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April 14, 2011

Mr. John E. Demby, P.E.
ESPA CORP
7120 Grand Boulevard, Suite 100
Houston, Texas 77054

Ph: (713) 680 0080
Fax: (713) 680 0738
Email: jdemby@espacorp.com

Re: **Geotechnical Engineering Services Report - Revised**
HCC San Jacinto Building Renovations at Central College Campus
1300 Holman Street
Houston, Texas

PSI Project No. 286-414R

Dear Mr. Demby:

Professional Service Industries, Inc. (PSI) is pleased to submit the revised Geotechnical Engineering Services Report for the above referenced project. Included in this revised report are the results of the geotechnical exploration, recommendations for the foundation design and recommendations regarding general site preparation procedures.

The report has been revised to include the recommendations for the site preparation in the abandoned swimming pool area and foundation recommendations for the new column footings.

We appreciate the opportunity to provide you with our geotechnical engineering services and look forward to participate in the construction phase of this project. If you have any questions concerning this report or if we may be of further service in any manner, please contact our office.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

Rupesh R. Kadam, P.E. (LA)
Project Manager

GEOTECHNICAL ENGINEERING SERVICES REPORT - REVISED

HCC SAN JACINTO BUILDING RENOVATIONS AT CENTRAL COLLEGE CAMPUS
1300 HOLMAN STREET
HOUSTON, TEXAS

PSI PROJECT NO. 286-414R

PREPARED FOR

ESPA CORP
7120 GRAND BOULEVARD, SUITE 100
HOUSTON, TEXAS 77054

APRIL 14, 2011

BY

PROFESSIONAL SERVICE INDUSTRIES, INC.
1714 MEMORIAL DRIVE, HOUSTON, TEXAS 77007. PH. 713-224-2047. FAX. 713-224-8914

Rupesh R. Kadam, P.E. (LA)
Project Manager

K. Mohan Vennalaganti, P.E.
Chief Engineer

Professional Service Industries, Inc,
DBA PSI, Inc
Registered Engineering Firm: F-3307
1901 S. Meyers Rd, Ste 400 Oakbrook Terrace, IL 60191

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PROJECT INFORMATION

PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI) has completed a subsurface exploration and geotechnical engineering analysis for the proposed renovations to the existing structures located within the HCC San Jacinto Central College Campus in Houston, Texas. PSI's scope of services was authorized by Mr. Karun Sreerama, President of ESPA Corp by signing PSI's Proposal No. 286-36736R, dated January 28, 2011.

PROJECT DESCRIPTION

The proposed project consists of renovations to the existing structures located within the HCC San Jacinto Central College Campus. The HCC Central College Campus is located in midtown area at 1300 Holman Street in Houston, Texas. The main building is a three-story steel framed structure with approximately 172,000 square feet in plan area.

The scope of the proposed renovations is to bring the existing historic structure and later additions to current standards for use as classrooms and administrative facility. The renovation is proposed as a phased construction to facilitate all the classroom needs for HCC while minimizing any temporary classroom space.

In 2001, a general level 1 assessment for the HCC College was performed. Based on the survey, the existing structures found to be in fair to poor conditions. Also, several foundation and structural distress were observed in the buildings.

The project includes construction of four (4) new stairwells, a new elevator, and addition to the existing Mechanical Central Plant Unit. Each stairwell and an elevator will be a three-story closed structure. Two (2) stairwells will be constructed at the south side of the main building, one (1) will be constructed at the east wing, and one (1) will be constructed at the auditorium area. The Mechanical Central Plant is a single story structure which will have a future addition to the south side. An existing Upholstery structure will be demolished and a new slab-on-grade construction is planned at that location. The existing indoor swimming pool is in poor condition. The existing pool area will be backfilled with soils. It is also understood that as a part of the building renovations new columns will be erected within the existing swimming pool area to support the roof structure. Also, a new floor slab will be constructed on the backfilled area.

Detailed information regarding the foundation systems of the existing buildings are not known to us. Based on the structural loading information provided by Mr. Fred Dally, P.E. of ASA Dally, the maximum dead load and live loads on the stairwell structure are anticipated to be on the order of 40 kips and 60 kips, respectively.

We understand that the finish grade elevation of the proposed structures as well as addition will match the finish grade elevation of the existing building. Minor grading operations are anticipated within the development area.

The geotechnical recommendations presented in this report are based on the available project information, site location, field and laboratory testing, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to explore the subsurface conditions at the site to enable an evaluation of the foundation systems for the proposed development. The scope of services included drilling a total of six (6) soil test borings, laboratory testing, and preparation of this geotechnical report. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations regarding the following:

- Site preparation;
- Foundation types, depths, allowable bearing capacities, and an estimate of probable settlement;
- Comments regarding factors that may impact construction and performance of the proposed construction.

The scope of PSI’s services did not include an environmental assessment for hazardous or toxic materials in the soil, surface water, ground water, or air on or below, or around this site. Statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

SITE AND SUBSURFACE CONDITIONS

SITE LOCATION AND DESCRIPTION

The HCC Central College Campus is located in midtown area at 1300 Holman Street in Houston, Texas. The HCC school site is historic and has two site survey entries as well as an historic marker data file on the Texas Historical Commission website.

The main college building has east and west additions and is located within the central portion of the campus. Two new buildings are present within the northeast and northwest portions of the campus. The site is bound by Holman Street to the north, San Jacinto Street to the west, Alabama Street to the south, and Austin Street to the east.

SUBSURFACE CONDITIONS

As requested, the subsurface conditions at this site were explored by drilling a total of 6 soil borings. Borings B-2 through B-6 were drilled outside the existing structures whereas boring B-1 was drilled inside the existing building area. Boring B-1 was drilled indoors to a depth of 15 feet below the existing floor slab and borings B-2 through B-6 were drilled to depths of 25 to 40 feet below the existing ground surface. Plate 1 in the Appendix shows the approximate boring locations in plan.

The boring locations were located in the field by a PSI representative by measuring off from existing landmarks based on the information provided by the client. The exterior borings were drilled using a truck mounted drill rig and continuous flight auger and wet rotary drilling methods. Soil samples were routinely obtained during the drilling process. Typically, continuous samples were obtained to a depth of about 10 feet below the existing ground surface and at 5-foot interval to the boring depth. Indoor soil boring was drilled using Geoprobe equipment using ‘direct push’ techniques to sample the subsurface soils.

The floor slab at B-1 location was cored using 4 inch diameter core bit. Based on the core thickness measured, the cement concrete floor slab is about 8 inches thick at the cored location. The compressive strength of the concrete is estimated to be 3,300 psi. About 2 inches of void space was observed below the concrete slab. Below this, fill consisting of clay with sand and gravel was encountered and extended to a depth of 6 feet. The fill was followed by fat clay extended to a depth of about 15 feet below the floor slab.

Borings B-2 through B-6 were performed in the parking lot and alley areas. After the completion of the coring and drilling, the floor slab area was patched with non-shrink grout. The boreholes were backfilled with soils cuttings and patched with non-shrink grout after the drilling operations.

Cohesive soil samples were obtained by hydraulically pushing a thin-walled Shelby tube in general accordance with ASTM D 1587. The samples obtained from the borings were identified by boring number and depth. The samples were transported to PSI's laboratory for further observation, classification, and testing. For cohesive soil samples, estimates of the shear strengths of the soils were obtained by using a pocket penetrometer on soil samples retrieved in the field.

The soil samples obtained during the field exploration were transported to the laboratory and selected soil samples were tested in the laboratory to determine the material properties for evaluation. Laboratory testing on selected samples included Moisture Content (ASTM D 2216), Atterberg Limits (ASTM D 4318), Percent Passing No. 200 Sieve (ASTM D 1140), Unconfined Compression Strength tests (ASTM D 2166), and Unconsolidated Undrained Triaxial Tests (ASTM D 2850).

The soil samples obtained from the drilling operation were classified in general accordance with ASTM D 2487 or D 2488. Laboratory test data along with detailed descriptions of the soils can be found on the logs of the borings. Plates 2 through 7 located in the Appendix show the logs of borings. A key to terms and symbols used on the logs is presented on Plate 8 located in the Appendix. A boring profile is presented on Plate 9 located in Appendix.

Based on the borings performed, the subsurface conditions at the site and summary of the test results are tabulated below. Detailed descriptions of the soils encountered are presented on the attached boring logs. Table 1A and Table 1B summarize the soil profiles and the properties of the soil.

Table 1A: Generalized Soil Profile

Stratum	Depth Range (feet)	Description
I	0 to 8	Fill: Dark Brown and Gray Sandy Clay/Clay with Sand and Gravel
II	4 to 40	Fat Clay (CH), Firm to Very Stiff, Tan & Gray

Note: A stratum of lean clay was encountered at a depth of 4 to 8 feet in boring location B-3, and at boring location B-4, intermittent layers of fat clay and sandy lean clay were encountered.

Table 1B: Summary of Soil Properties

Stratum Description	Moisture Content (%)	Atterberg Limits	Minus #200 (%)	Shear Strength (tsf)
I: FILL	10 to 31	LL = 31 to 62 PI = 18 to 39	62 to 75	0.5 to 4.3
II: CH	15 to 32	LL = 55 to 79 PI = 40 to 65	71 to 89	0.25 to 1.5

Note: A stratum of lean clay was encountered at a depth of 4 to 8 feet in boring location B-3. At boring location B-4, intermittent layers of fat clay and sandy lean clay were encountered and extended to the boring termination depth of 25 feet.

The aforementioned subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the Appendix should be reviewed for specific information at the boring locations. These records include soil descriptions, stratifications, locations of the samples, and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected across the site. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on the boring logs. The samples, which were not altered by laboratory testing, will be retained for 60 days from the date of this report and then will be discarded without any further notice.

EXISTING FILL

Based on the information obtained from the borings, fill consisting of clay with sand and gravel was encountered across the project site and extended to depths of 4 to 8 feet below the existing grades. At boring location B-2, the fill consists of brick pieces. The fill appears to be of varied in material type and varied in consistency.

GROUNDWATER CONDITION

Groundwater was encountered at depths of about 13 to 23 feet below the existing ground surface during the drilling operations and at depths of about 13 to 14 feet below the existing ground surface after completion of drilling operations.

It is possible that seasonal variations (temperature, rainfall, etc) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered in discontinuous zones within the overburden. The groundwater levels presented in this report are the levels that were measured at the time of our field activities. We recommend that the contractor determine the actual groundwater levels at the site at the time of the construction activities to determine the impact, if any, on the construction procedures.

EVALUATION AND RECOMMENDATIONS

POTENTIAL VERTICAL RISE (PVR)

The results of laboratory plasticity tests indicate that the soils at this site have moderate to high potential to shrink or swell. The soils have a tendency to swell when soil moisture increases and shrink when the soil moisture decreases. The amount of potential movement to shrink and swell with soil moisture variations is represented or indicated by Potential Vertical Rise (PVR). The estimates of PVR were computed using two different methods and are shown below. In designing the floor slab or foundation system, the structural engineer should take movements associated with shrinking-swelling soils into account.

PVR estimates are based on an assumed depth known as “Active Depth” to which the soil moisture variations could occur due to seasonal variations. The typical active depth for this area is assumed to be at 7 feet. It is noted that the active depth assumed herein may not represent the moisture variations that can occur to deeper depths due to the presence of large tree root systems that could desiccate the soils, or the presence of other heating units, or possible soil wetting due to pipe leaks, poor drainage, etc. It is very difficult to predict the lateral and vertical moisture variations under the structure during its service life. Even if the moisture variations were to be predicted, the current state of soil mechanics cannot predict the soil movements associated with shrinking and swelling accurately. This is largely due to the inability of laboratory tests, including swell tests, to accurately replicate the field conditions in their present state or during the entire service life of the structure. Hence, the PVR estimates provided herein should be considered approximate probable estimates based on industry standard practice and experience, and the movements predicted herein should not be construed as absolute values that could occur in the field.

A PVR value of about 1 inch was calculated for this site using the chart presented in the American Association of State Highway Transportation Officials (AASHTO) Guide for Designing Pavement Structure, 1986, Vol. 1; Figure G.3, Appendix G: Chart for Estimate Approximate Potential Vertical Rise for Natural Soils. This method assumes a linear variation of percent swell within the active depth, such that percent swell is a maximum at the ground surface and zero at the bottom of the active depth. This method may be considered appropriate for normal soil moisture variations due to average rainfall variations in this area.

A PVR value of about 2 inches was calculated for this site using the Texas Department of Transportation (TxDOT) TEX-124-E method. For this site, PVR estimates were calculated using the “Dry Swell Line” shown on Figure 1 of TEX-124-E method. This method uses the uniform percent swell through the entire active depth. This method may be considered appropriate for extreme soil moisture variations such as extreme rainfall variations in this area.

The PVR or the movements associated with soil shrink-swell potential will cause movements to the structure (floor-slab) when a structure is placed directly in contact with the soil. In order to mitigate the movements, the structure (floor-slab) should be isolated from contact with the soil and should be supported above grade, structurally on appropriate deep foundation system with adequate void space between the structure (floor-slab) and the soil.

Alternatively, it has been the industry practice to place a non-expansive (i.e., low plasticity structural fill) soil layer between the natural soil and the structure (floor slab) to reduce the movements associated with shrinking and swelling soils. This method to reduce the movements is dependent on the assumption that a certain amount of movement can be tolerated and it is noted that the success of method is primarily dependent on: 1) the thickness of the non-expansive structural fill material placed below the structure (floor slab), and 2) the methods (i.e., civil drainage, landscape, other designs) adopted to prevent moisture variations below the structure (floor slab). The owner's should understand the assumptions of this method and the associated risk that movements could occur when this method is adopted.

For this site, in order to reduce the PVR to about one inch, it is recommended that at least 3 feet of low plasticity structural fill should be placed between the soils and the structure (floor-slab). The structural fill should be placed within the plan area of the structure and to a distance of at least five feet beyond the perimeter of the structure. Plasticity requirements for the structural fill are provided in the Site Preparation section of this report.

Fill were encountered at the site and extended to depths on the order of 4 to 8 feet below the existing grades. Removal of the existing fill (within the building pad) in its entirety and replacement with structural fill will reduce the PVR to about ¾ inch.

It is not uncommon to assume the differential movement to be about half the value of the PVR. This is based on the assumption that a certain amount of moisture variation may occur beneath the plan area of the floor slab. It is possible that under extreme moisture variation conditions, the differential movements could be equal to, or even double, the value of PVR.

Poor drainage and water infiltration into the foundation soils for an extended period of time can be detrimental to the floor slab and foundation. Excessive wetting of soil (due to accumulation of water), or, excessive drying (due to the presence large trees, etc) could possibly result in greater PVR values than those estimated herein as the moisture variations could occur down to deeper depths; or, the moisture variations or shrinking and swelling predictions can be greater than those inherently assumed by the methods mentioned above. It is recommended that the moisture-related problems be corrected immediately as they can be detrimental to the foundation and floor slab.

Swelling or shrinkage occurs in soils due to changes in moisture content. Water ponding around the foundations/slab may result in reduction of soil strength, thereby causing adverse and damaging movements. Poor drainage and water seepage for an extended period of time can be detrimental to the slab and foundation. It is important to control the possibility of moisture changes by following the precautions shown below:

1. Direct surface runoff away from structures by sloping the subgrade away from the slabs.
2. Extend paving or other impervious coverings, such as sidewalks, to the slab edge.
3. Extend roof drain downspouts so that the discharge is at least 5 feet from the slab.
4. Avoid placing trees or shrubs adjacent to slab.
5. Avoid excessive drying of soil around the slab.

SITE PREPARATION

We recommend that, within the area of construction, existing fill materials with roots and organic material, existing pavement, concrete curbs and other miscellaneous debris be removed from the site and wasted. Any underground utilities be located and rerouted as necessary.

Based on the borings, old existing fill materials were encountered to depths of 4 to 8 feet below the existing grades. The existing fill encountered at the site may contain concentrated amount of deleterious materials and soft compressible zones not revealed by the borings at the explored locations. It is recommended that the existing fill be removed in its entirety or undercut to the natural subgrade. A PSI representative should determine the actual depth of removal at the time of construction.

After stripping and excavating to the natural subgrade, the exposed soil should be proof-rolled to locate any soft or loose areas. Proof rolling can be performed in accordance with Item 216 of TxDOT Specification. Soils that are observed to rut or deflect under the moving load should be undercut and replaced with properly compacted structural fill. The proof-rolling and undercutting activities should be witnessed by a PSI representative and should be performed during a period of dry weather.

After proof-rolling has been completed, any necessary fill placement may begin. The first layer of fill should be placed in a relatively uniform horizontal lift and be adequately keyed into the subgrade soils. Structural fill materials should be sandy clay soils free of organic or other deleterious materials, have a maximum clay lump size of less than three inches, and have a liquid limit not greater than 35 and a plasticity index between 8 and 20. Structural fill should be compacted to at least 95 percent of standard Proctor maximum dry density as determined by ASTM D 698.

Structural fill should be placed in maximum lifts of eight inches of loose material and should be compacted within the range of zero to three percentage (0% to +3%) points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of structural fill should be tested by a representative of the geotechnical engineer prior to placement of subsequent lifts. Care should be taken to apply compactive effort throughout the fill and fill scope areas. The moisture content and the degree of compaction of the structural fill soils should be maintained until the construction of the structure within the area.

GEOTECHNICAL DISCUSSION

Based on the borings, fill has encountered to depths of 4 to 8 feet below the existing grades. Structures supported on shallow foundations bearing on fill materials could undergo excessive and uneven settlement. The proposed structures may be supported on spread footings provided the existing fill materials are removed in their entirety and replaced with structural fill. Alternatively, the proposed structures may be supported on drilled and underreamed piers extended to a minimum depth of 10 feet below the existing grade.

A new slab-on-grade construction is planned at the existing Upholstery structure. Existing foundation system and buried utility line locations are not known to us. Existing foundation elements should be located prior to start of the new construction.

It is also understood that the existing swimming pool is in poor condition. The pool area will be backfilled with compacted soils and new column foundation will be designed in the existing pool area. Footings for the new columns may be constructed below the bottom of the pool slab. Also, a new floor slab will be constructed on the compacted backfill materials. New floor slab area should be prepared as recommended in this report. The new floor slab will be connected to the existing floor slab. This may result in differential settlement of the floor slabs due to the varying thickness of the compacted materials beneath the floor slabs.

Considering the presence of the existing footings, the constructability of the new column foundations should be assessed as the (new foundation) excavations could undermine the integrity of the old foundations.

The design recommendations provided herein are developed based on the project information and subsurface conditions identified. If there are any changes in these project criteria, including structure locations on the site, a review must be made by PSI to determine if any modifications in the recommendations will be required.

IMPACT OF EXISTING STRUCTURES ON PROPOSED CONSTRUCTION

It is understood that the existing upholstery structure will be demolished to accommodate the new development. At this time, the foundation system of the existing building is unknown. It is noted demolition activities of the old foundation system may impact the design and construction of the new foundation. It is recommended that the existing foundations be partly removed or removed in their entirety as discussed below.

- If the foundation system of the existing structure is “Shallow” foundations, it is recommended that the existing “shallow” foundations including spread footings grade beams, and strip footings that are at a depth of 5 feet or shallower be removed entirely.

- If the foundation system of the existing structures is “Drilled and Underream Piers” or “Drilled Shafts”, it is recommended that the shaft portions of the existing pier foundations should be saw-cut down to a depth of at least 4 feet below the shallow foundation system of the new facility. After the demolition of existing super structures, the existing piers should be surveyed to identify their exact locations. The existing foundations should be located on the new foundation layout drawing to assess the impact on the design and construction of the new foundation system. PSI should be given an opportunity to assess the impact of the new foundation system and amend or alter the foundation recommendations provided herein. At least 2-feet of structural fill should be maintained between the top of the old pier foundations and the new shallow foundation system.

It is also recommended that any underground utilities and other below grade components should be removed or grouted in-place. Any voids formed due to the removal of the existing structures should be backfilled with compacted structural fill. Construction debris, loose soils should be removed, undercut from the construction area.

DRILLED AND UNDERREAMED PIERS FOUNDATIONS

The proposed structures and addition can be supported on drilled and underreamed piers. The piers should be placed at a depth of at least 10 feet below the existing ground surface bearing on the stiff clays. Piers should not extend below a depth of 13 feet due to presence of silt seams and that the pier excavation may experience ground water infiltration. Individual piers bearing in the stiff clays can be designed for a maximum allowable net bearing pressure of 6,000 psf for total dead plus live loads and 4,000 psf for dead plus sustained live loads, whichever results in a larger bearing area.

Piers extending through expansive soils are potentially subjected to vertical uplift loads, should the soils become moist or wet and swell. For this reason, each pier should be designed with sufficient steel reinforcement to resist the tensile stresses caused by the uplift forces of the expansive soil. Piers placed within natural swelling soils at this site should be checked for reinforcement with a tension load of 17d kips; where d is the diameter of the pier in feet. The reinforcement of the pier should be checked for this tension load alone neglecting any dead loads on the pier.

A single isolated pier with a bell diameter of about eight feet or less and designed as discussed should experience a settlement on the order of one-half inch or less. However, if a cluster of closely spaced piers is planned, PSI should be contacted to calculate the amount of settlement.

Wall loads (building addition) should be transmitted to the drilled and underreamed piers by grade beams and the grade beam should be structurally connected to the piers. Typically, void boxes are provided under the grade beams to avoid movements associated with shrinking and swelling soils. Presence of void boxes has advantageous and disadvantages. Based on experience, in some cases, it was seen that presence of void boxes created conduits for water resulting in moisture increase and swelling. If void boxes are not provided, the grade beams may experience uplift pressures or movements due to swelling soil or existing fill and the grade beams should be designed to account for the swelling movements. With the void boxes in place, the grade beam movements due to shrinking and swelling soils could be negligible.

For the construction of the underream or bell, a bell diameter to shaft diameter ratio of 2 to 1 is recommended. We believe that a bell to shaft diameter ratio of 3 to 1 can be achieved at this site, if the bell angle to the horizontal is 60°.

The uplift capacity of drilled and underreamed piers can be determined from the following semi-empirical relationship:

$$Qu = Nu * Su * \pi * (D^2 - d^2)/4$$

Where: Qu is ultimate uplift capacity, tons

$Nu = 3.5*(H/D) \leq 9$
Su = Undrained Shear Strength, tons per square feet
D = diameter of underream or bell, feet
d = diameter of shaft, feet
H = depth to base of bell below ground surface, feet

For bells excavated within the natural clay, the value of Undrained Shear Strength, “Su” in the above equation can be taken as 0.75 tons per square foot. The computed ultimate value should be reduced by a factor of safety of 2.0 for transient and wind loads and 3.0 for sustained loads.

The lateral loads on shallow drilled and underreamed piers can be resisted by passive resistance of the soil. The allowable passive resistance of the natural soil may be taken as 1,000 psf. The allowable value includes a factor of safety of 2.0. Determination of the lateral load carrying capacity using the passive earth pressure does not predict the lateral pier-head load versus pier-head deflection behavior of the drilled pier. It is recommended that the passive resistance from the upper two feet of soil be neglected. Also, the passive resistance from any uncompacted fill material should be neglected.

The successful completion of drilled-and-underreamed excavations will depend, to a large extent, on the suitability of the drilling and underreaming equipment together with the skill of the operator. The sequence of operations should be scheduled so that each underream can be completed, reinforcing steel placed and the concrete poured in a continuous, rapid and orderly manner to reduce the time that the excavation is open.

Underream excavations and the bearing area should be clear and be free of loose materials prior to placement of concrete. Placement of concrete in the excavations should commence immediately after the underream excavation is completed. A PSI representative should verify that the underream installation procedures meet specifications. Installation of the piers can be carried out in general accordance with the guidelines provided in the Drilled Shaft Manual, Publication No. FHWA-IS-99-025.

SHALLOW FOUNDATIONS

Provided the site preparation recommendations are followed, the planned construction can be supported on a shallow foundation system bearing on properly compacted structural fill soils. It should be noted that the existing fill should be removed in its entirety and replaced with the compacted structural fill as recommended in the report.

A shallow foundation system supported on compacted structural fill may be designed for a net allowable bearing capacity of 3,000 psf for dead load plus live loads, and 2,000 psf for dead plus sustained live loads, whichever results in a larger bearing area.

Minimum dimensions of 24 inches for footings and 18 inches for continuous footings should be used in the design. Single isolated footing with a width no larger than eight feet, or grade beams designed as discussed above, should experience a settlement of less than one inch. If a cluster of closely spaced footings (i.e., if the center to center spacing of the footings is less than two times the width of the footing) are planned, PSI should be contacted to calculate the amount of settlement.

The base adhesion/frictional resistance and the passive soil resistance will resist the horizontal loads on shallow foundations. For a footing cast against (natural clay soil or) compacted fill, the adhesion/frictional resistance and the passive soil resistance values for both transient and sustained loading conditions are given herein. For transient loading conditions, an ultimate base adhesion resistance of 550 psf and an ultimate passive resistance of 2,000 psf can be used. For sustained loading conditions, a frictional co-efficient of 0.36 and an ultimate passive resistance of 240 psf per foot depth is recommended. A factor of safety of 2.0 is recommended to arrive at the allowable values. Passive resistance from the upper two feet of soil should be neglected. Also, the passive resistance of any un-compacted fill material should be neglected.

The uplift resistance of a shallow foundation formed in an open excavation will be limited to the weight of the foundation concrete and the soil above it. For design purposes, the ultimate uplift resistance should be based on effective unit weights of 120 and 150 pcf for soil and concrete, respectively. This value should then be reduced by an appropriate factor of safety to arrive at the allowable uplift load. If there is a chance of submergence, the buoyant unit weights should be used.

The foundation excavations should be observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials are suitable for supporting the design loads and are consistent with the materials discussed in this report. Soft or loose soil zones encountered at the bottom of the footing excavations should be removed and replaced with properly compacted fill as directed by the geotechnical engineer.

After opening, footing excavations should be observed and concrete placed as quickly as possible to avoid exposure of the footing bottoms to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. The foundation concrete should be placed during the same day the excavation is made. If it is required that footing excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

FLOOR SLAB RECOMMENDATIONS

A slab-on-grade floor slab can be constructed provided the shrink/swell potential of the soil is taken into account and the site is prepared in accordance with the recommendations mentioned herein. As previously mentioned, the soils at this site have moderate to high potential to shrink and swell with changes in soil moisture content. A PVR value of about 1 inch was computed using the AASHTO method. A PVR value as high as 2 inches was calculated using TEX-124-E method. A more detailed discussion for the potential of shrinking/swelling soil movements is presented earlier in this report and the structural engineer should take this into account for the design.

For this site, in order to reduce the PVR to about 1 inch, it is recommended at least 3 feet of low plasticity structural fill should be placed between the natural soils and the structure (floor-slab). The structural fill should be placed within the plan area of the structure and to a distance of at least five feet beyond the perimeter of the structure. Plasticity requirements for the structural fill are provided in the Site Preparation section of this report. It is understood that the existing fill is encountered at the site and extended to depth of 4 to 8 feet. The existing fill should be removed in its entirety and replaced with compacted structural fill. With 4 feet of fill below the floor slab, the PVR could be about ¾ inch.

An allowable net bearing pressure of 600 psf can be used for slab-on-grade bearing on compacted fill. For the recommended structural fill thickness of 3 feet, a total estimated settlement of less than 1 inch should be expected under the floor slab. However, if due to grading requirements more than the recommended fill thickness is to be placed, the settlement estimates will change. If the structural fill thickness under the floor slab is more than 6 feet, PSI should be contacted as this may change the estimated settlement values.

INDOOR SWIMMING POOL AREA

The existing swimming pool is about 40 feet long by 20 feet wide and 7 to 8 feet in depth. It is planned to backfill the pool area and new column foundations will be constructed within the plan area of the pool. It is also reported that the pool has been leaking for nearly 20 years. It is possible that the water leaking from the pool may have softened the soils below the pool bottom. As such, it is recommended that an exploratory boring be performed in the pool area to obtain the subsurface conditions below the pool slab. Recommendations for the pool back-fill and foundation recommendations for the new columns located within the pool will be provided based on the results of the boring. The recommendations provided herein should be considered preliminary. The construction sequence can be as follows:

1. After draining the water in the pool, the slab area around the pool as well as the pool walls and the pool bottom should be demolished.

2. The soils within the slab area around the pool should be excavated to a depth of 3 feet.

3. The exposed pool bottom and the exposed area around the pool should be proof rolled. Soft, wet or loose subgrade soils should be removed to the firm subgrade level and replaced with compacted structural fill

4. The foundations for the columns located within the pool can be excavated to a depth of at least 3 feet below the bottom of the pool and installed. A shallow foundation system supported on natural firm subgrade below a depth of 3 feet below the pool slab elevation may be designed for a net allowable bearing capacity of 4,500 psf for dead load plus live loads, and 3,000 psf for dead plus sustained live loads, whichever results in a larger bearing area. These recommendations should be considered preliminary and will be finalized after the subsurface information from the exploratory boring is obtained.

5. The pool area can be backfilled up to a depth of 3 feet below the new finished floor slab with flowable fill. Flowable fill should be in accordance with Item 434 of HCPID specifications or Section 2322 of HCFCD specifications. Upper tow feet of soils may consist of compacted structural fill.

6. The area of the pool and the surrounding area (i.e., below the new floor slab), should be backfilled with 3 feet of properly compacted structural fill. It is also suggested that construction joints be included at appropriate locations within the floor-slab system such that the slab is independent of the structure (columns or walls).

The impact of the renovations/excavations on the adjacent wall footings should be assessed. PSI should be given the opportunity to review the new design to amend the recommendations provided herein appropriately.

CONSTRUCTION CONSIDERATIONS

It is recommended that PSI be retained to provide observation and testing of construction activities involved in the foundations, earthwork, and related activities of this project. PSI cannot accept any responsibility for any conditions, which deviated from those, described in this report, nor for the performance of the foundations if not engaged to also provide construction observation and testing for this project.

MOISTURE SENSITIVE SOILS/WEATHER RELATED CONCERNS

Soils at the site are extremely sensitive to moisture changes, the subgrade soils should be protected and adequate drainage should be maintained at the time of the construction. During inclement weather, the subgrade soils may get disturbed due to construction traffic. It is extremely important to provide good site drainage during construction.

During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

DRAINAGE AND GROUNDWATER CONCERNS

Water should not be allowed to collect in the foundation excavation or on prepared subgrade of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the foundation. The grades should be sloped away from the foundation and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and foundation area.

For groundwater conditions, refer to the Groundwater Information section of this report. Any water accumulation should be removed from excavations by pumping. Should excessive and uncontrolled amounts of seepage occur, the geotechnical engineer should be consulted.

FEDERAL EXCAVATION SAFETY REGULATIONS

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person”, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

REPORT LIMITATIONS

The recommendations submitted are based on the available soil information obtained by PSI and design details furnished by the Client representatives for the proposed additions. If there are any revisions to the plans for the proposed structure, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be retained to determine if changes in the foundation recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the performance of the structure.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made after being prepared in accordance with generally accepted professional engineering practices in the local areas. No other warranties are implied or expressed.

After the plans and specifications are more complete, it is recommended that the geotechnical engineer be provided the opportunity to review the final design and specifications to determine if the engineering recommendations have been properly interpreted and implemented. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of ESPA Corp for the specific application to the proposed renovations to the existing structures located within the HCC San Jacinto Central College Campus in Houston, Texas.

APPENDIX

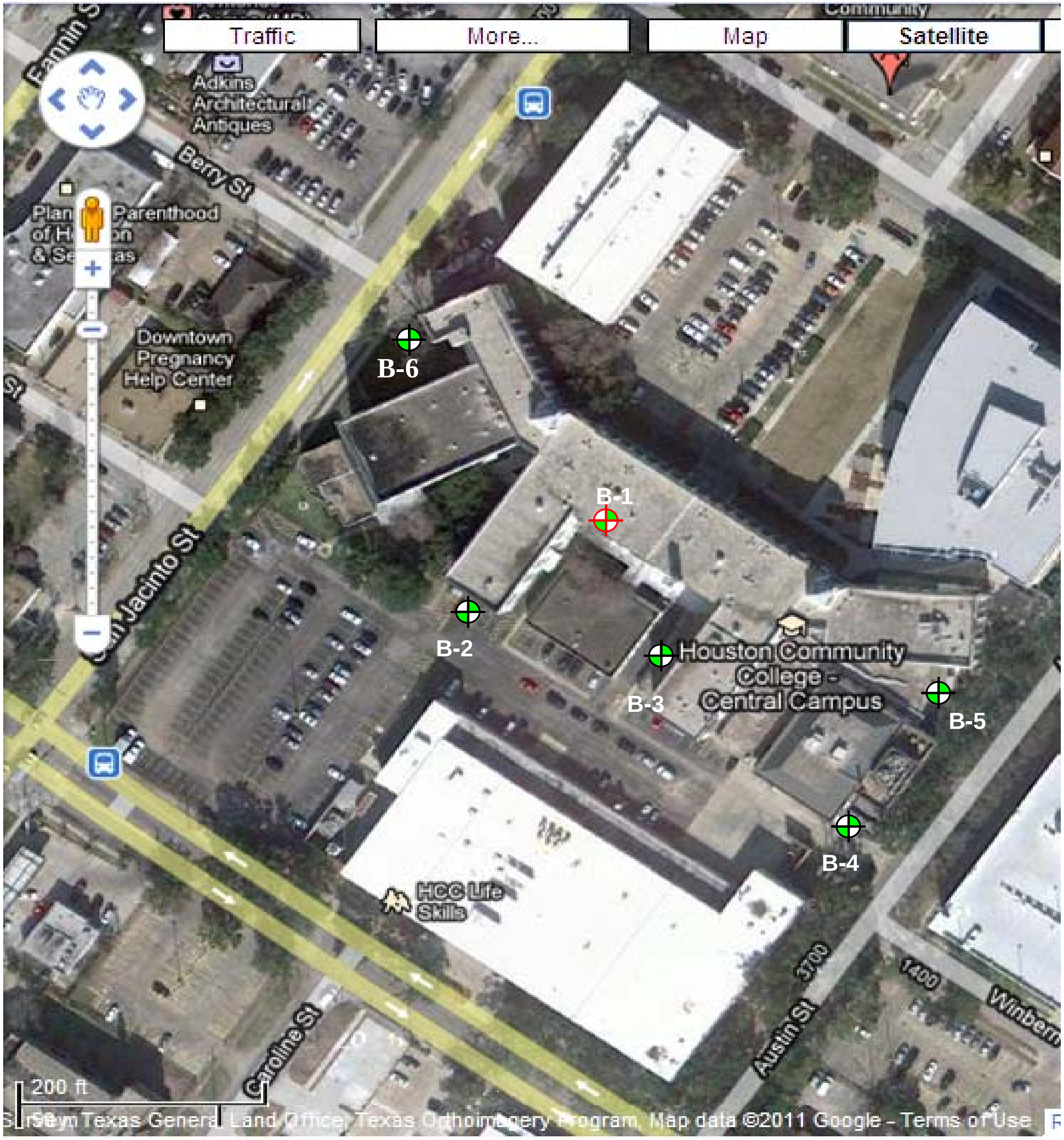
BORING LOCATION PLAN

BORING LOGS

KEY TO TERMS AND SYMBOLS USED ON THE LOGS

BORING PROFILE

BORING LOCATION PLAN



NOTES

EXTERIOR BORINGS B-2, B-3,B-4, B-5, and B-6

INDOOR BORING B-1 15 FEET DEEP

IMAGE OBTAINED FROM GOOGLE EARTH

N

HCC SAN JANCINTO BUILDING RENOVATION
NEAR HOLMAN AND SANCINTO ST
HOUSTON, TEXAS

psi
Professional Service Industries, Inc.
1714 Memorial Drive
Houston, Texas 77007

Drawn:	Scale:	Project No.:
MV	NOT TO SCALE	
Chkd:	Date:	
RK	01/28/11	

PLATE 1

LOG OF BORING B-1

HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: CONTINUOUS PUSH TUBE

PSI Project No.: 286-414

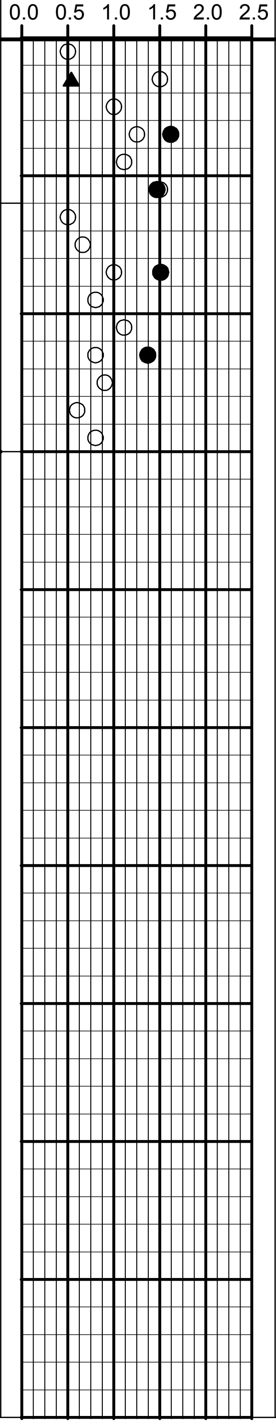
COORDINATE (X) OR EASTING: 50
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 49 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
	▨	FILL	■	8 inches Concrete, 2 inches void						18	
	▨		■	Fill: Dark Brown and Gray Clay		73	46	15	31	18	107
	▨		■	- with sand and gravel						20	
	▨		■				54	15	39	20	109
	▨		■	- brown and tan below 5 feet						19	
5	▨		■				31	13	18	15	111
	▨	CH	■	FAT CLAY (CH), FIRM TO VERY STIFF, TAN AND GRAY						20	
	▨		■							21	
	▨		■	- dark brown, 6 to 8 feet			61	17	44	19	107
	▨		■	- with ferrous nodules, 6 to 15 feet						17	
10	▨		■							16	
	▨		■				79	14	65	19	109
	▨		■							25	
	▨		■							22	
	▨		■							32	
15											
20											
25											
30											
35											
40											
45											
50											

SHEAR STRENGTH

(tons/square foot)

○ HP ● UC △ TV ▲ UU



DEPTH OF BORING: 15 FEET

DATE DRILLED: 2/25/11

INITIAL GROUND WATER: Dry
FINAL GROUND WATER: Dry

NOTES:

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LOG OF BORING B-2

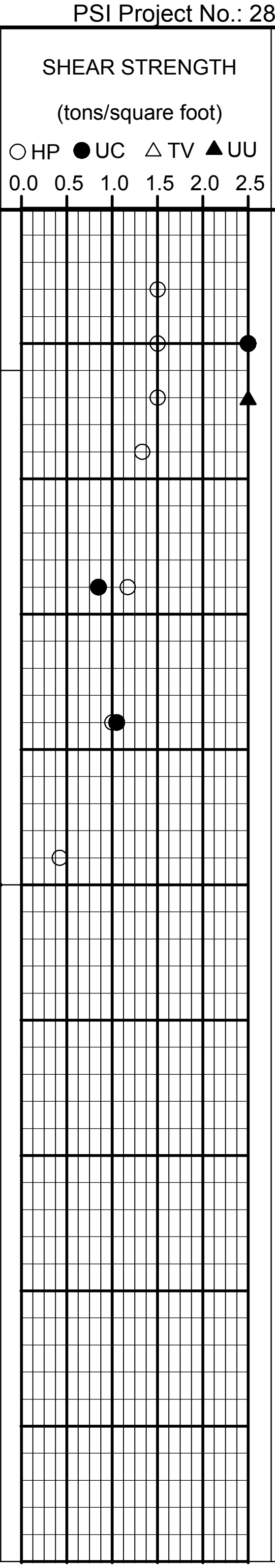
HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: SOLID FLIGHT AUGER

PSI Project No.: 286-414

COORDINATE (X) OR EASTING: 100
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 45 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
	▩	FILL		1.5 inches Asphalt, 5 inches gravel base							
	▩		■	Fill: Dark Brown and Gray Clay		73	48	15	33	13	
	▩		■	- with sand and gravel							
	▩		■	- brick pieces, 0 to 6 feet						15	
5	▩		■				47	14	33	24	(2.95) 115
	▨	CH	■	FAT CLAY (CH), STIFF TO VERY STIFF, TAN AND GRAY						17	(2.52) 114
	▨		■	- with ferrous nodules, 6 to 10 feet							
	▨		■	- with calcareous materials, 8 to 15 feet						16	
10	▨										
	▨		■				73	24	49	28	106
15	▨										
	▨		■							19	112
20	▨										
	▨	▼	■	- sandy clay, 23 to 25 feet						19	
	▨		■	- firm, 23 to 25 feet							
25											
30											
35											
40											
45											
50											



DEPTH OF BORING: 25 FEET
DATE DRILLED: 2/16/11

INITIAL GROUND WATER: 23 feet during drilling
FINAL GROUND WATER: 23 feet after drilling

NOTES:

LOG OF BORING B-3

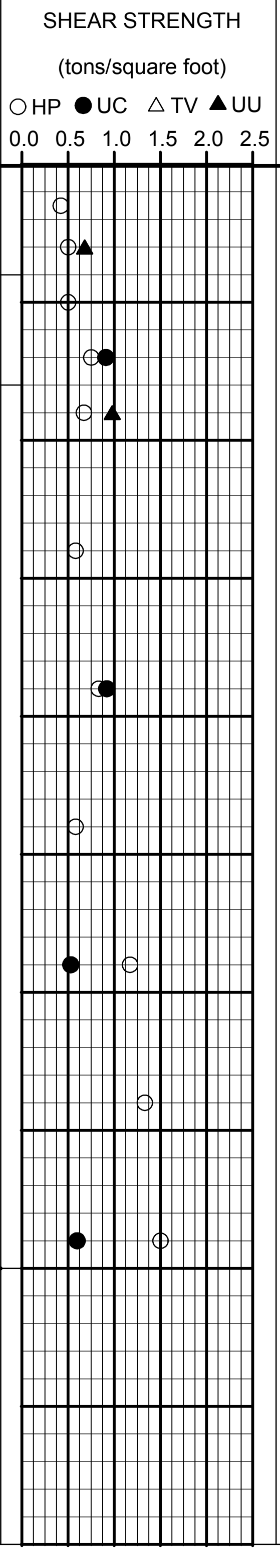
HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: SOLID FLIGHT AUGER 1' to 25' and WET ROTARY THEREAFTER

PSI Project No.: 286-414

COORDINATE (X) OR EASTING: 150
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 45 feet
LATITUDE:
LONGITUDE:

USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
FILL	2.5 inches Asphalt, 6 inches gravel base Fill: Dark Brown and Gray Clay with Sand						19	
			84	62	15	47	31	96
CL	LEAN CLAY (CL), STIFF, TAN AND GRAY - ferrous nodules, 4 to 8 feet						21	
				43	14	29	18	105
CH	FAT CLAY (CH), STIFF TO VERY STIFF, TAN AND GRAY - ferrous nodules, 8 to 23 feet						24	107
▼							34	
			89	63	19	44	24	100
▽	- tan and dark brown clay with sand, 23 to 25 feet						16	
	- trace of lignite, 25 to 26 feet	7	71				18	
	- reddish brown below 28 feet						17	112
							18	
	- calcareous materials, 38 to 40 feet						15	109



DEPTH OF BORING: 40 FEET
DATE DRILLED: 2/16/11

INITIAL GROUND WATER: 23 feet during drilling
FINAL GROUND WATER: 14 feet after 1 hour

NOTES:

LOG OF BORING B-4

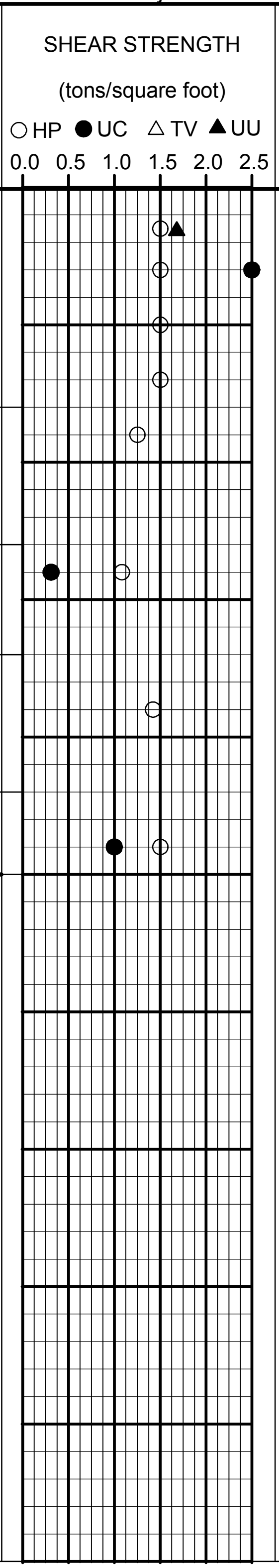
HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: SOLID FLIGHT AUGER

PSI Project No.: 286-414

COORDINATE (X) OR EASTING: 200
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 45 feet
LATITUDE:
LONGITUDE:

USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
FILL	6 inches Concrete, 6 inches gravel base Fill: Dark Brown and Gray Sandy Clay to Clay with Sand		62	36	14	22	11	118
							13	116
			72	49	15	34	12	
							16	
CH	FAT CLAY (CH), VERY STIFF, TAN AND GRAY - ferrous nodules, 8 to 10 feet						22	
CL	SANDY LEAN CLAY (CL), FIRM, REDDISH BROWN -with interbedded silt seams and silt layers			27	18	9	21	107
CH	FAT CLAY (CH), VERY STIFF, REDDISH BROWN						31	
CL	SANDY LEAN CLAY (CL), STIFF, REDDISH BROWN AND GRAY						18	108



DEPTH OF BORING: 25 FEET
DATE DRILLED: 2/22/11

INITIAL GROUND WATER: 13.5 feet during drilling
FINAL GROUND WATER: 13 feet upon completion

NOTES:

LOG OF BORING B-5

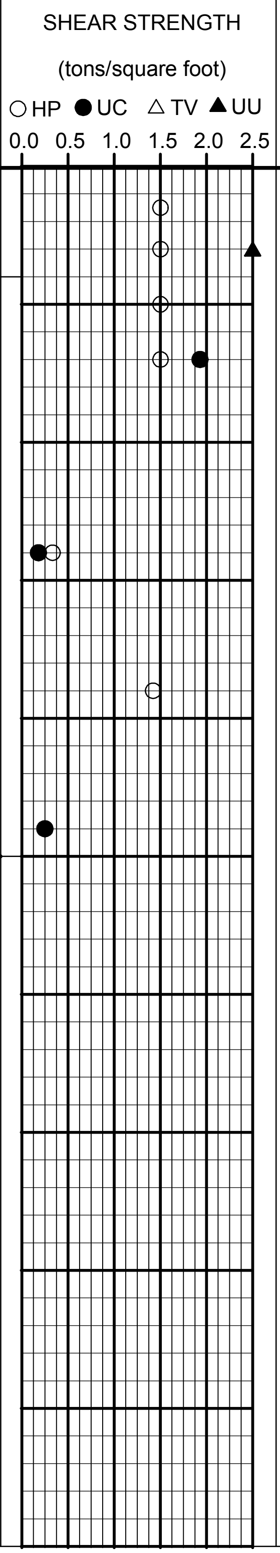
HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: SOLID FLIGHT AUGER

PSI Project No.: 286-414

COORDINATE (X) OR EASTING: 250
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 45 feet
LATITUDE:
LONGITUDE:

SOIL TYPE / USCS SYMBOL	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
FILL	6 inches of Topsoil						10	
	Fill: Dark Brown and Gray Clay with Sand						14	
	- with sand and gravel		75	56	17	39	19	(4.34) 114
CH	FAT CLAY (CH), FIRM TO VERY STIFF, TAN AND GRAY						27	
	- with ferrous nodules, 4 to 10 feet			57	16	41	18	117
							18	
	- soft, 13 to 15 feet - with silt and sand seams, 13 to 15 feet						25	106
	-becomes reddish brown below a depth of 18 feet			65	23	42	24	
	- calcareous material, 23 to 25 feet						17	121



DEPTH OF BORING: 25 FEET
DATE DRILLED: 2/22/11

INITIAL GROUND WATER: 13 feet during drilling
FINAL GROUND WATER: 13 feet upon completion

NOTES:

LOG OF BORING B-6

HCC SAN JACINTO CENTRAL
1300 HOLMAN STREET, HOUSTON, TEXAS

TYPE OF BORING: HAND AUGER

PSI Project No.: 286-414

COORDINATE (X) OR EASTING: 300
COORDINATE (Y) OR NORTHING: 0
APPROXIMATE SURFACE ELEVATION: 46 feet
LATITUDE:
LONGITUDE:

DEPTH, FT.	SOIL TYPE	USCS SYMBOL	SAMPLES	SOIL DESCRIPTION	N-BLOWS/FT.	% PASSING No. 200 SIEVE	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)	SHEAR STRENGTH (tons/square foot) ○ HP ● UC △ TV ▲ UU 0.0 0.5 1.0 1.5 2.0 2.5	DRY UNIT WEIGHT (pcf)
		FILL		6 inches of Topsoil								
				Fill: Dark Brown and Gray Clay with Sand		72	43	15	28	20		
				- with sand and gravel								
				- with organics						23		
		CH		FAT CLAY (CH), TAN AND GRAY								
5				- ferrous nodules, 4 to 7 feet			55	15	40	22		
				- with calcareous material, 6 to 7 feet						21		
10												
15												
20												
25												
30												
35												
40												
45												
50												

DEPTH OF BORING: 7 FEET

DATE DRILLED: 2/22/11

INITIAL GROUND WATER: Dry
FINAL GROUND WATER: Dry

NOTES:

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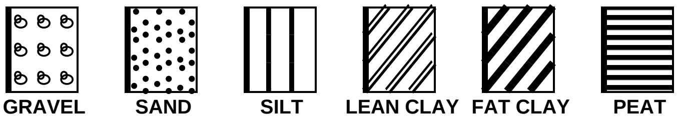
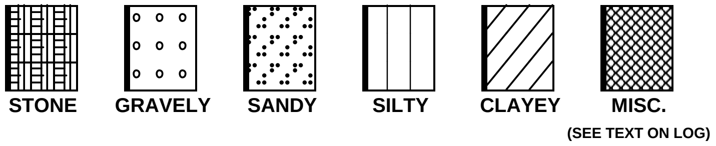
PLATE NO: 7 (Page 1 of 1)

KEY TO TERMS AND SYMBOLS USED ON LOGS

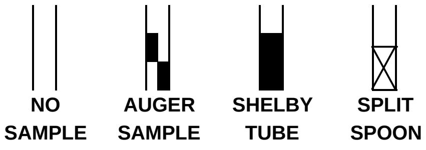
SOIL TYPE

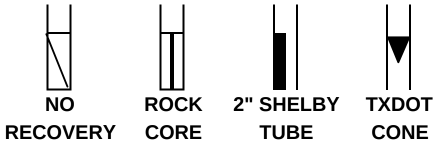


MODIFIERS

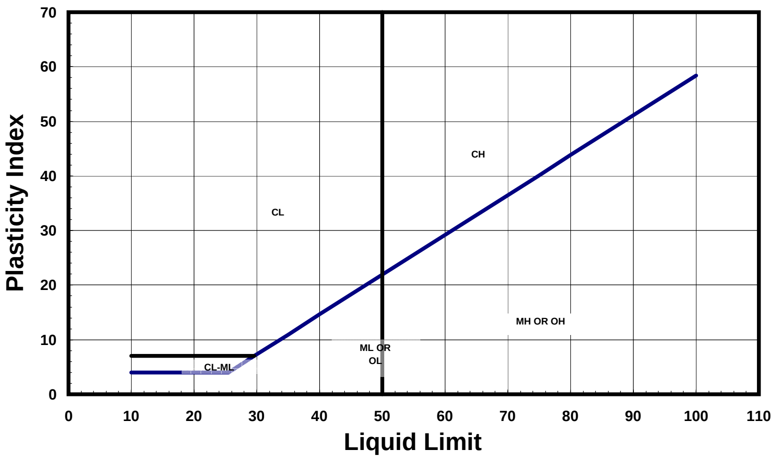


SAMPLER TYPE





UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D 2487



CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	SHEAR STRENGTH IN TONS/FT^2
VERY SOFT	0 TO 0.125
SOFT	0.125 TO 0.25
FIRM	0.25 TO 0.5
STIFF	0.5 TO 1.0
VERY STIFF	1.0 TO 2.0
HARD	> 2.0 OR 2.0+

RELATIVE DENSITY - GRANULAR SOILS

CONSISTENCY	N-VALUE (BLOWS/FOOT)
VERY LOOSE	0 TO 4
LOOSE	5 TO 9
MEDIUM DENSE	10 TO 29
DENSE	30 TO 50
VERY DENSE	> 50 OR 50+

DEGREE OF PLASTICITY OF COHESIVE SOILS

DEGREE OF PLASTICITY	PLASTICITY INDEX	SWELL POTENTIAL
NONE OR SLIGHT	0 TO 4	NONE
LOW	4 TO 20	LOW
MEDIUM	20 TO 30	MEDIUM
HIGH	30 TO 40	HIGH
VERY HIGH	> 40	VERY HIGH

MOISTURE CONDITION COHESIVE SOILS

DESCRIPTION	CONDITION
Absence of moisture, dusty, dry to touch	DRY
Damp but no visible water	MOIST
Visible free water	WET

CONSISTENCY OF COHESIVE SOILS AFTER TERZAGHI (1948)

CONSISTENCY	N-VALUE (BLOWS/FOOT)
VERY SOFT	< 2
SOFT	2 TO 4
FIRM	4 TO 8
STIFF	8 TO 15
VERY STIFF	15 TO 30
HARD	> 30

ABBREVIATIONS

HP - HAND PENETROMETER
TV - TORVANE
MV - MINIATURE VANE

UC - UNCONFINED COMPRESSION TEST
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
CU - CONSOLIDATED UNDRAINED

NOTE: PLOT INDICATES SHEAR STRENGTH AS OBTAINED BY ABOVE TESTS

▼ FINAL GROUND WATER LEVEL

▽ INITIAL GROUND WATER LEVEL

CLASSIFICATION OF GRANULAR SOILS

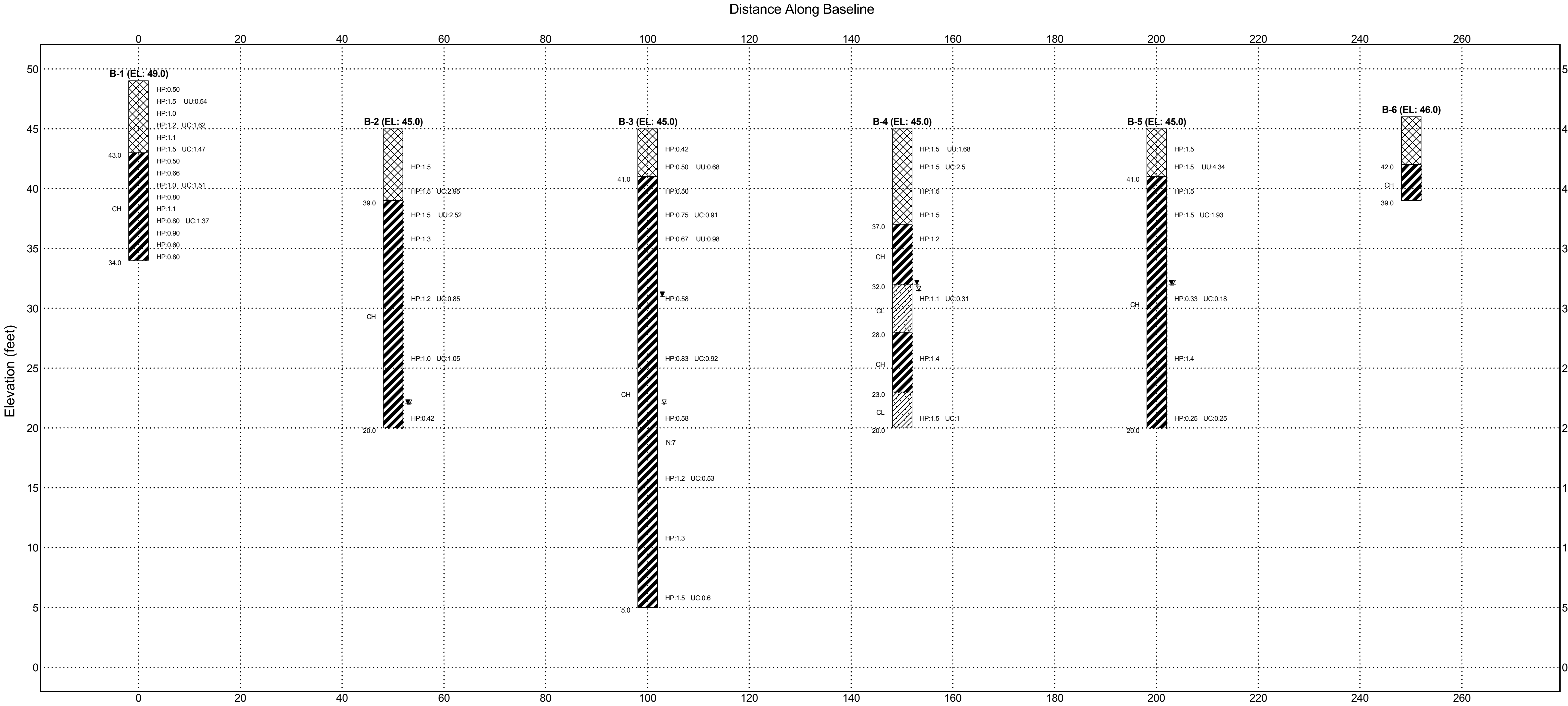
U.S. STANDARD SIEVE SIZE(S)

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.0 0.42 0.074 0.002

GRAIN SIZE IN MM



Stratifications shown are generalized and variations could occur in the field. The HP, UC, TV, UU values are shear strengths in tsf.

▨ Fill Material

▨ Fat Clay (CH)

▨ Sandy Lean Clay (CL)

STRATUM ID	START	END	STRATA DESCRIPTION
I	0	8	FILL: Dark Brown and Gray Sandy Clay/Clay with Sand and Gravel
II	4	40	Fat Clay (CH), Firm to Very Stiff, Tan and Gray

GENERALIZED SUBSURFACE PROFILE

HCC SAN JACINTO CENTRAL

1300 HOLMAN STREET, HOUSTON, TEXAS

PROJECT No.	DATE	PLATE
286-414	Mar 2011	Plate 9