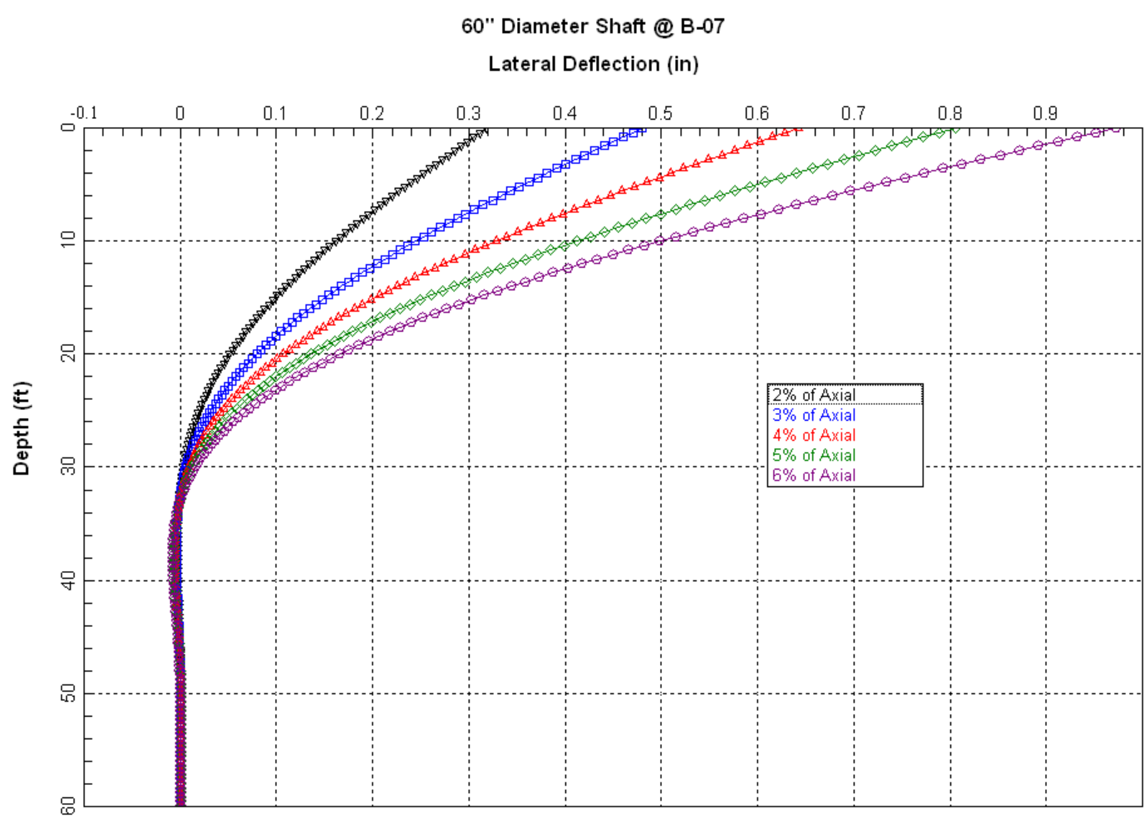


48" Diameter Shaft @ B-07
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial


54" Diameter Shaft @ B-01
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

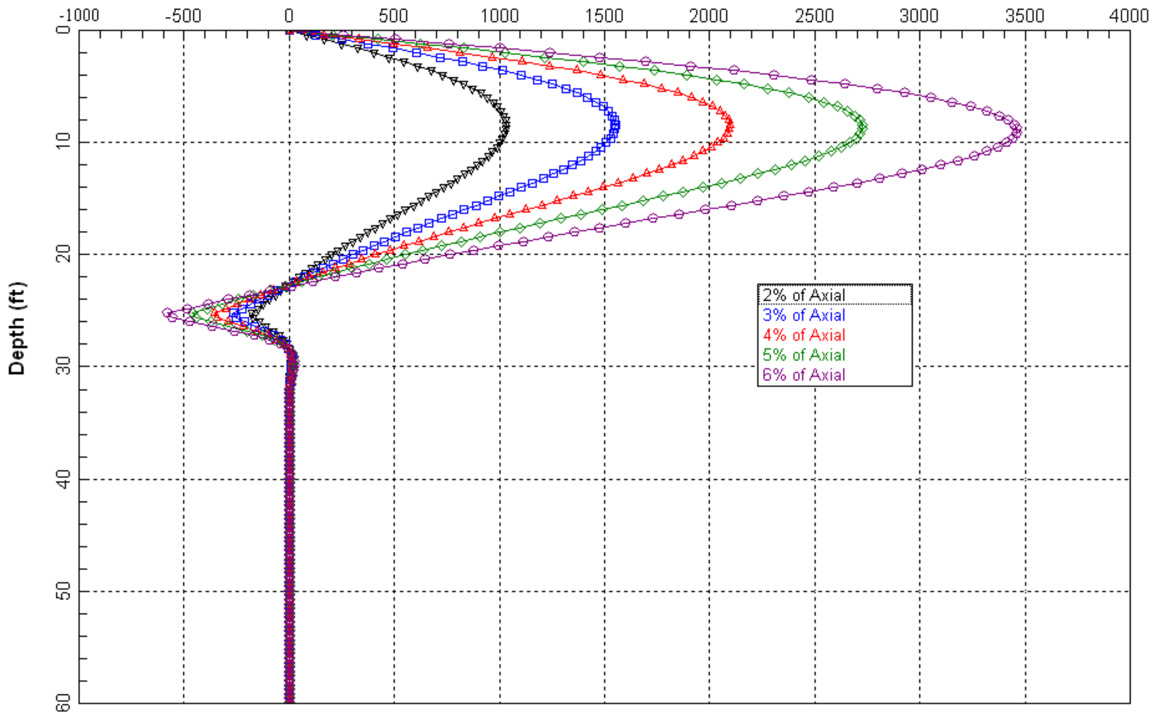

54" Diameter Shaft @ B-07
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial


60" Diameter Shaft @ B-01
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial


60" Diameter Shaft @ B-07
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
10
20
30
40
50
60
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

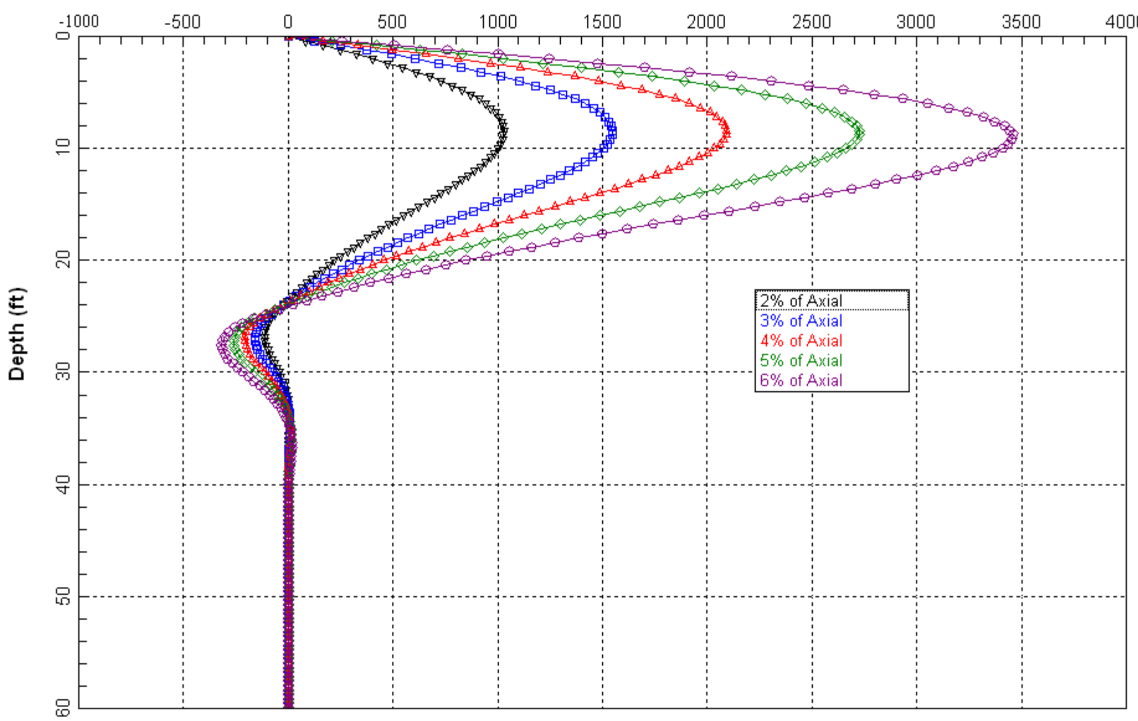
24" Diameter Shaft @ B-01

Unfactored Bending Moment (in-kips)



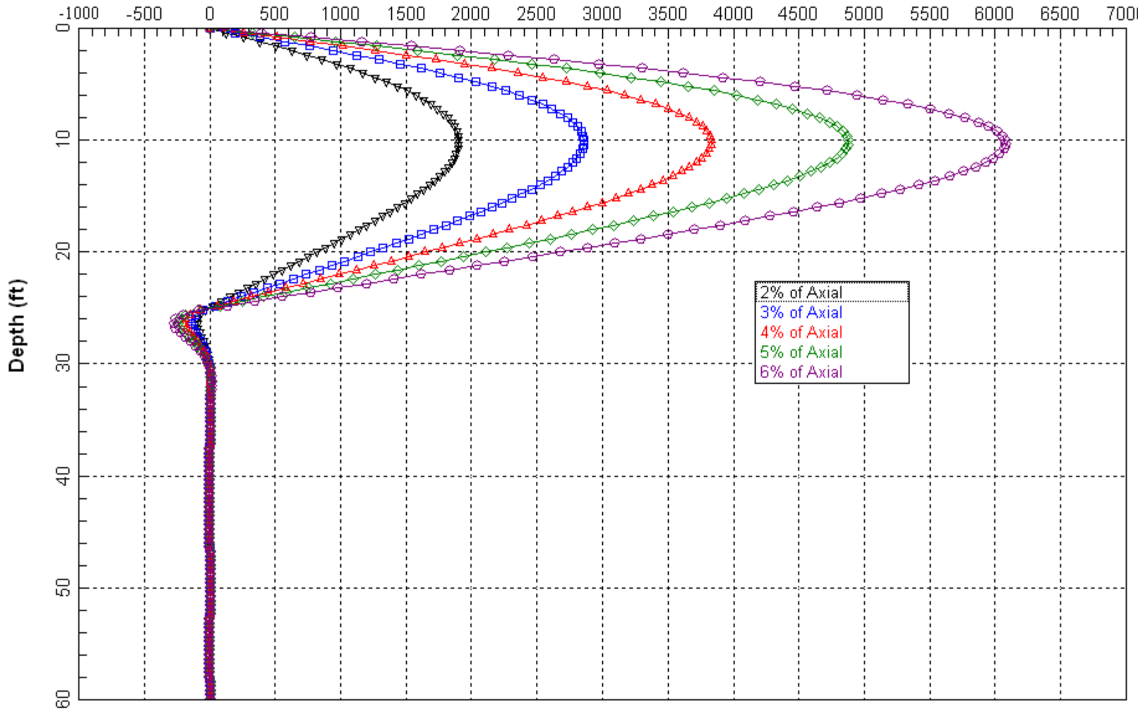
24" Diameter Shaft @ B-07

Unfactored Bending Moment (in-kips)



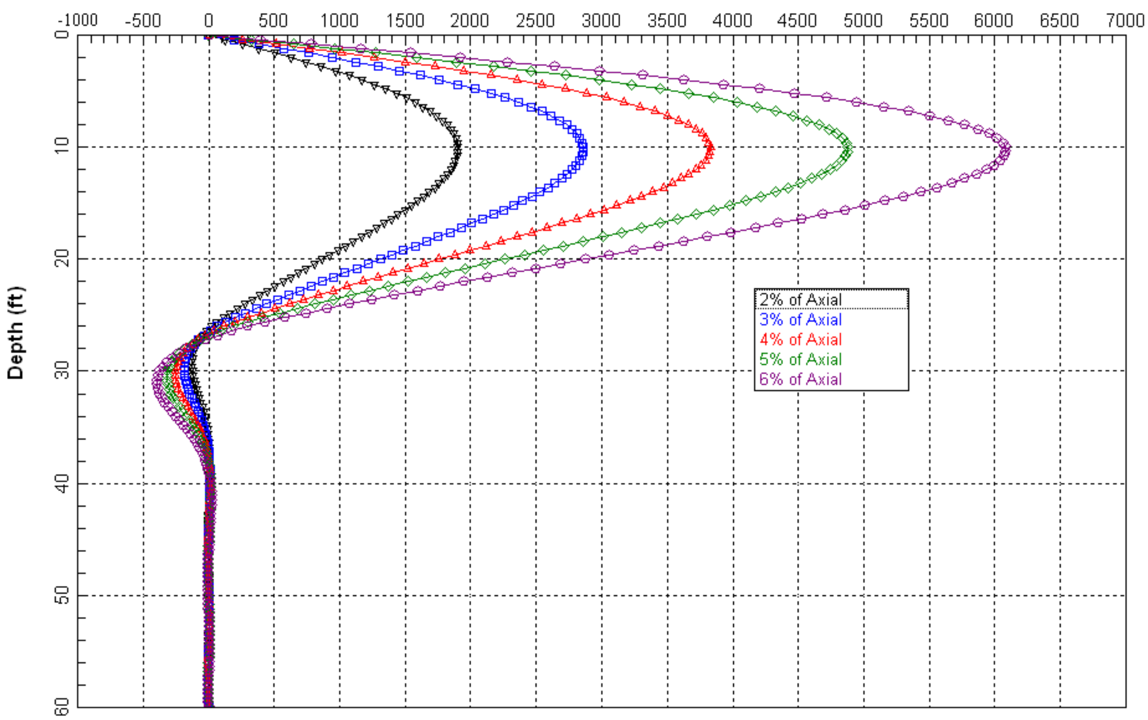
30" Diameter Shaft @ Boring B-01

Unfactored Bending Moment (in-kips)



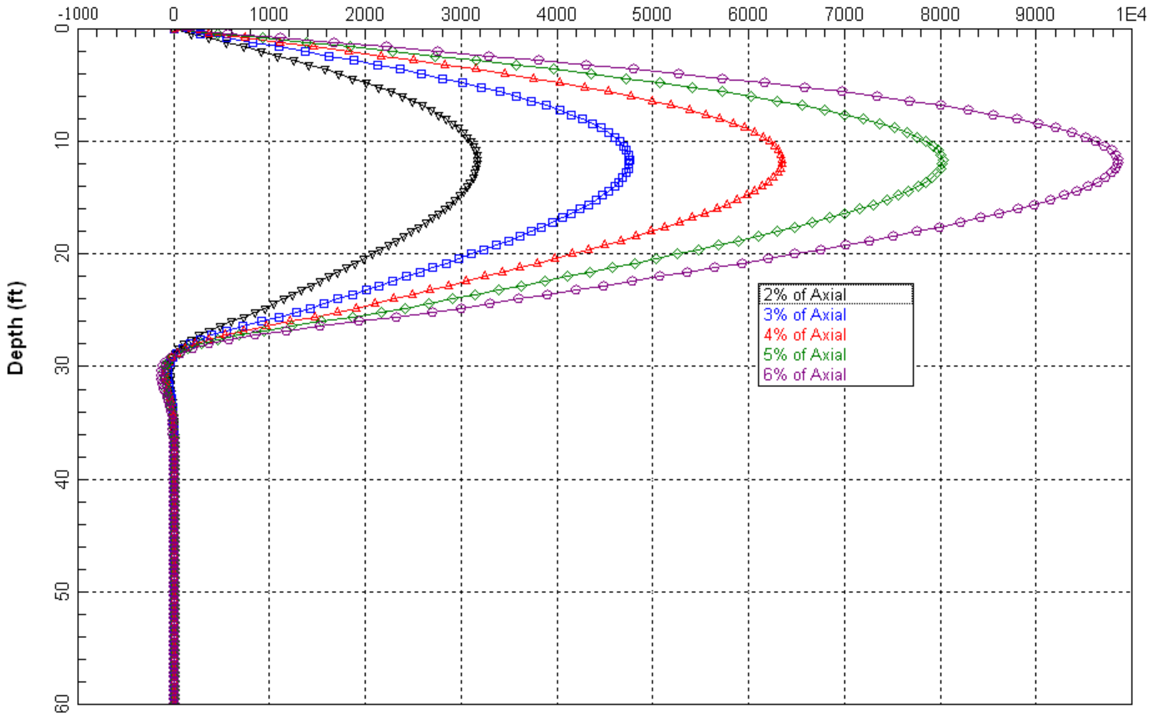
30" Diameter Shaft @ Boring B-07

Unfactored Bending Moment (in-kips)



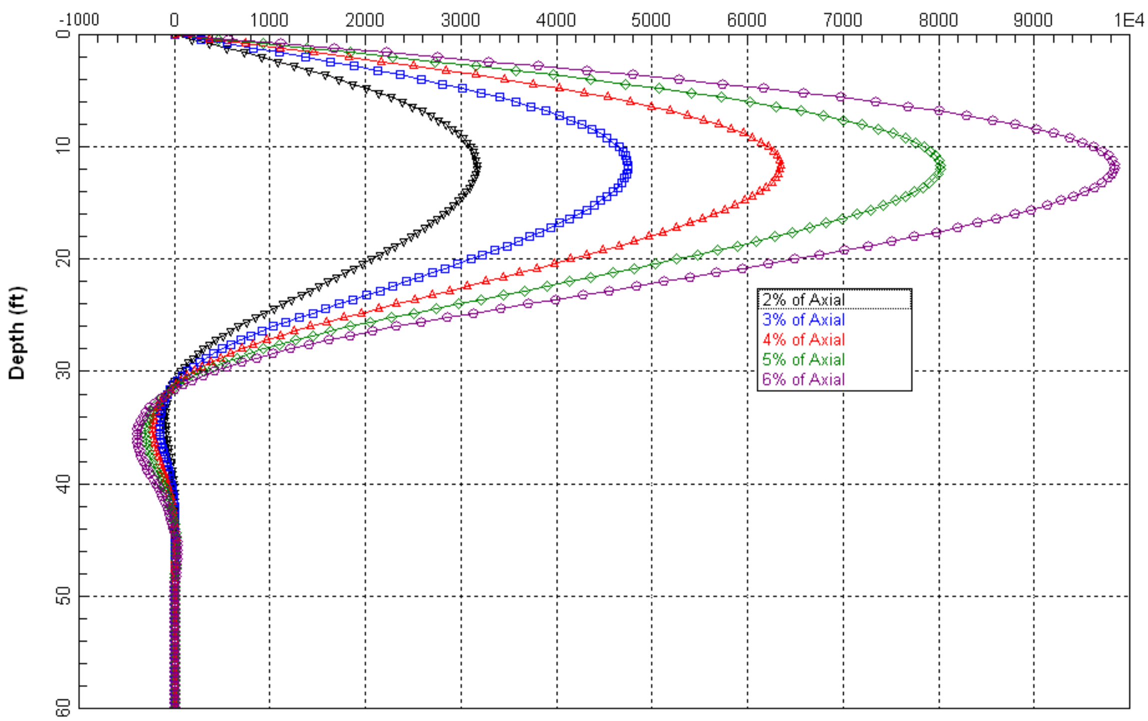
36" Diameter Shaft @ Boring B-01

Unfactored Bending Moment (in-kips)



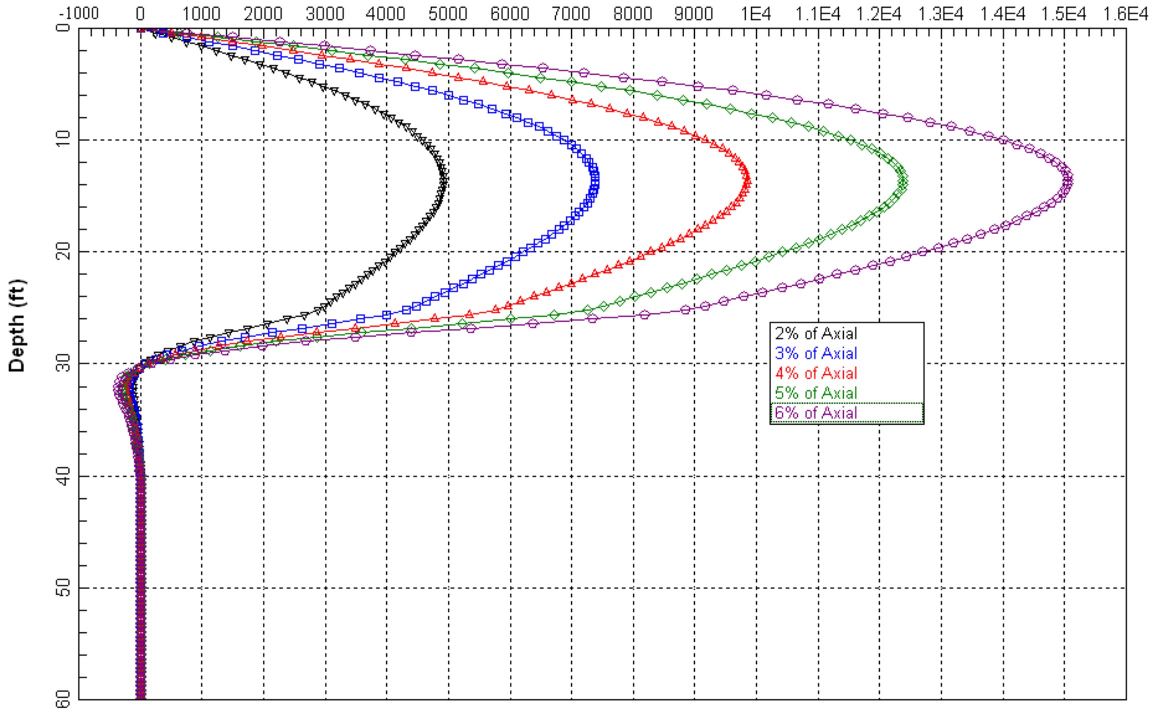
36" Diameter Shaft @ Boring B-07

Unfactored Bending Moment (in-kips)



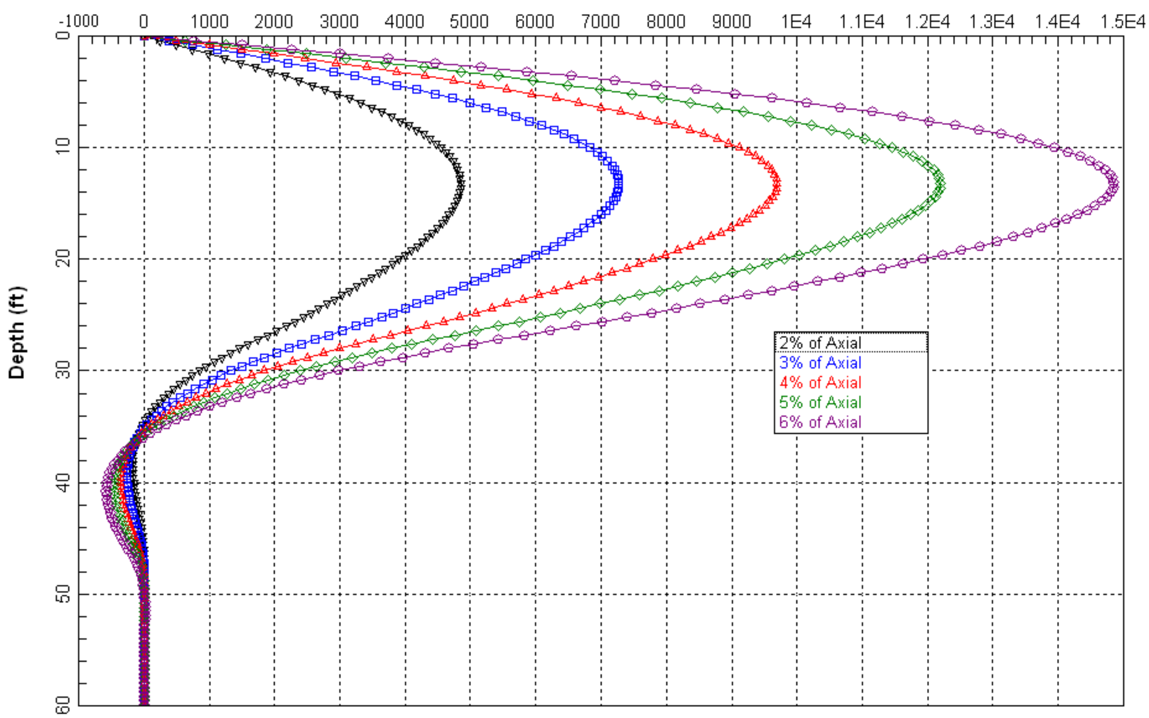
42" Diameter Shaft @ Boring B-01

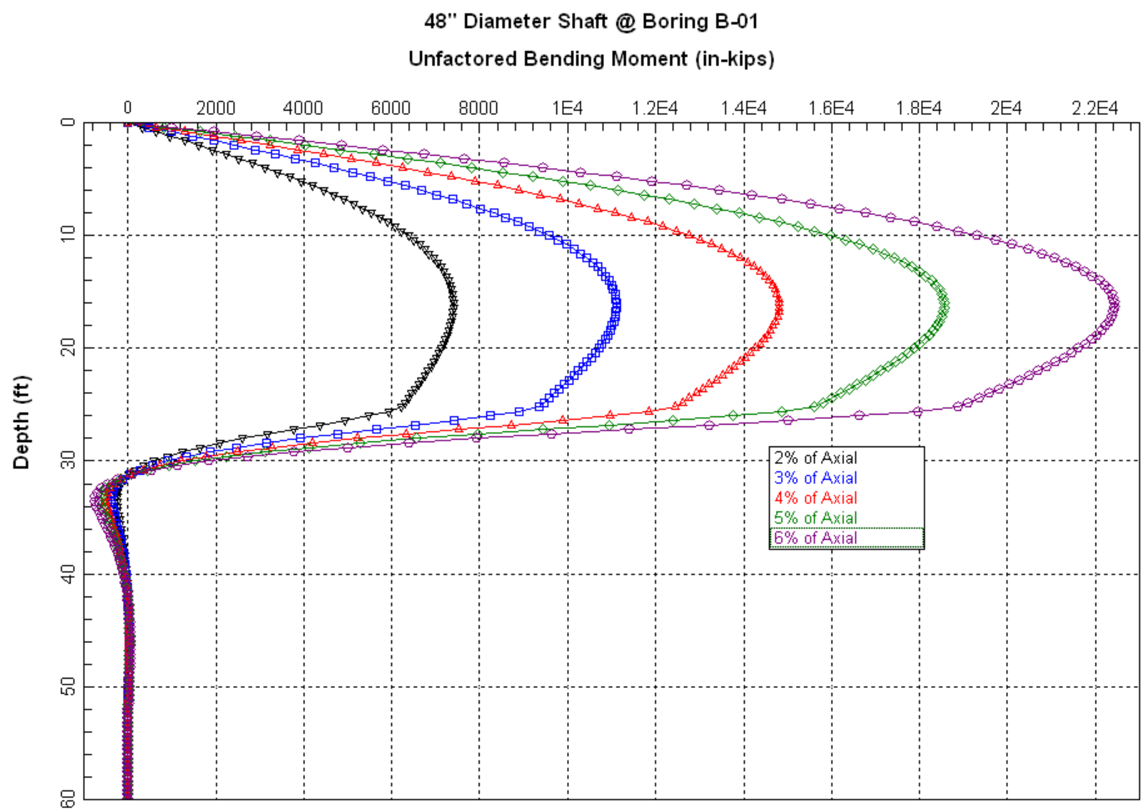
Unfactored Bending Moment (in-kips)



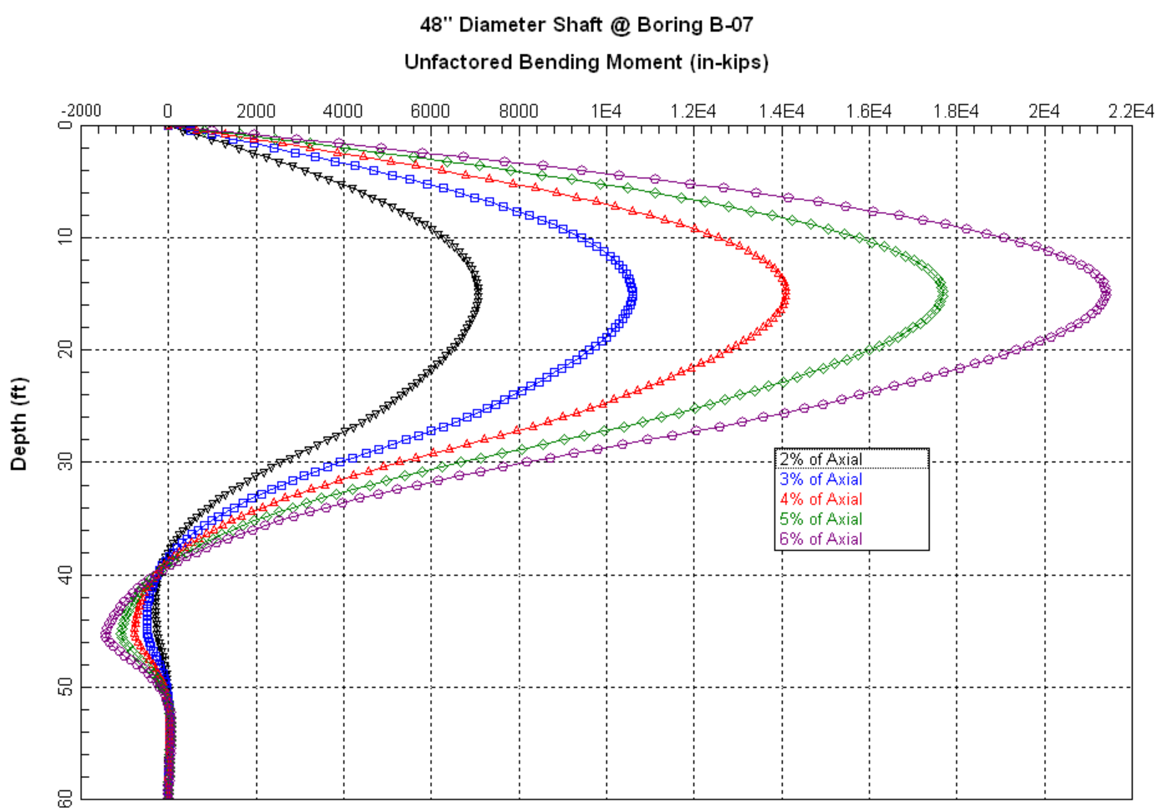
42" Diameter Shaft @ Boring B-07

Unfactored Bending Moment (in-kips)





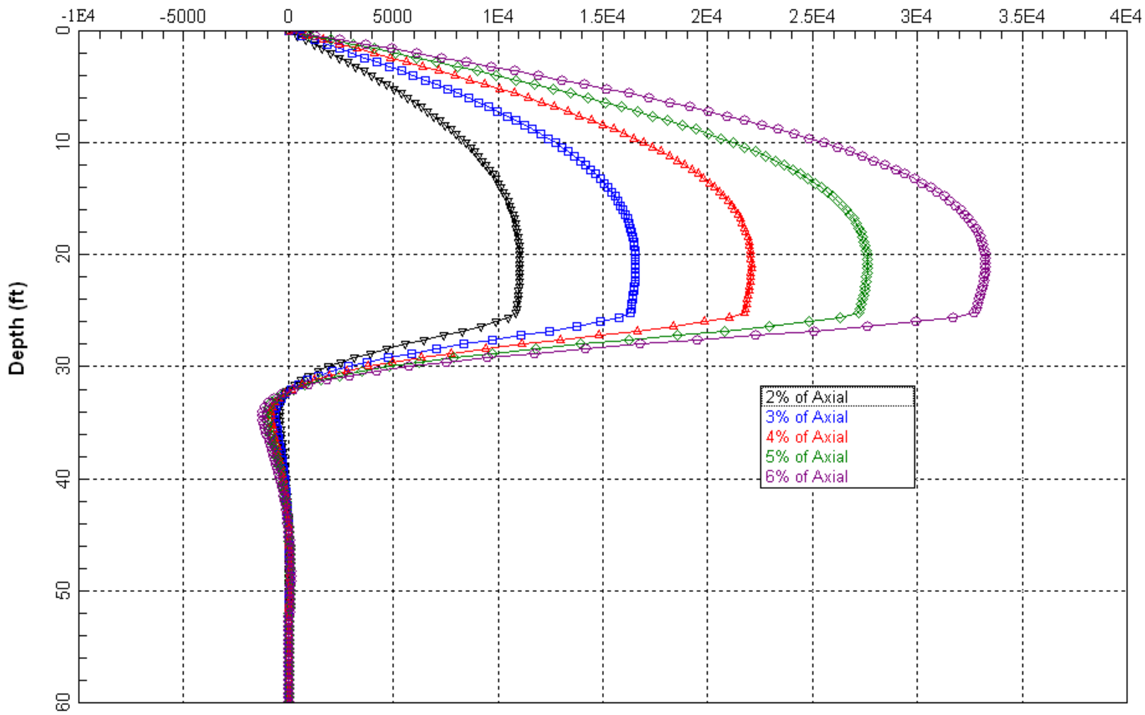
48" Diameter Shaft @ Boring B-01
Unfactored Bending Moment (in-kips)
Depth (ft)
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial



48" Diameter Shaft @ Boring B-07
Unfactored Bending Moment (in-kips)
Depth (ft)
2% of Axial
3% of Axial
4% of Axial
5% of Axial
6% of Axial

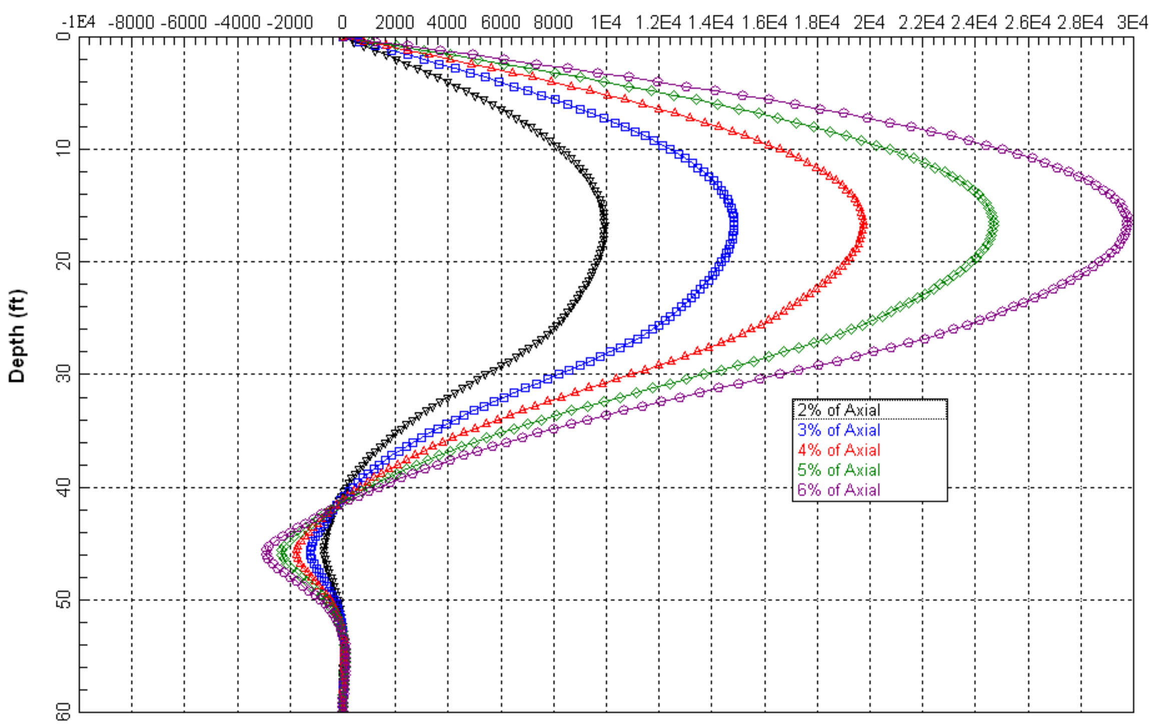
54" Diameter Shaft @ Boring B-01

Unfactored Bending Moment (in-kips)



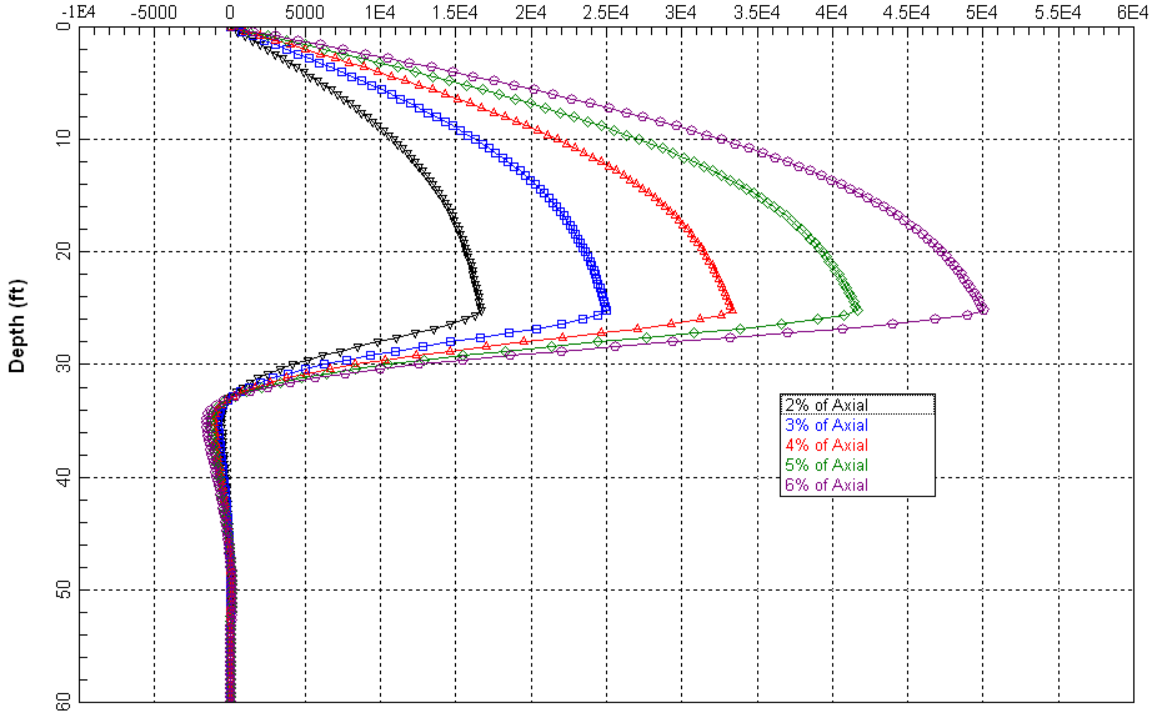
54" Diameter Shaft @ Boring B-07

Unfactored Bending Moment (in-kips)



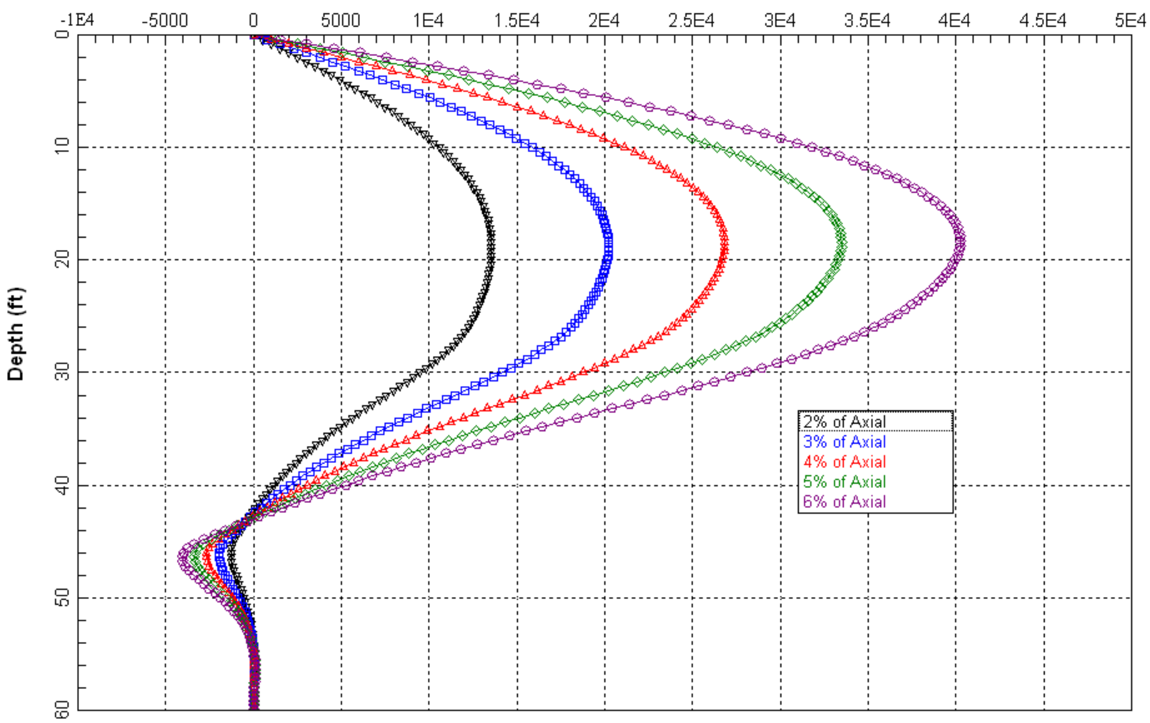
60" Diameter Shaft @ Boring B-01

Unfactored Bending Moment (in-kips)



60" Diameter Shaft @ Boring B-07

Unfactored Bending Moment (in-kips)



APPENDIX E

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

8811 Colesville Road Suite G106 Silver Spring, MD 20910
Telephone: 301-565-2733 Facsimile: 301-589-2017
email: info@asfe.org www.asfe.org

IIGER06983.5M

CONSTRAINTS AND RESTRICTIONS

WARRANTY

Universal Engineering Sciences has prepared this report for our client for his exclusive use, in accordance with generally accepted soil and foundation engineering practices, and makes no other warranty either expressed or implied as to the professional advice provided in the report.

UNANTICIPATED SOIL CONDITIONS

The analysis and recommendations submitted in this report are based upon the data obtained from soil borings performed at the locations indicated on the Boring Location Plan. This report does not reflect any variations which may occur between these borings.

The nature and extent of variations between borings may not become known until construction begins. If variations appear, we may have to re-evaluate our recommendations after performing on-site observations and noting the characteristics of any variations.

CHANGED CONDITIONS

We recommend that the specifications for the project require that the contractor immediately notify Universal Engineering Sciences, as well as the owner, when subsurface conditions are encountered that are different from those present in this report.

No claim by the contractor for any conditions differing from those anticipated in the plans, specifications, and those found in this report, should be allowed unless the contractor notifies the owner and Universal Engineering Sciences of such changed conditions. Further, we recommend that all foundation work and site improvements be observed by a representative of Universal Engineering Sciences to monitor field conditions and changes, to verify design assumptions and to evaluate and recommend any appropriate modifications to this report.

MISINTERPRETATION OF SOIL ENGINEERING REPORT

Universal Engineering Sciences is responsible for the conclusions and opinions contained within this report based upon the data relating only to the specific project and location discussed herein. If the conclusions or recommendations based upon the data presented are made by others, those conclusions or recommendations are not the responsibility of Universal Engineering Sciences.

CHANGED STRUCTURE OR LOCATION

This report was prepared in order to aid in the evaluation of this project and to assist the architect or engineer in the design of this project. If any changes in the design or location of the structure as outlined in this report are planned, or if any structures are included or added that are not discussed in the report, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions modified or approved by Universal Engineering Sciences.

USE OF REPORT BY BIDDERS

Bidders who are examining the report prior to submission of a bid are cautioned that this report was prepared as an aid to the designers of the project and it may affect actual construction operations.

Bidders are urged to make their own soil borings, test pits, test caissons or other explorations to determine those conditions that may affect construction operations. Universal Engineering Sciences cannot be responsible for any interpretations made from this report or the attached boring logs with regard to their adequacy in reflecting subsurface conditions which will affect construction operations.

STRATA CHANGES

Strata changes are indicated by a definite line on the boring logs which accompany this report. However, the actual change in the ground may be more gradual. Where changes occur between soil samples, the location of the change must necessarily be estimated using all available information and may not be shown at the exact depth.

OBSERVATIONS DURING DRILLING

Attempts are made to detect and/or identify occurrences during drilling and sampling, such as: water level, boulders, zones of lost circulation, relative ease or resistance to drilling progress, unusual sample recovery, variation of driving resistance, obstructions, etc.; however, lack of mention does not preclude their presence.

WATER LEVELS

Water level readings have been made in the drill holes during drilling and they indicate normally occurring conditions. Water levels may not have been stabilized at the last reading. This data has been reviewed and interpretations made in this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, tides, and other factors not evident at the time measurements were made and reported. Since the probability of such variations is anticipated, design drawings and specifications should accommodate such possibilities and construction planning should be based upon such assumptions of variations.

LOCATION OF BURIED OBJECTS

All users of this report are cautioned that there was no requirement for Universal Engineering Sciences to attempt to locate any man-made buried objects during the course of this exploration and that no attempt was made by Universal Engineering Sciences to locate any such buried objects. Universal Engineering Sciences cannot be responsible for any buried man-made objects which are subsequently encountered during construction that are not discussed within the text of this report.

TIME

This report reflects the soil conditions at the time of exploration. If the report is not used in a reasonable amount of time, significant changes to the site may occur and additional reviews may be required.