GEOTECHNICAL INVESTIGATION
FY2013 STORM WATER STATION
AND FLOOD WARNING SYSTEM
IMPROVEMENTS
WBS NO. M-430241-0009-3
HOUSTON, TEXAS

REPORT NO. 1140203801

Reported to:

CHESTER ENGINEERS, INC.

Houston, Texas

Submitted by:

GEOTEST ENGINEERING, INC.

Houston, Texas

October 7, 2014

Key Map No. 455 U

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Report No. 1140203801
October 7, 2014

Mr. Adarsh Shah, P.E.
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Reference: Geotechnical Investigation
FY2013 Storm Water Pump Station and Flood Warning System Improvements
WBS No. M-430241-0009-3
Houston, Texas

Dear Mr. Shah:

Presented herein is our final geotechnical investigation report for the referenced project. A draft report was submitted to you on August 13, 2014. This final report supersedes all previously submitted reports, transmittals, etc. for the referenced project. This study was authorized by Work Order No. P2-001 dated July 7, 2014 and by accepting Geotest Engineering, Inc. (Geotest) proposal No. 1140347299 dated April 17, 2014.

We appreciate this opportunity to be of service to you. If you have any questions regarding the report, or if we can be of further service to you, please call us.

Sincerely,
GEOTEST ENGINEERING, INC.
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EXECUTIVE SUMMARY

A geotechnical investigation was performed for the design and construction of the proposed improvements at the Mesa Road Pump Station as part of FY2013 Storm Water Pump Station and Flood Warning System Improvements in Houston, Texas.

The proposed improvements include construction of a new VFD/control building (10 feet x 18 feet) for the pump station, reconstruct the vault structure and replace the existing asphalt driveway. The proposed vault will be approximately 9 feet deep.

The purposes of this study were to evaluate soil and groundwater conditions and to provide geotechnical recommendations for the proposed building, vault and pavement. The investigation included drilling and sampling two (2) borings to depths ranging from 10 feet to 25 feet, performing laboratory tests on soil samples recovered from the borings, performing engineering analyses and developing geotechnical recommendations and preparing a geotechnical report.

The principal findings and conclusions developed from this investigation are as follows:

- The existing pavement as encountered in boring B-2 consists of 1-inch of asphalt underlain by 16 inches of lime stabilized sand and shell mix. The boring B-1 was drilled in the grass area.

- The subsurface soil beneath pavement and existing ground as encountered in borings B-1and B-2 consists of fill material consisting of soft to hard gray and brown and reddish brown and gray sandy lean clay to depth of 14 feet in boring B-1 and 10 feet in boring B-2, the termination depth of the boring. In boring B-1, the fill material is underlain by medium dense fine sand with silt to a depth 25 feet, the termination depth of the boring.

- Based on the available information from U.S. Geological Survey (USGS) Maps and in-house records relating to geologic faults for the project site, the nearest fault is the Clinton Fault and is located approximately 2,000 feet east of the project site.

- No groundwater was encountered in borings B-1 and B-2 drilled for this study.

- All excavation operations should be carried out in accordance with OSHA standards and the City of Houston Standard Specifications.

- The foundation recommendations for the proposed building and vault structure are presented in Section 5.3 and 5.4 of this report.

- The details of the recommended pavement section for the drive way replacement is given in Section 5.5 of this report:

1.0 INTRODUCTION

1.1 General

The City of Houston selected Chester Engineers, Inc. to perform engineering services for design and construction of proposed improvements at the Mesa Road Pump Station as part of FY2013 Storm Water Pump Station and Flood Warning System Improvements in Houston, Texas. Chester Engineers, Inc. retained Geotest Engineering, Inc. as part of the design team to perform geotechnical investigation for the above project.

1.2 Authorization

This study was authorized by Work Order No. P2-001 dated July 7, 2014 and by accepting Geotest Engineering, Inc. (Geotest) proposal No. 1140347299 dated April 17, 2014.

1.3 Location and Description of Project

The project site is located at 5405 Mesa Drive in Houston, Texas, within the Key Map Page and Grid 455 U.

The proposed improvements include construction of a new VFD/control building (10 feet x 18 feet) for the pump station, reconstruct the vault structure and replace the existing asphalt driveway. The proposed vault will be approximately 9-foot deep. The vicinity map of the project site is shown on Figure 1.

1.4 Purpose and Scope

The purposes of this study were to evaluate soil and groundwater conditions and to provide geotechnical recommendations for the proposed building, vault and pavement. The scope of this investigation consisted of the following:

- Drilled and sampled one (1) 25-foot boring and one (1) 10-foot boring.

- Performed appropriate laboratory tests in accordance with ASTM methods on selected samples to develop engineering properties of the soil.

- Reviewed available fault information to evaluate the potential for known active faults that may impact the project.

- Performed engineering analyses in accordance with the City of Houston Design Manual (July 2012) to develop geotechnical recommendations for the design and construction of the proposed building, vault structure and drive way pavement.

- Prepared a geotechnical report that will include all field data, laboratory test data and geotechnical recommendations.

2.0 FIELD INVESTIGATION

2.1 General

After obtaining the utilities clearance of proposed borings in the field, two (2) borings were drilled to the explored depths of 10 and 25 feet utilizing a truck mounted drilling rig. All the drilling and sampling were performed in accordance with appropriate ASTM procedures.

2.2 Geotechnical Borings

Subsurface conditions were explored by drilling and sampling two (2) soil borings (designated as B-1 and B-2) to depths of 10 and 25 feet. The approximate boring locations are shown on Figures 2.1 and 2.2, Plan of Borings. The survey information of the borings was provided by Chester Engineers, Inc.

In general, samples were obtained continuously to the depth of 10 feet, the explored depth in boring B-2. Samples were obtained continuously to 20 feet and intermittent sampling at 5 foot intervals to the termination depth of 25 feet in boring B-1. Cohesive soils were obtained with a 3-inch thin-walled tube sampler in general accordance with ASTM Method D1587. Cohesionless soils were obtained with 2-inch diameter split spoon sampler in general accordance with ASTM D1586. Each sample was removed from the sampler in the field, carefully examined and then logged by an experienced soils technician. Suitable portions of each sample were sealed and packaged for transportation to Geotest's Laboratory. The shear strength of cohesive soil samples was estimated using a pocket penetrometer in the field. Driving resistances for the split-barrel samples were recorded as "blows per foot" on the boring log. Water level measurements were made in the open boreholes at the time of drilling. All the borings were grouted with cement-bentonite grout after completion of drilling and obtaining water level measurements.

Detailed descriptions of the soils encountered in the borings are given on the boring logs presented on Figures A-1 and A-2 in Appendix A. A key to symbols and terms used on boring logs is given on Figure A-3 in Appendix A.

2.3 Piezometer Installation

No Piezometers were installed for this study.

3.0 LABORATORY TESTING

The laboratory testing program was designed to evaluate the pertinent physical properties and shear strength characteristics of the subsurface soils. Classification tests were performed on selected samples to aid in soil classification. All the tests were performed in accordance with ASTM Standards.

Undrained shear strengths of selected cohesive samples were measured by unconsolidated undrained (UU) triaxial compression tests (ASTM D 2850). The results of the UU triaxial compression tests are plotted on the boring logs as solid squares. The shear strength of cohesive samples was measured in the field with a calibrated hand pocket penetrometer and also in the laboratory with a Torvane. The shear strength values obtained from the penetrometer and Torvane are plotted on the boring logs as open circles and triangles, respectively.

Measurements of moisture content and dry unit weight were taken for each UU triaxial compression test sample. Moisture content (ASTM D 2216) measurements were also made on other samples to define the moisture profile at each boring location. The liquid and plastic limit tests (ASTM D 4318) and percent passing No. 200 sieves (ASTM D 1140) and sieve analysis (ASTM D 422) were performed on appropriate samples.

The result of all tests are tabulated or summarized on the boring logs presented on Figures A-1 and A-2 in Appendix A. The summary of laboratory tests is also presented in a tabular form on Figures B-1 and B-2 in Appendix B. The grain size distribution curve is presented on Figure B-3 in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Geology

The project area lies in the Beaumont Formation. The clays and sands of the Beaumont Formation are over-consolidated as a result of desiccation from frequent rising and lowering of the sea level and the groundwater table. Consequently, clays of this formation have moderate to high shear strength and relatively low compressibility. The sands of the Beaumont Formation are typically very fine and often silty. Further, there is occasional evidence in the Houston area of the occurrence of cemented material (sandstone and siltstone) deposits within the Beaumont Formation.

4.2 General Fault Information

A review of information in the Geotest library, relating to known surface and subsurface geologic faults in the general area of the project alignments, was undertaken. The available information consisted of U.S. Geological and NASA maps, open file reports and information contained in our files relating to geologic faults in the project area.

Based on the available information from U.S. Geological Survey (USGS) Maps and in-house records relating to geologic faults for the project site, the nearest fault is the Clinton Fault and is located approximately 2,000 feet east of the project site. Hence, a Phase I Geological Fault Study is not required for the project.

4.3 Existing Paving

The existing pavement as encountered in boring B-2 consists of 1-inch of asphalt underlain by 16 inches of lime stabilized sand and shell mix. The boring B-1 was drilled in the grass area.

4.4 Soils Stratigraphy

Based on the subsurface soils encountered in the boreholes, one (1) boring log profile was developed and is presented on Figure 3. To the left of each boring shown on the profile is an

indication of the consistency or density of each stratum. More than one consistency or density for an individual stratum indicates that the consistency or density is different at different depths within the stratum. For cohesive soils, consistency is related to the undrained shear strength of the soil. For cohesionless soils, the density of soil is measured by standard penetration test blows of the soil. To the right of each boring shown on the profile is the overall classification of the soil contained within each stratum. The symbols and abbreviations used on the boring log profile are given on Figure 4. The soil classification is based on ASTM Standards.

The subsurface soil beneath pavement and existing ground as encountered in borings B-1and B-2 and as shown on boring log profile presented on Figure 3, consist of fill material consisting of soft to hard gray and brown and reddish brown and gray sandy lean clay to depth of 14 feet in boring B-1 and 10 feet in boring B-2, the termination depth of the boring. In boring B-1, the fill material is underlain by medium dense fine sand with silt to a depth 25 feet, the termination depth of the boring.

The sandy lean clay fill is of medium to high plasticity with liquid limits ranging from 26 to 37 and plasticity indices ranging from 10 to 20. The fines content (passing No. 200 sieve) of fine sand with silt is about 5 percent. The fines content of sandy lean clay fill ranges from 52 to 66 percent.

4.5 Unsatisfactory Soil Conditions

In boring B-1, soft to medium stiff sandy lean clay fill material was encountered between the depths of 4 to 14 feet. Thus, extra precaution should be carried out during construction using appropriate construction equipment.

4.6 Water Levels

No groundwater was encountered in the borings B-1 and B-2 drilled for this study.

However, it should be noted that various environmental and man-made factors such as amount of precipitation, nearby subsurface construction activities, and change in area drainage can substantially influence the groundwater level.

4.7 Environmental Concerns

No environmental concerns were observed or noticed in any of the borings (B-1 and B-2) drilled for this study.

5.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

5.1 General

The proposed improvements include construction of a new VFD/control building (10 feet x 18 feet) for the pump station, reconstruction of the vault structure and replacement of the existing asphalt driveway. The proposed vault will be approximately 9-foot deep.

5.2 Excavation

Based on the information provided by Chester Engineers, Inc., it is understood that the proposed vault structure will be installed by open cut method of construction. The following subsections provide information for the design and construction of the installation of vault structure.

5.2.1 Geotechnical Parameters. Based on the soil conditions revealed by the boring B-1, geotechnical parameters were developed for the design of open cut construction for the installation of vault structure. The design parameters are provided in Table 1. For design, the groundwater level should be assumed to exist at the ground surface.

5.2.2 Excavation Stability. The open excavation may be shored or supported by some other equivalent means used to provide safety for workers and adjacent structures, if any. The excavating operations should be in accordance with OSHA Standards, OSHA 2207, Subpart P, latest revision and the City of Houston Standard Specification.

- Excavation Shallower Than 5 Feet - Excavations that are less than 5 feet deep **(critical height)** should be effectively protected when an indication of dangerous ground movement is anticipated.

- Excavations Deeper Than 5 Feet - Excavations that are deeper than 5 feet should be sloped, shored, sheeted, braced or supported by some other equivalent means or protection such that workers are not exposed to moving ground or cave-ins. The shoring should be in accordance

with the trench safety requirements as per OSHA Standards. The following items provide design criteria for excavation stability.

(i) OSHA Soil Type. Based on the soil conditions revealed by borings drilled for this study and assumed groundwater level at surface, OSHA soil type "C" should be used for determination of allowable maximum slope and/or the design of shoring along the alignment for full proposed depth of open excavation. For shoring deeper than 20 feet (if needed), an engineering evaluation is required and deeper soil borings will be needed.

(ii) Excavation Support Earth Pressure. Based on the subsurface conditions indicated by our field investigation and laboratory testing results, excavation support earth pressure diagram was developed and is presented on Figure 5. This pressure diagram can be used for the design of temporary trench bracing. For a trench box, a lateral earth pressure resulting from an equivalent fluid with a unit weight of 94 pcf can be used. The effects of any surcharge loads at the ground surface should be added to the computed lateral earth pressures. A surcharge load, q, will typically result in a lateral load equal to 0.5 q. The above value of equivalent fluid pressure is based on assumption that the groundwater level is near the ground surface, since these conditions may exist after a heavy rain or flooding.

(iii) Bottom Stability. In braced cuts, if tight sheeting is terminated at the base of the cut, the bottom of the excavation can become unstable. The parameters that govern the stability of the excavation base are the soil shear strength and the differential hydrostatic head between the groundwater level within the retained soils and the groundwater level at the interior of the trench excavation. For cut in cohesive soils as encountered for the proposed excavation depth of 9 feet, the bottom stability can be evaluated as outlined on Figure 6.

5.2.3 Groundwater Control. Excavations for the proposed vault structure may encounter groundwater seepage to varying degrees depending upon the groundwater conditions at the time of

construction and the location and depth of the trench. Based on the soil conditions identified in the boring for the proposed vault installation, all the excavation will be in cohesive soils.

In general for cohesive soils as predominantly encountered in boring B-1 (for the excavation depth of 9 feet), the groundwater (if encountered) may be managed by collection in excavation bottom sumps for pumped disposal. It is recommended that the actual groundwater conditions should be verified by the contractor at the time of construction. Groundwater control should be performed in general accordance with the City of Houston Standard Specifications, Section 01578.

5.3 Proposed Vault Structure Foundation

5.3.1 Foundation Design Recommendations. The following items provide recommendations and design criteria for construction of the proposed vault structure. Based on the provided information, the total loading for the vault structure is about 1,200 psf.

- Allowable Bearing Pressures. The mat foundation for supporting the proposed vault structure placed at a depth 9 feet [into soft to medium stiff sandy lean clay] should be designed for an allowable (net) bearing pressure of 1,200 psf for total loads. These allowable bearing pressures include a safety factor of 2.0. The above recommendations assume that the final bearing surfaces consist of undisturbed natural soils and that underlying semi-transmissive zones are properly pressure-relieved and stable undisturbed bearing surfaces are attained.

- Lateral Earth Pressure. The pressure diagram presented on Figure 5 can be used for the design of braced excavation. The lateral earth pressure diagram presented on Figure 7 is applicable for the design of the permanent walls.

- Hydrostatic Uplift Resistance. Structures extending below the groundwater level should be designed to resist uplift pressure resulting from excess piezometric head. Design uplift pressures should be computed based on the assumption that the water table is at ground surface. To resist the hydrostatic uplift at the bottom of the structure, one of the following sources of resistance can be utilized in each of the

designs.

a. Dead weight of structure,
b. Weight of soil above base extensions plus weight of structure, or
c. Soil-wall friction plus dead weight of structure.

The uplift force and resistance to uplift should be computed as detailed on Figure 8. In determining the configuration and dimensions of the structure using one of the approaches presented on Figure 8, the following factors of safety are recommended.

a. Dead weight of concrete structure, $S_{f1} = 1.10$,
b. Weight of soil (backfill) above base extension, $S_{f2} = 1.5$, and
c. Soil-wall friction, $S_{f3} = 3.0$.

Friction resistance should be discounted for the upper 5 feet, since this zone is affected by seasonal moisture changes.

5.3.2 Protection of Below Grade Structures. The design of the proper means for protection of below grade structures will depend upon the potential of the aggressivity or corrosivity of soil and groundwater properties. The aggressivity testing was not within the scope of this study. The design of the protection of below grade structures is beyond the scope of services for this study.

5.3.3 Structure Backfill. Excavations for the proposed structures should be backfilled in accordance with the City of Houston Standard Specifications, Section 02316, "Excavation and Backfill for Structures."

5.4 Proposed VFD/Control Building.

5.4.1 Description. The proposed VFD/Control Building is approximately 10 ft x 18 ft in plan dimensions and will be supported on slab on grade foundation. Based on the provided information, the total load for the VFD/Control Building is approximately 800 psf.

5.4.2 Foundation Type, Depth and Allowable Bearing Pressure. Based on the information provided, we understand that the new building will be at the existing grade with minimal cut and fill. As revealed by boring B-1, the surficial soils consists of medium plasticity soft to medium stiff sandy lean clay. These soils have low to moderate potential for shrink/swell movements. Hence, the proposed building can be supported on slab-on-grade foundation.

The slab on grade foundation can be designed for an allowable (net) bearing pressure of 1,200 psf for total loads or 800 psf for sustained loads whichever results in the larger foundation area. These allowable (net) bearing pressures contain safety factors of 2.0 for total loads and 3.0 for sustained loads, respectively. As per the information provided by structural engineer, the grade beam will be designed in accordance with City of Houston Guideline Drawing ZOA1. The grade beams should have tensile reinforcement both at the top and at the bottom of the beam.

Due to the soft soils encountered in the boring (from 4 to 14 feet), a higher bearing pressure can be achieved by removal of the existing soils to a depth of 4 feet and replace with cement stabilized sand to provide a suitable bearing and bridging over the soft soil encountered below 4 feet. The cement stabilized sand structural fill should be placed in accordance with Section 5.4.4 of this report. After removal and replacement of the existing soils, the grade beam for slab-on-grade foundation placed at 24 inches below the existing grade into cement stabilized sand, can be designed for an allowable (net) bearing pressure of 2,500 psf for total loads or 1,600 psf for sustained loads whichever results in the larger foundation area. These allowable (net) bearing pressures contain safety factors of 2.0 for total loads of 3.0 for sustained loads, respectively.

5.4.3 Foundation Settlement. Depending upon the footing size and magnitude of the sustained footing pressure, some total and differential settlements should be anticipated due to consolidation of the foundation soils. A settlement analysis was performed based on the total load of 800 psf. The results of the settlement analyses indicated that the differential settlement for the proposed building is less than 0.5 inches.

5.4.4 Site Preparation and Structural Fill Requirements. The site should be cleared of all debris, grubbed and stripped of all organic material, soft soils and foreign material from the building and paved areas. Stripped areas should be appropriately graded and shaped to prevent ponding of water on the site.

Should any structural fill required to raise the grade or backfill grub holes should consist of lean clay with a liquid limit less than 40 and a plasticity index between 8 and 20 or cement stabilized sand in building footprint. The structural fill should be compacted at moisture content within three percent above optimum to reduce swelling potential of the compacted fill. The fill material should be placed in loose lifts not exceeding eight inches and should be compacted to a minimum of 98 percent of the maximum dry density as determined by ASTM D 698 in building area and 95 percent of the maximum dry density as determined by ASTM D 698 in driveway. The cement stabilized sand should be placed in accordance with COH Standard Specification Section 02321 "Cement Stabilized Sand." The structural fill or cement stabilized sand should extend at least five feet outside the building and paving area. The onsite surficial medium plasticity sandy lean clay soils free of debris and organic material can be used as structural fill material.

5.4.5 Building Pad. During construction, it is essential that the finished surface be protected from excessive drying. Any material required to raise the grade should meet the criteria described in the section "Site Preparation and Structural Fill Requirements." The structural fill, if needed, should extend at least 5 feet beyond the slab area.

5.4.6 Floor Slab Construction. The floor slab can be supported on existing fill material or cement stabilized sand (for higher bearing pressure).

5.4.7 Landscaping. It is recommended that no large trees exist or be planted within 15 feet of the building and preferably within the mature drip line. Any flowerbeds or open lawn areas, if provided near the building areas, should have a good sprinkler system to minimize the moisture variations in the subsurface soils. It is imperative that the sprinkler systems installed in the proximity of structures be free from leaks, which could provide a continuous source of moisture and promote differential swelling of the near surface soils.

5.4.8 Surface Drainage. The following drainage precautions should be observed during construction and maintained at all times after the building has been completed:

1. All backfill soils around the building should be moistened and compacted to at least 95 percent of Standard Proctor Density (ASTM D 698).

2. The ground surface surrounding the exterior of the building should be sloped to drain away from the building in all directions.

3. Roof downspouts and drains should discharge well beyond the limits of the foundation backfill and into pipes or paved areas.

5.5 Pavement Structure Design

5.5.1 Preparation of Pavement Subgrade. Subgrade preparation for the proposed pavement should consist of demolition, stripping, proof-rolling and stabilization. The following procedures for subgrade are recommended:

1. After demolition of the existing pavement, strip the surficial soil to a suitable depth to remove all surficial debris and vegetation and to achieve grade. In any isolated area where soft, compressible or very loose soils are encountered, additional stripping may be required. Stripping should extend to a minimum of 2 feet (where possible) beyond the edge of the proposed pavement.

2. The surface exposed after stripping should be proof-rolled with a minimum of 3 passes of a 30-ton pneumatic-tired roller or a heavy loaded truck utilizing a tire pressure of approximately 90 psi. If rutting develops, the tire pressure should be reduced. The purpose of the proof rolling operation is to identify any underlying zones or pockets of soft soils and to remove such weak materials.

3. Based on the borings, the subgrade support soils consist of low to medium plasticity sandy lean clay. To accelerate the construction and provide stable subgrade on which to construct the pavement section, it is recommended that the paved drive areas be stabilized to a minimum depth of 6 inches with 5% lime (by dry unit weight of soil).

Following treatment, the subgrade material should be compacted to at least 95 percent of Standard Proctor maximum density in accordance with ASTM D 698, at a moisture content within 3 percent above the optimum moisture content.

Pavement Thickness. Traffic design data is not available for this project. However, after the subgrade is prepared as described above, the minimum recommended thickness of rigid or flexible pavement is presented below:

Type of Loads	**Rigid Pavement**	**Flexible Pavement**
Heavy Loads (Heavy Truck traffic, 20,000 lbs)	7″ PCC	3″ HMAC over 6″ Black base or 9″ Limestone
Medium Loads (Medium Truck and Heavy Use Driveways)	6″ PCC	2″ HMAC over 5″ Black base or 8″ Limestone
Light Loads	5″ PCC	2″ HMAC over 4″ Black base or 6″ Limestone

PCC = Portland cement Concrete
HMAC = Hot Mix Asphaltic Concrete
Note: Modulus of Rupture of Concrete, M_R = 600 psi (assumed)

6.0 CONSTRUCTION CONSIDERATIONS

It is recommended that the foundation excavation be inspected by an experienced geotechnical engineer or senior soils technician, prior to placing steel and concrete. The excavation should be checked to verify that (a) the foundation has been constructed to the specified dimensions and is placed at correct depth and into appropriate stratum with adequate bearing capacity as recommended in the report, (b) the loose cuttings and any soft-compressible materials have been removed from the bottom of the excavation.

Placement of the concrete should be accomplished as soon as possible to prevent changes in the state of stress and caving of the foundation soils. No footings should be poured without the prior approval of the project engineer, architect or owner's representative.

7.0 LIMITATIONS

The description of subsurface conditions and the design information contained in this report are based on the soil borings made at the time of drilling at specific locations. However, some variation in soil conditions may occur between soil borings. Should any subsurface conditions other than those described in our boring logs be encountered, Geotest should be immediately notified so that further investigation and supplemental recommendations can be provided. The depth of the groundwater level may vary with changes in environmental conditions such as frequency and magnitude of rainfall. The stratification lines on the log of borings represent the approximate boundaries between soil types, however, the transition between soil types may be more gradual than depicted.

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TABLE

Table

Geotechnical Design Parameter Summary – Open Cut Excavation 1

TABLE 1

GEOTECHNICAL DESIGN PARAMETER SUMMARY
OPEN-CUT EXCAVATION

Boring Nos.	Stratigraphic Unit	Range of Depths, ft	Wet Unit Weight, γ, pcf	Submerged Unit Weight, γ', pcf	Undrained Cohesion, psf	Internal Friction Angle, φ, degree
B-1	Cohesive	0-2	130	65	1,600	--
		2-14	125	63	500	--
	Cohesionless	14-25	106	53	--	30

Note: 1) Cohesive soils include Sandy Lean Clay.
2) Cohesionless soils include Fine Sand w/silt.

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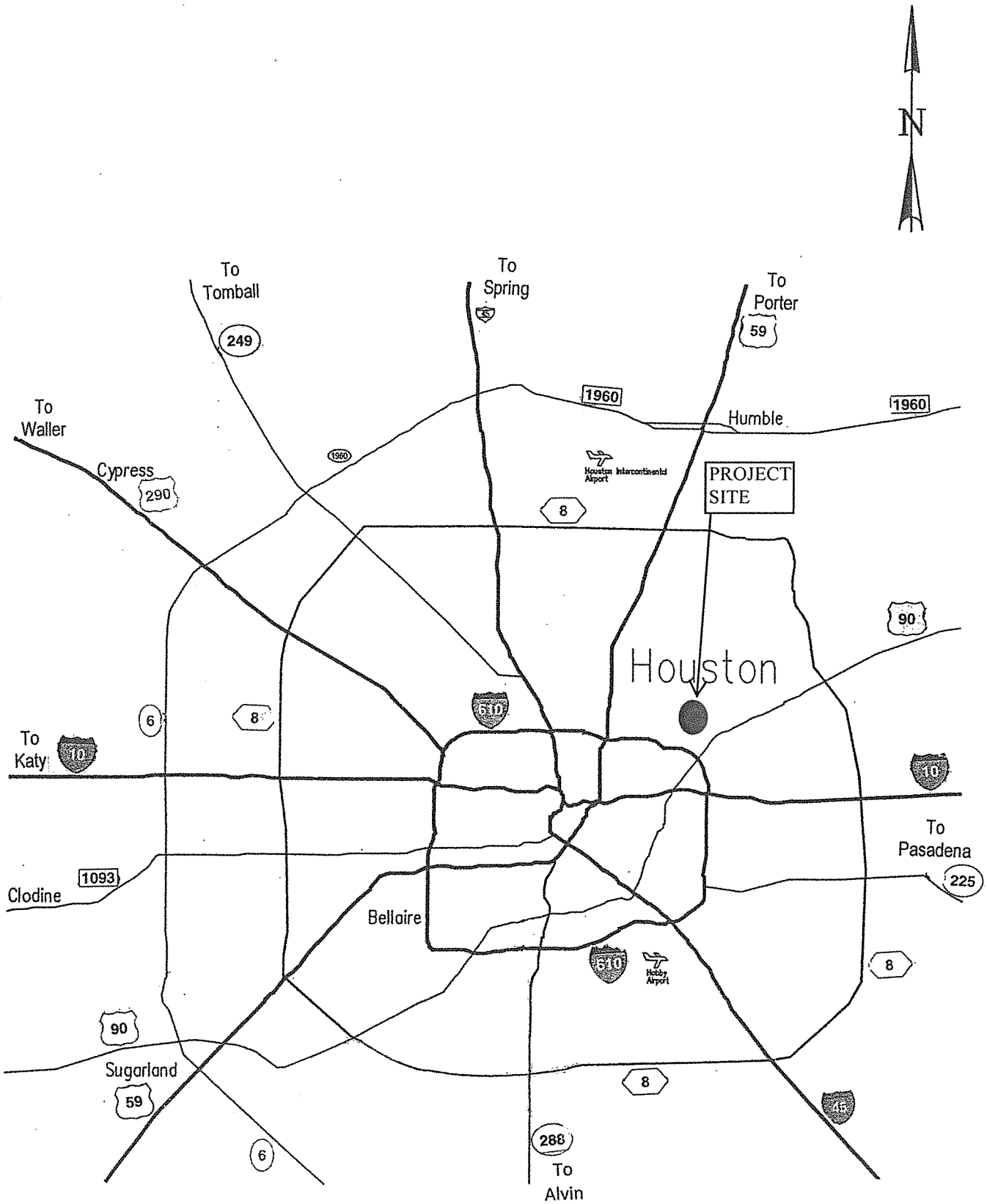
Job No. 1140203801



VICINITY MAP

(NOT TO SCALE)

Geotest Engineering, Inc.

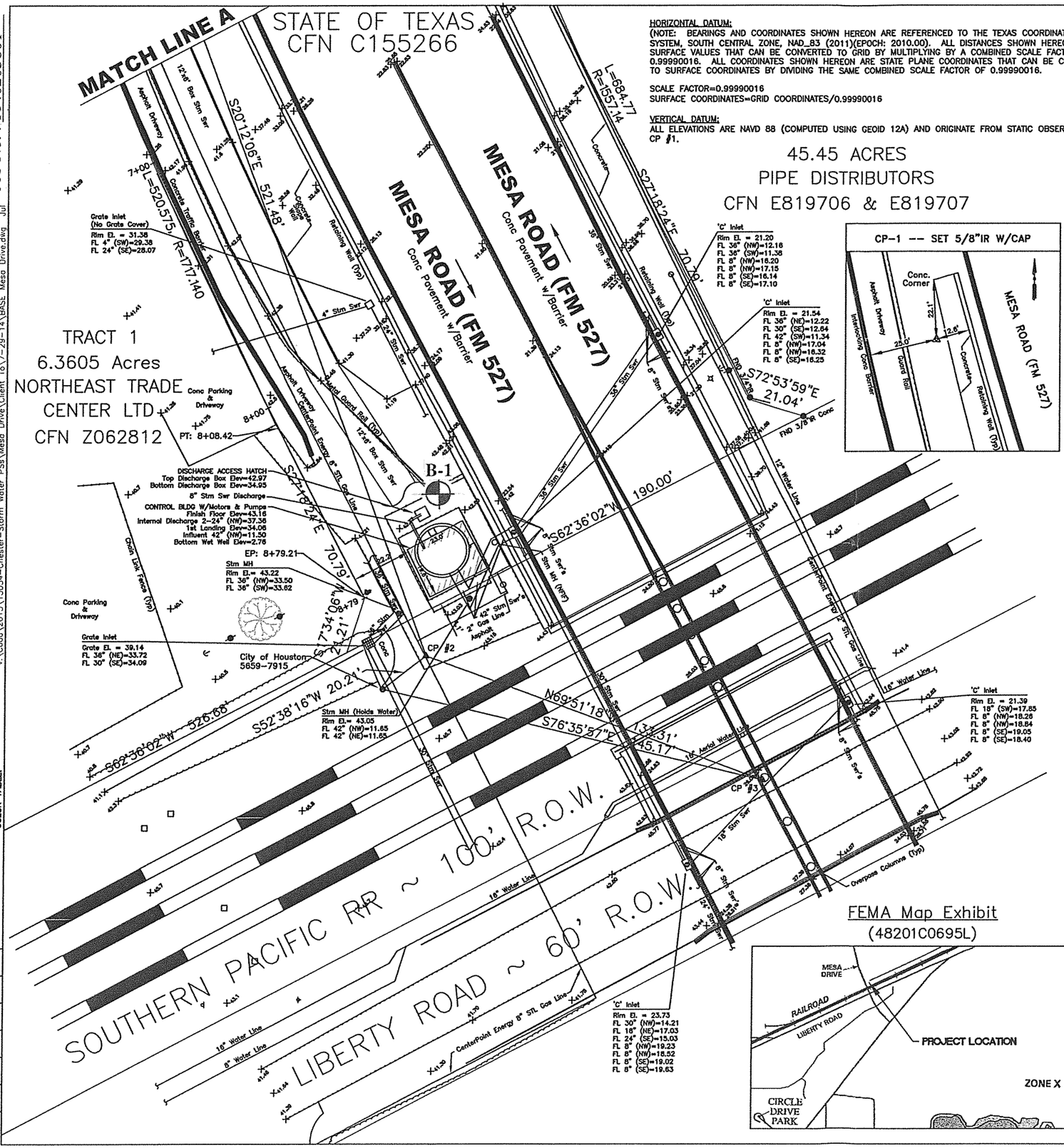
FIGURE 1

Job No. : 1140203801

V:\Cadd\2013\13054-Chester-Storm Water PSs\Mesa Drive\Client To\7-29-14\BASE Mesa Drive.dwg Jul

COLOR TABLE:

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS



HORIZONTAL DATUM:
(NOTE: BEARINGS AND COORDINATES SHOWN HEREON ARE REFERENCED TO THE TEXAS COORDINATE SYSTEM, SOUTH CENTRAL ZONE, NAD_83 (2011)(EPOCH: 2010.00). ALL DISTANCES SHOWN HEREON ARE SURFACE VALUES THAT CAN BE CONVERTED TO GRID BY MULTIPLYING BY A COMBINED SCALE FACTOR OF 0.99990016. ALL COORDINATES SHOWN HEREON ARE STATE PLANE COORDINATES THAT CAN BE CONVERTED TO SURFACE COORDINATES BY DIVIDING THE SAME COMBINED SCALE FACTOR OF 0.99990016.

SCALE FACTOR=0.99990016
SURFACE COORDINATES=GRID COORDINATES/0.99990016

VERTICAL DATUM:
ALL ELEVATIONS ARE NAVD 88 (COMPUTED USING GEOID 12A) AND ORIGINATE FROM STATIC OBSERVATION AT CP #1.

45.45 ACRES
PIPE DISTRIBUTORS
CFN E819706 & E819707

CP-1 -- SET 5/8"IR W/CAP

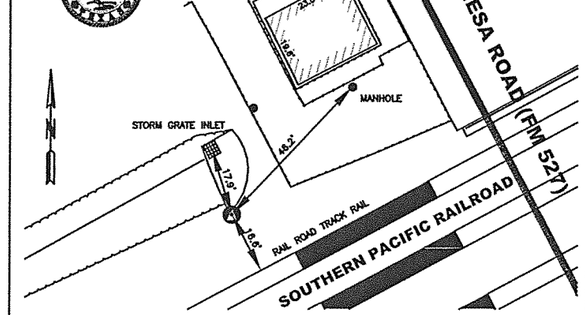
FEMA Map Exhibit
(48201C0695L)

TBM/CONTROL POINTS (GRID COORDINATES-GEOID 12A)				
CP #	NORTHING	EASTING	ELEVATION	DESCRIPTION
1	13,861,818.22	3,153,547.85	40.95	5/8"IR W/CAP PBM (STATIC)
2	13,861,328.87	3,153,727.91	43.21	MAG NAIL
3	13,861,282.96	3,153,711.85	43.63	MAG NAIL

CITY OF HOUSTON MONUMENT (5659-7915) (GRID COORDINATES-GEOID 12A)				
#	NORTHING	EASTING	ELEVATION	DESCRIPTION
3091	13,861,316.60	3,153,711.85	42.60	BRASS DISC SET IN CONC

CITY OF HOUSTON

CITY SURVEY 5659-7915
SITE CONTROL MONUMENT



Project WBS# M-000241-0008-3
Texas Coordinate System of NAD 1983, South Central Zone, U. S. Survey Feet
X= 3,153,711.85'
Y= 13,861,316.60'
Lat.= N 29°48'39.08"
Lon.= W 95°15'43.05"
Reference Frame Used: N/A
Vertical Adjustment used: Conventional
General Location: ON WEST SIDE OF MESA ROAD (FM 527), APPROXIMATELY 115' NORTH OF LIBERTY ROAD.

Keymap Page: 455 U
Orthometric Elevation = 42.60
Ellipsoid Height= NA
Geoid: 12A
Datum Source & Adjustment: Static GPS Observation made on CP#1, 11/08/13.

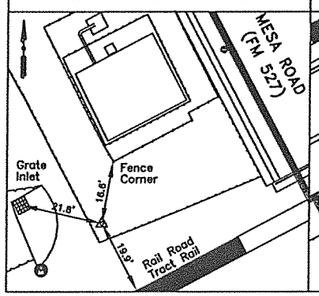
Date Set: 12/19/2013 Type of Mark: BRASS DISK
3 Nearest project control points (bearings and distances stated below):
1. CP #2, SET MAG NAIL, N52°38'16"E, 20.21'
2. CP #3, SET MAG NAIL, S76°35'57"E, 145.17'
3. CP #1, SET 5/8IR W/CAP, N18°06'17"W, 527.80'

NOTE:
1. Bearings are grid bearings.
2. Scale Factor (S.F)= 0.99990016
3. Surface= Grid / S.F.

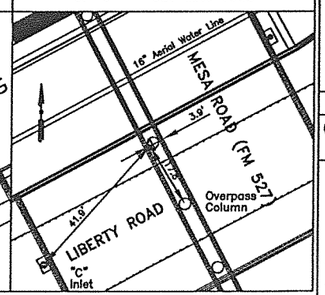
Surveyed By:
AMANI ENGINEERING, INC.
8313 SW FWY., SUITE 350
HOUSTON, TX 77074

STATE OF TEXAS REGISTERED RICKY J. RICHARDSON 5445 PROFESSIONAL LAND SURVEYOR

CP-2 -- SET MAG NAIL



CP-3 -- SET MAG NAIL



NOTES:

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2.) ALL DISTANCES SHOWN ARE SURFACE.

3.) ELEVATIONS ARE BASED ON CP #1 (AS SHOWN).

PROJECT AREA IS IN ZONE X UNSHADED (AREAS OUTSIDE 500-YEAR FLOOD) PER FEMA FIRM MAP NUMBERS 48201C0685L & 48201C0695L, MAPS REVISED JUNE 18, 2007. DETERMINATION OF FLOOD ZONE IS BASED ON THE GRAPHICAL DELINEATION OF ZONES AS DEPICTED ON THE FLOOD PLAIN MAPS.

WARNING
0 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DWG NO
C-01

KEY MAP NO: 455U

LEGEND

- POWER POLE
- GUARD POST
- ELECTRIC JUNCTION BOX
- POWER POLE W/GUY
- EXIST WATER VALVE
- EXISTING MANHOLE (STORM OR SANITARY)
- H.L. & P. MANHOLE
- INLET (GRATE TYPE)
- TRAVERSE POINT
- ELECTRICAL LIGHT POLE
- BORE HOLE
- MONITORING WELL
- CITY OF HOUSTON RECORD

GRAPHIC SCALE
20 0 10 20
(IN FEET)
1 inch = 20 ft.

No.	Date	Revisions	App.

PRIVATE UTILITY LINES SHOWN

CENTER POINT ENERGY/GAS FACILITIES

SBC CO. VALID FOR ONE YEAR
APPROVED ONLY FOR UNDERGROUND CONDUIT FACILITIES UNLESS NOTED.

CENTER POINT ENERGY/ELECTRIC FACILITIES
APPROVED ONLY FOR CROSSING UNDERGROUND DUCTLINES UNLESS NOTED. VALID AT TIME OF REVIEW ONLY.

CABLE COMPANY

APPROVED: DATE:
DESIGNED BY: DRAWN BY:

CHESTER ENGINEERS
1001 WEST LOOP SOUTH, SUITE 605
HOUSTON, TEXAS 77027
P:713-877-8774 F:713-877-8785

AMANI ENGINEERING, INC.
8313 SOUTHWEST FREEWAY
SUITE 350
HOUSTON, TEXAS 77074
TEL: (713) 270-5700
TBPE REG. NO.: F-4528
TBPLS REG. NO.: 100282-00

STATE OF TEXAS REGISTERED RICKY J. RICHARDSON 5445 PROFESSIONAL LAND SURVEYOR

SURVEYED BY: AMANI FB NO. P-[illegible]

CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS AND ENGINEERING

5405 MESA DRIVE PUMP STATION
TOPOGRAPHIC SURVEY
AND
CONTROL MAP

PLAN OF BORING

WBS NUMBER
M-430241-0009-4
DRAWING SCALE
1"=20' on 22"x34" 1"=40' on 11"x17"
CITY OF HOUSTON PM
REZA ARTI, P.E.
SHEET NO. 13 OF

FIGURE 2.1

Job No. : 1140203801

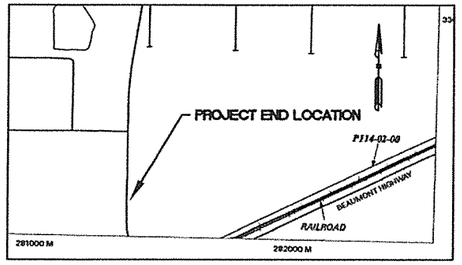
V:\Cadd\2013\13054-Chester-Storm Water PSs\Mesa Drive\Client To\7-29-14\BASE Mesa Drive 2.dwg

COLOR TABLE:

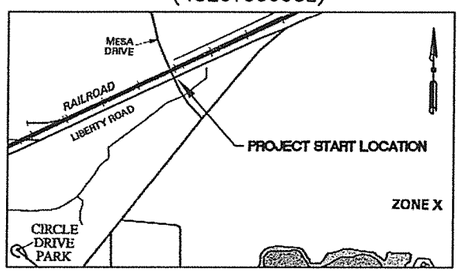
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS

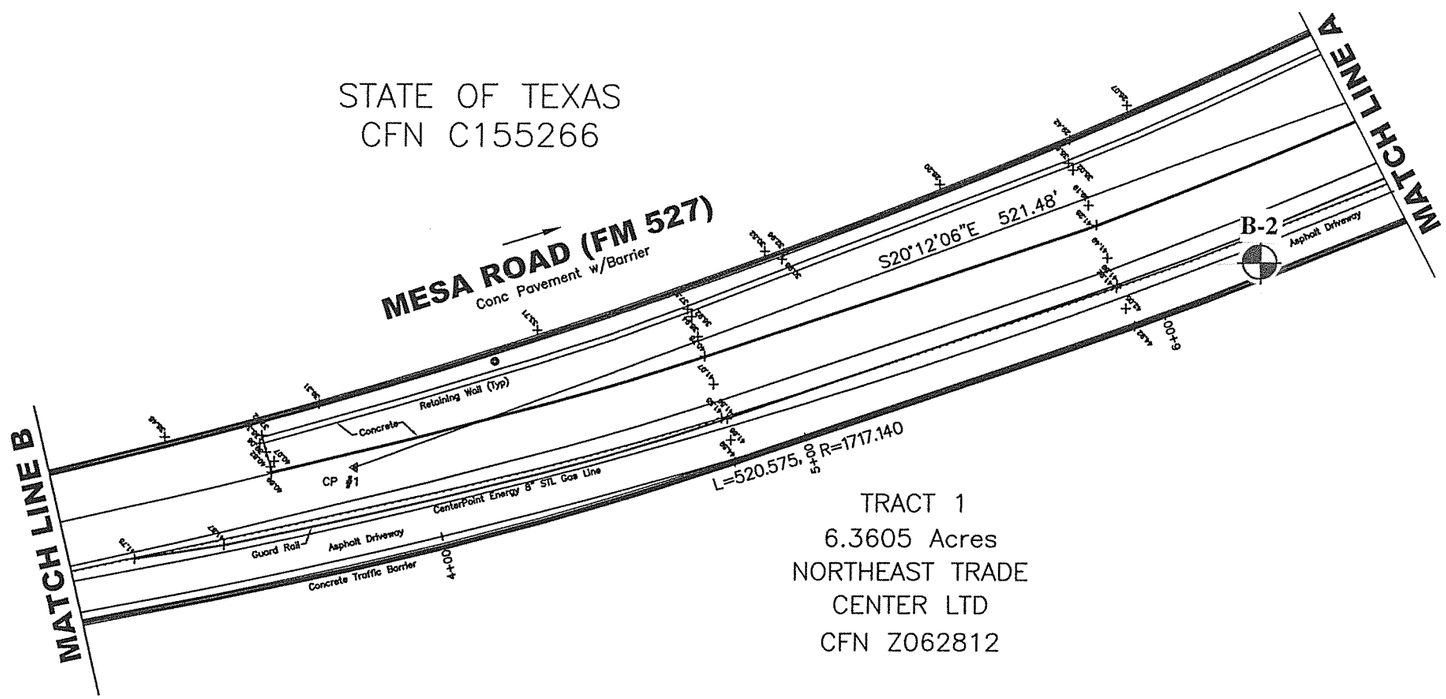
0 1 2 3

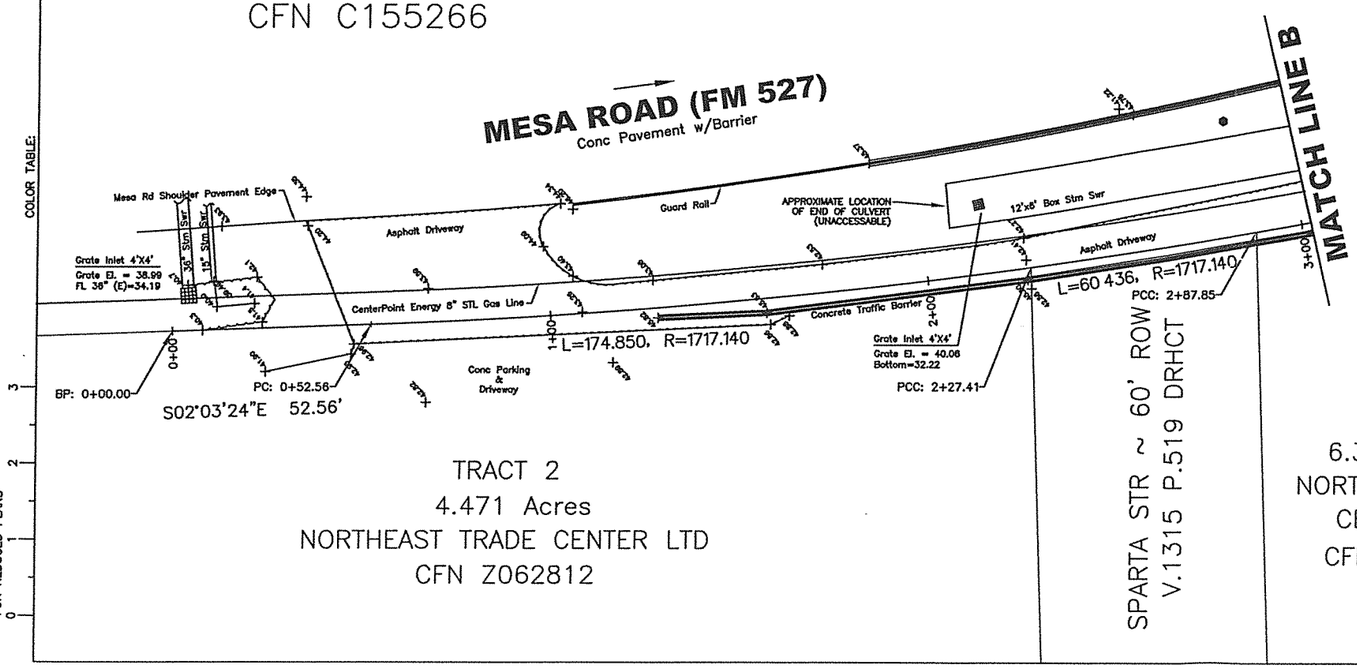
FEMA Map Exhibit
(48201C0685L)



FEMA Map Exhibit
(48201C0695L)







GRAPHIC SCALE
20 0 10 20
(IN FEET)
1 inch = 20 ft.

KEY MAP NO: 455U

LEGEND

- POWER POLE
- GUARD POST
- ELECTRIC JUNCTION BOX
- POWER POLE W/GUY
- EXIST WATER VALVE
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- H.L. & P. MANHOLE
- INLET (GRATE TYPE)
- TRAVERSE POINT
- ELECTRICAL LIGHT POLE
- BORE HOLE
- MONITORING WELL
- CITY OF HOUSTON RECORD
- TRAFFIC SIGNAL POLE

NOTES:

1.) BEARINGS AND COORDINATES ARE BASED ON TEXAS COORDINATE SYSTEM, SOUTH CENTRAL ZONE, NAD 1983.

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WARNING
0 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DWG NO
C-02

No.	Date	Revisions	App.

PRIVATE UTILITY LINES SHOWN

CENTER POINT ENERGY/GAS FACILITIES

SBC CO. VALID FOR ONE YEAR
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CENTER POINT ENERGY/ELECTRIC FACILITIES
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TBPE REG. NO.: F-4528
TBPLS REG. NO.: 100282-00

STATE OF TEXAS REGISTERED RICKY J. RICHARDSON 5445 LAND SURVEYOR
5/22/14

SURVEYED BY: AMANI FB NO. P-5807

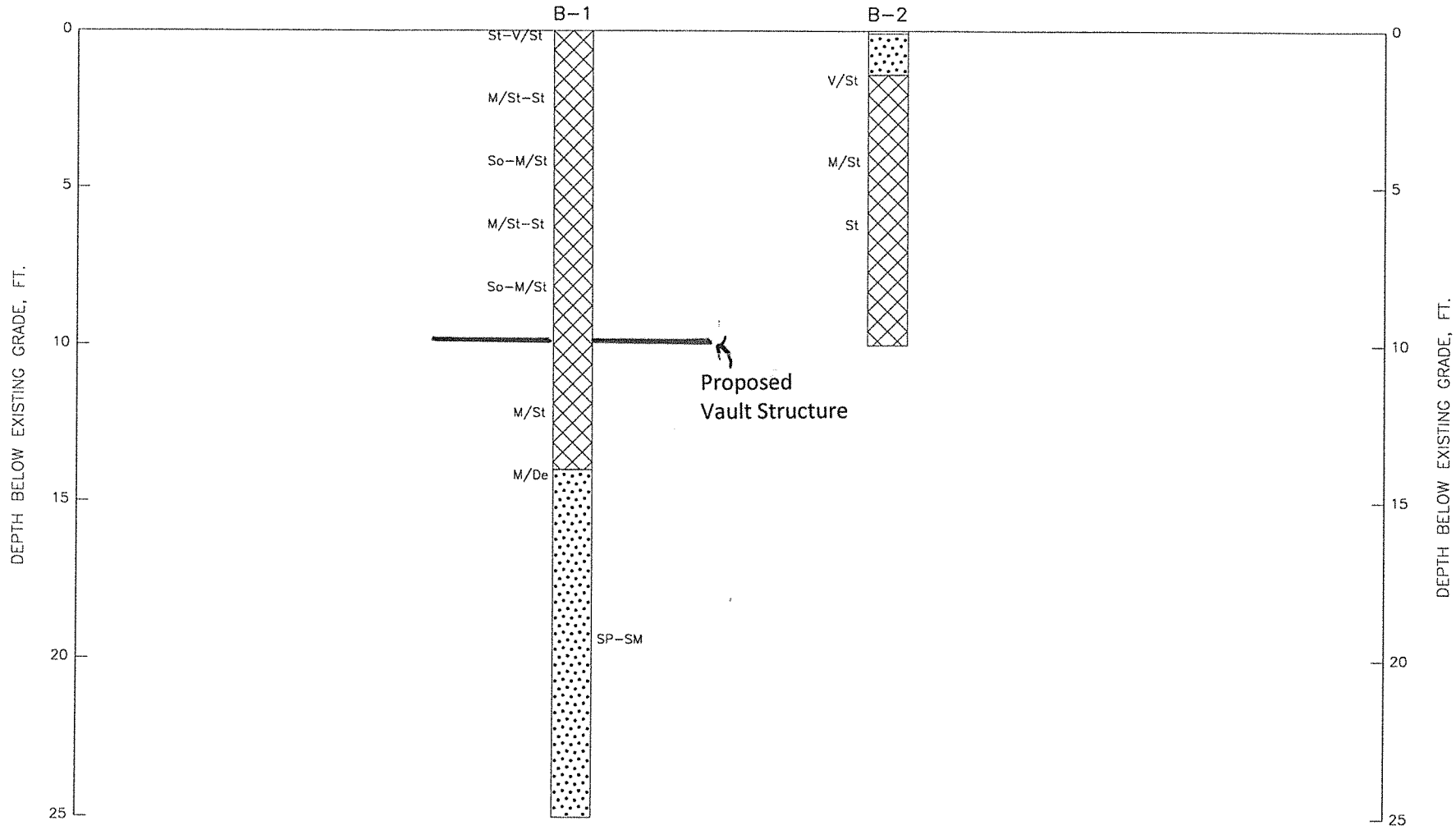
CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS AND ENGINEERING

5405 MESA DRIVE PUMP STATION
TOPOGRAPHIC SURVEY
AND
CONTROL MAP

PLAN OF BORING

WBS NUMBER	M-430241-0009-4
DRAWING SCALE	1"=20' on 22"x34" 1"=40' on 11"x17"
CITY OF HOUSTON PM	REZA ARTI, P.E.
SHEET NO. 14 OF	

FIGURE 2.2



GENERAL NOTES:

1. See Figure 2.1 and 2.2 for approximate location of borings and profile section.
2. Data concerning subsurface conditions have been obtained at boring locations only. Actual conditions between borings may differ from the profile shown here.
3. See logs of boring for detailed description of soils encountered in each borehole.
4. See Figure 4 for symbols and abbreviations used on this profile.

BORING LOG PROFILE
MESA ROAD PUMP STATION

0 50 100 150 200

HORIZONTAL SCALE IN FEET

Job No. 1140203801

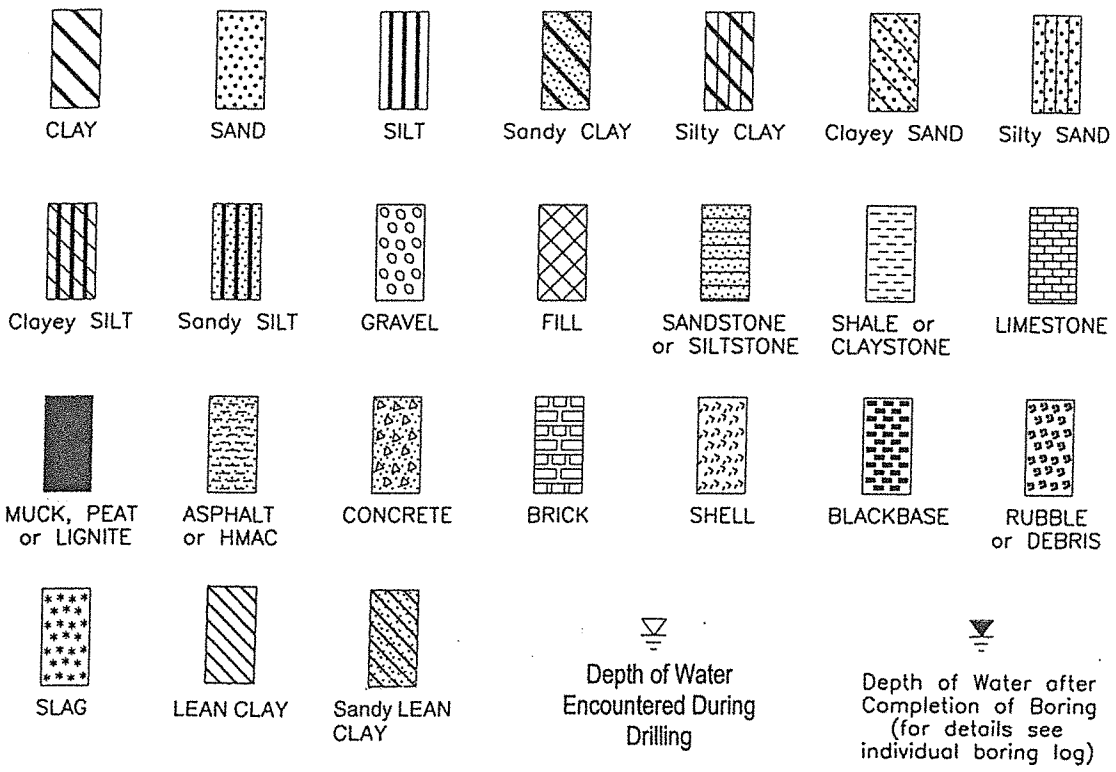
Geotest Engineering, Inc.

FIGURE 3

Job No. 1140203801

SYMBOLS AND ABBREVIATIONS USED ON BORING LOG PROFILE

LEGEND



ABBREVIATIONS USED FOR CONSISTENCY/DENSITY

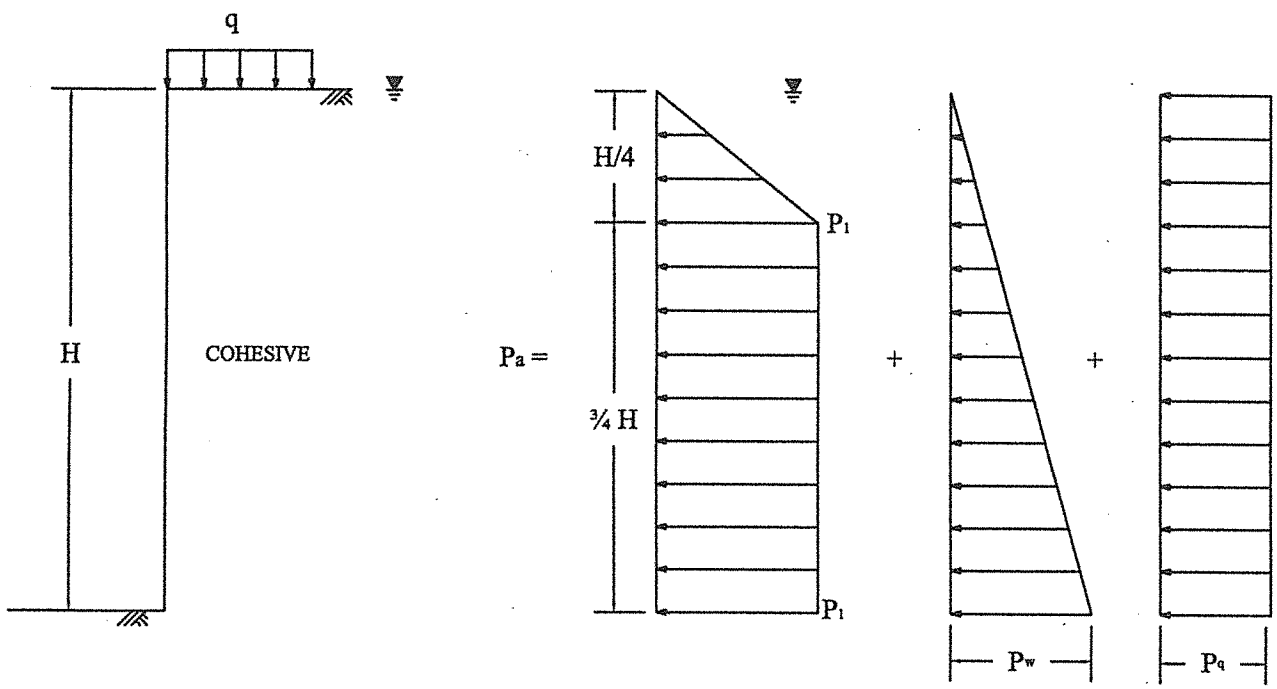
COHESIVE SOILS

V/So	:	Very Soft
So	:	Soft
Fm	:	Firm
M/St	:	Medium Stiff
St	:	Stiff
V/St	:	Very Stiff
Hd	:	Hard
V/Hd	:	Very Hard

COHESIONLESS SOILS

V/Lo	:	Very Loose
Lo	:	Loose
S/Co	:	Slightly Compact
Co	:	Compact
M/De	:	Medium Dense
De	:	Dense
V/De	:	Very Dense

Job No. 1140203801



TYPICAL SOIL PARAMETERS

See Table 1 for typical values of soil parameters

BRACED WALL

For $\gamma H/c > 4$

$P_1 = \gamma_c' H [1-(4c/\gamma_c' H)]$ or $P_1 = 0.3\ \gamma_c' H$ (whichever is higher)

$P_w = \gamma_w H = 62.4\ H$

$P_q = 0.5\ q$

Where:

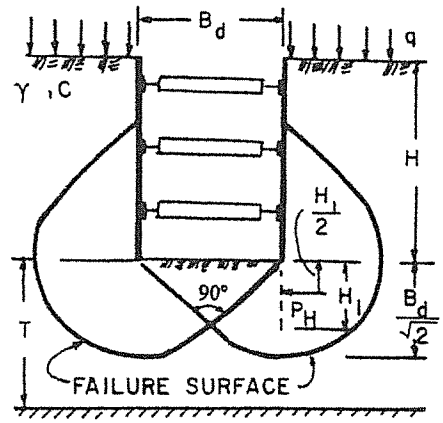
γ_c' = Submerged unit weight of cohesive soil, pcf;
γ_w = Unit weight of water, pcf;
q = Surcharge load at surface, psf;
P_a = Lateral pressure, psf;
P_1 = Active earth pressure, psf;
P_q = Horizontal pressure due to surcharge, psf;
P_w = Hydrostatic pressure due to groundwater, psf;
H = Depth of braced excavation, feet
c = Shear strength of cohesive soil, psf;

EXCAVATION SUPPORT EARTH PRESSURE

SUBMERGED SOFT TO MEDIUM STIFF COHESIVE SOIL

Job No. 1140203801

CUT IN COHESIVE SOIL,
DEPTH OF COHESIVE SOIL UNLIMITED ($T > 0.7\ B_d$)
L = LENGTH OF CUT



If sheeting terminates at base of cut:

$$\text{Safety factor, } F_S = \frac{N_C C}{\gamma H + q}$$

N_C = Bearing capacity factor, which depends on dimensions of the excavation : B_d , L and H (use N_C from graph below)

C = Undrained shear strength of clay in failure zone beneath and surrounding base of cut

γ = Wet unit weight of soil (see Table 1)

q = Surface surcharge (assumed q = 500 psf)

If safety factor is less than 1.5, sheeting or soldier piles must be carried below the base of cut to insure stability - (see note)

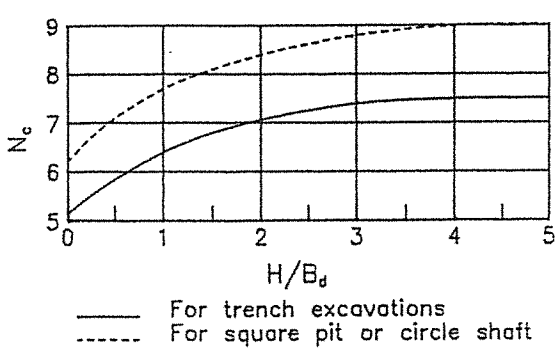
$$H_1 = \text{Buried length} = \frac{B_d}{2} \geq 5 \text{ feet}$$

Note : If soldier piles are used, the center to center spacing should not exceed 3 times the width or diameter of soldier pile .

Force on buried length, P_H:

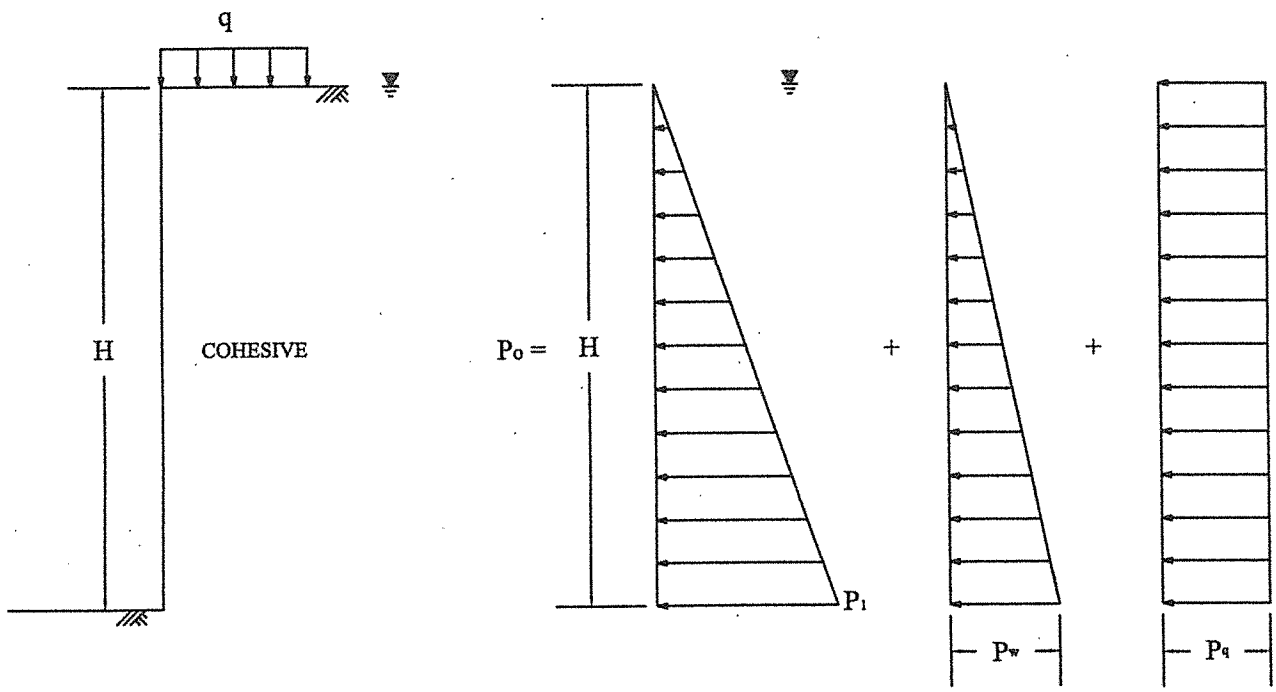
$$\text{If } H_1 > \frac{2}{3}\frac{B_d}{\sqrt{2}}, \quad P_H = 0.7\,(\gamma\, HB_d - 1.4CH - \pi CB_d) \text{ in lbs/ linear foot}$$

$$\text{If } H_1 < \frac{2}{3}\frac{B_d}{\sqrt{2}}, \quad P_H = 1.5H_1\left(\gamma\, H - \frac{1.4CH}{B_d} - \pi C\right) \text{ in lbs/ linear foot}$$



STABILITY OF BOTTOM
FOR
BRACED CUT

Job No. 1140203801



TYPICAL SOIL PARAMETERS

See Table 1 for typical values of soil parameters

$K_{oc} = 1.0$

PERMANENT WALL

$P_1 = K_{oc}\,\gamma_c'\,H$
$P_w = \gamma_w\,H = 62.4\,H$
$P_q = 0.5\,q$

Where:

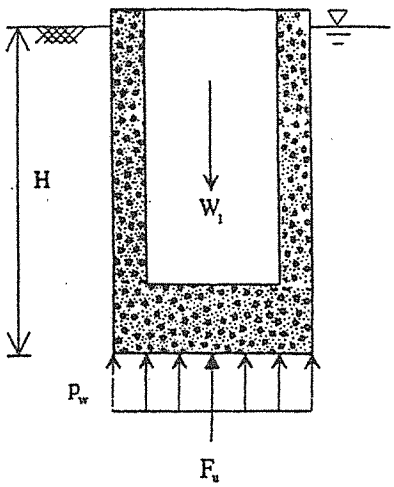
γ_c' = Submerged unit weight of cohesive soil, pcf;
K_{oc} = Coefficient of at-rest earth pressure in cohesive soil;
γ_w = Unit weight of water, pcf;
q = Surcharge load at surface, psf;
P_o = Lateral pressure, psf;
P_1 = At-rest earth pressure, psf;
P_q = Horizontal pressure due to surcharge, psf;
P_w = Hydrostatic pressure due to groundwater, psf;
H = Depth of excavation, feet

LATERAL EARTH PRESSURE DIAGRAM FOR PERMANENT WALL

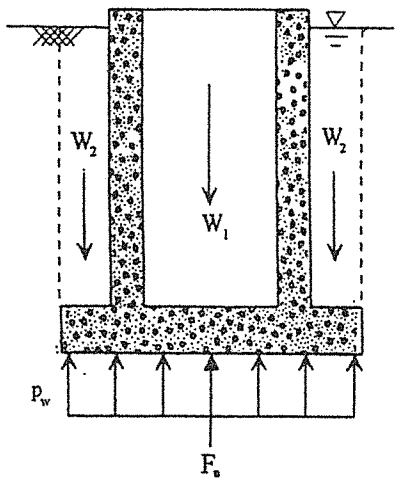
SUBMERGED COHESIVE SOIL

Job No. 1140203801

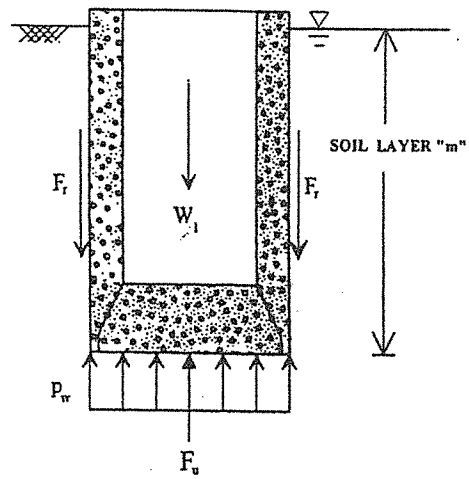
(a) DEAD WEIGHT OF STRUCTURE



(b) WEIGHT OF SOIL ABOVE BASE EXTENSION PLUS DEAD WEIGHT OF STRUCTURE



(c) SOIL-WALL FRICTION PLUS DEAD WEIGHT OF STRUCTURE



$p_w = H\gamma_w$

$F_u = A_b p_w$

$\frac{W_1}{Sf_1} = F_u$

See Table 1 for typical values of soil parameters

$p_w = H\gamma_w$

$F_u = A_b p_w$

$\frac{W_1}{Sf_1} + \frac{W_2}{Sf_2} = F_u$

$p_w = H\gamma_w$

$F_u = A_b p_w$

$\frac{W_1}{Sf_1} + \frac{F_r}{Sf_3} = F_u$

Predominantly Cohesive Soils, $F_r = \alpha c_m A_m$

Predominantly Cohesionless Soils, $F_r = p_m A_m K \tan \delta_m$

Where:

A_b	=	area of base, sq. ft.
A_m	=	cylindrical surface area of layer "m", sq. ft.
c_m	=	undrained cohesion of soil layer "m", psf.
F_u	=	hydrostatic uplift force, lbs.
F_r	=	frictional resistance, lbs.
H	=	height of buried structure, ft.
K	=	coefficient of lateral pressure = 0.5.
p_m	=	average overburden pressure for layer "m," psf.
p_w	=	hydrostatic uplift pressure, psf.
$Sf_{1,2,3}$	=	factor of safety.
W_1	=	dead weight of concrete structure, lbs.
W_2	=	weight of backfill above base extension, lbs.
α	=	cohesion reduction factor = 0.5.
δ_m	=	friction angle between soil layer "m" and concrete wall, degrees = 0.75 ϕ_m
ϕ_m	=	internal angle of friction of soil layer "m", degrees.
γ_w	=	unit weight of water = 62.4 pcf.

UPLIFT PRESSURE AND RESISTANCE

APPENDIX A

	Figure
Log of Borings	A-1 and A-2
Symbols and Terms Used on Boring Logs	A-3

LOG OF BORING NO. B-1

PROJECT : FY2013 Storm Water Pump Station and Flood Warning System Improvements WBS No. M-430241-0009-3
Houston, Texas

PROJECT NO. : 1140203801

LOCATION : N 13862773.21, E 3154045.98
See Plan of Borings (Figure 2)

COMPLETION DEPTH : 25.0 FT.

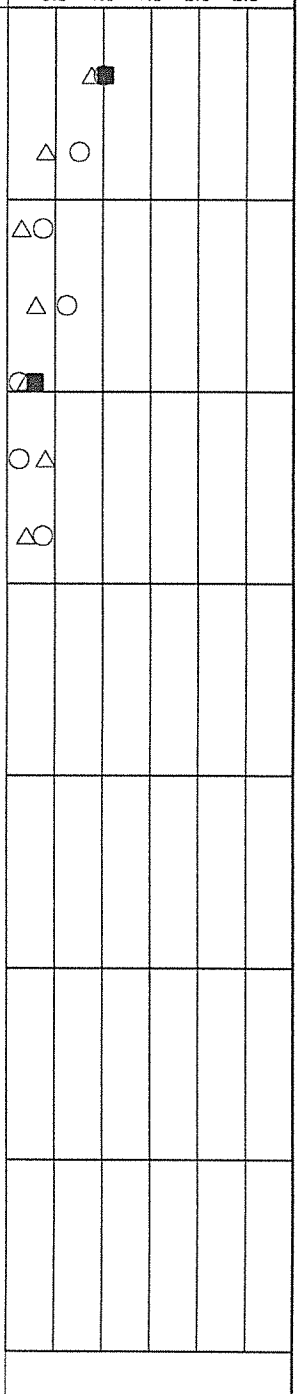
SURFACE ELEVATION : 36.00 FT.

DATE : 07-09-14

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 25.0 FT.
WET ROTARY : -- TO -- FT.

UNDRAINED SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
■ UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION
△ TORVANE

ELEVATION, FEET	DEPTH, FEET	DESCRIPTION OF MATERIAL	STANDARD PENETRATION TEST, BLOWS PER FOOT	PERCENT PASSING NO. 200 SIEVE	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %
36.0	0	FILL: stiff to very stiff brown sandy lean clay w/grass roots and calcareous nodules			120	15	31	14	17
		-medium stiff to stiff 2'-4'							
		-soft to medium stiff 4'-6'		52		16	29	15	14
	5								
		-medium stiff to stiff 6'-8'				15			
		-soft to medium stiff 8'-12'				17			
	10	-reddish brown and gray 10'-14'		54	115	17	26	16	10
		-medium stiff 12'-14'				20			
22.0						21			
	15	Medium dense brown and gray FINE SAND (SP-SM) w/silt	14			9			
			15	5		14			
	20		13			25			
		-w/clay seams 23'-25'							
11.0	25		22			33			
	30								
	35								



DEPTH TO WATER IN BORING :
NO GROUNDWATER ENCOUNTERED DURING DRILLING.
HOLE OPEN TO 25.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-1

LOG OF BORING NO. B-2

PROJECT : FY2013 Storm Water Pump Station and Flood Warning System Improvements WBS No. M-430241-0009-3
Houston, Texas

PROJECT NO. : 1140203801

LOCATION : N 13862970.92, E 3153911.88
See Plan of Borings (Figure 2)

COMPLETION DEPTH : 10.0 FT.

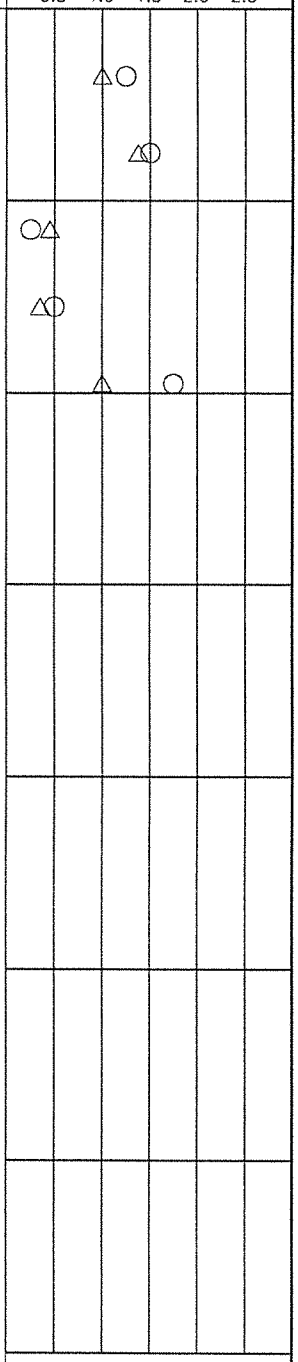
SURFACE ELEVATION : 35.50 FT.

DATE : 07-09-14

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 10.0 FT.
WET ROTARY : -- TO -- FT.

ELEVATION, FEET	DEPTH, FEET	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	STANDARD PENETRATION TEST, BLOWS PER FOOT	PERCENT PASSING NO. 200 SIEVE	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %
35.5	0			1" Asphalt							
34.1				16" Lime Stabilized Gray Sand and Shell Mix		66		17	37	17	20
				FILL: very stiff brown and gray very sandy lean clay				14			
	5			-medium stiff 4'-6'							
				-medium stiff to stiff yellowish brown and gray 6'-8'		60		22	35	17	18
				-gray and brown 8'-10'				20			
25.5	10							16			
	15										
	20										
	25										
	30										
	35										

UNDRAINED SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
■ UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION
△ TORVANE



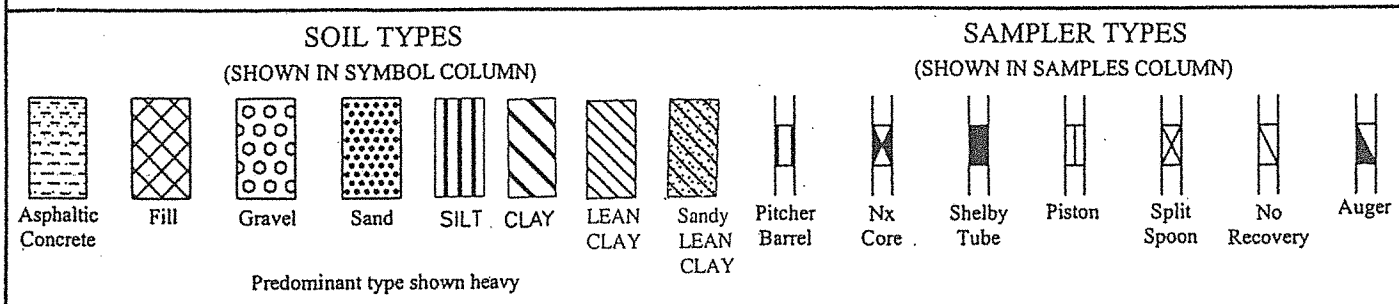
DEPTH TO WATER IN BORING :
NO GROUNDWATER ENCOUNTERED DURING DRILLING.
HOLE OPEN TO 10.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-2

Job No. 1140203801

SYMBOLS AND TERMS USED ON BORING LOGS



TERMS DESCRIBING CONSISTENCY OR CONDITION

Basic Soil Type	Density or Consistency	Standard Penetration Resistance, (1) Blows/ft.	Unconfined Compressive Strength (q_u), (2) Tons/sq. ft.
Cohesionless	Very loose	Less than 4	Not applicable
	Loose	4 to <10	Not applicable
	Medium dense	10 to <30	Not applicable
	Dense	30 to <50	Not applicable
	Very dense	50 or greater	Not applicable
Cohesive	Very soft	Less than 2	Less than 0.25
	Soft	2 to <4	0.25 to <0.5
	Firm/Medium stiff	4 to <8	0.5 to <1.0
	Stiff	8 to <15	1.0 to <2.0
	Very stiff	15 to <30	2.0 to <4.0
	Hard	30 or greater	4 or greater

(1) Number of blows from 140-lb. weight falling 30-in. to drive 2-in. OD, 1-3/8-in. ID, split barrel sampler (ASTM D1586)

(2) q_u may also be approximated using a pocket penetrometer

TERMS CHARACTERIZING SOIL STRUCTURE

Parting: -paper thin in size Seam: -1/8" to 3" thick Layer: -greater than 3"

Slickensided	- having inclined planes of weakness that are slick and glossy in appearance.
Fissured	- containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated	- composed of thin layers of varying color and texture.
Interbedded	- composed of alternate layers of different soil types.
Calcareous	- containing appreciable quantities of calcium carbonate.
Well graded	- having wide range in grain sizes and substantial amounts of all intermediate particle sizes.
Poorly graded	- predominantly of one grain size, or having a range of sizes with some intermediate size missing.
Flocculated	- pertaining to cohesive soils that exhibit a loose knit or flakey structure.

APPENDIX B

	Figure
Summary of Laboratory Test Results	B-1 and B-2
Grain Size Distribution Curves	B-3

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: FY2013 Storm Water Pump Station and Flood Warning System Improvements WBS No. M-430241-0009-3 Houston, Texas

PROJECT NUMBER: 1140203801

BORING NO.	SAMPLE No.	SAMPLE Depth (ft.) Top	SAMPLE Depth (ft.) Bottom	SAMPLE Type	SPT (blows/ft.)	WATER CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	PASSING NO. 200 SIEVE (%)	UNCONFINED COMPRESSION TEST Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Conf. Press. (tsf)	TORVANE Shear Strength (tsf)	POCKET PENE-TROMETER Shear Strength (tsf)	TYPE OF MATERIAL
B-1	1	0.0	2.0	UD		15	120	31	14	17			1.02	0.14	0.88	1.00	Fill
	2	2.0	4.0	UD		16		29	15	14	52				0.40	0.75	Fill
	3	4.0	6.0	UD		15									0.15	0.38	Fill
	4	6.0	8.0	UD		17									0.30	0.63	Fill
	5	8.0	10.0	UD		17	115	26	16	10	54		0.29	0.72	0.20	0.13	Fill
	6	10.0	12.0	UD		20									0.40	0.13	Fill
	7	12.0	14.0	UD		21									0.20	0.38	Fill
	8	14.5	16.0	SS	14	9											Sand
	9	16.5	18.0	SS	15	14					5						Sand
	10	18.5	20.0	SS	13	25											Sand
	11	23.5	25.0	SS	22	33											Sand

LEGEND:
UD = UNDISTURBED SAMPLE, EXTRUDED IN FIELD
SS = SPLIT SPOON SAMPLE
AG = AUGER CUTTINGS
PB = PITCHER BARREL SAMPLE
Nx = Nx-DOUBBLE BARREL SAMPLE

SPT = Standard Penetration Test
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index

FIGURE B-1

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: FY2013 Storm Water Pump Station and Flood Warning System Improvements WBS No. M-430241-0009-3 Houston, Texas

PROJECT NUMBER: 1140203801

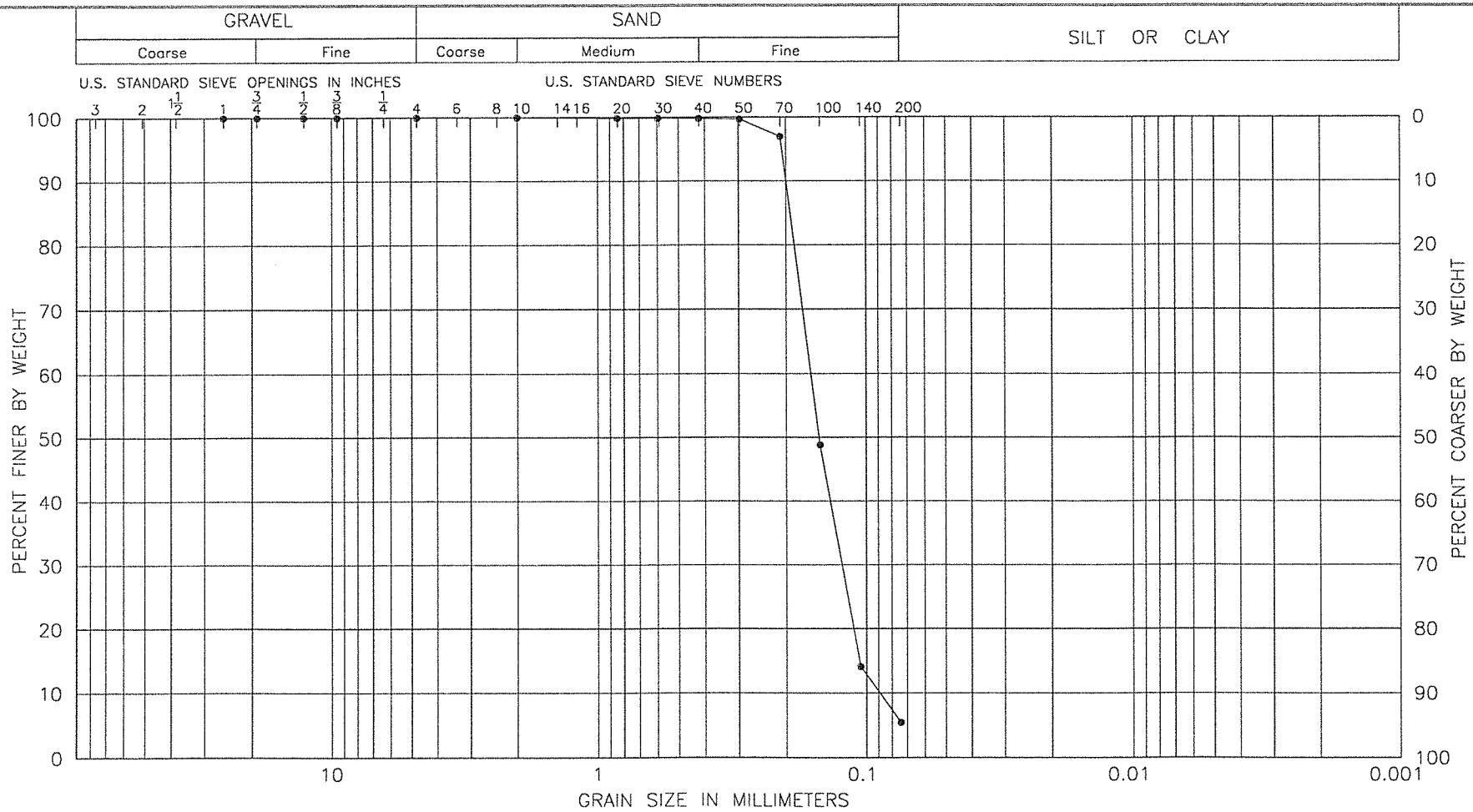
BORING NO.	SAMPLE No.	SAMPLE Depth (ft.) Top	SAMPLE Depth (ft.) Bottom	SAMPLE Type	SPT (blows/ft.)	WATER CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	PASSING NO. 200 SIEVE (%)	UNCONFINED COMPRESSION TEST Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Conf. Press. (tsf)	TORVANE Shear Strength (tsf)	POCKET PENE-TROMETER Shear Strength (tsf)	TYPE OF MATERIAL
B-2	1	0.0	2.0	UD		17		37	17	20	66				1.00	1.25	Fill
	2	2.0	4.0	UD		14									1.38	1.50	Fill
	3	4.0	6.0	UD		22		35	17	18	60				0.45	0.25	Fill
	4	6.0	8.0	UD		20									0.35	0.50	Fill
	5	8.0	10.0	UD		16									1.00	1.75	Fill

LEGEND:
UD = UNDISTURBED SAMPLE, EXTRUDED IN FIELD
SS = SPLIT SPOON SAMPLE
AG = AUGER CUTTINGS
PB = PITCHER BARREL SAMPLE
Nx = Nx-DOUBBLE BARREL SAMPLE

SPT = Standard Penetration Test
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index

FIGURE B-2

Job No. 1140203801



LEGEND	BORING NO.	SAMPLE NO.	DEPTH FT.	SAMPLE DESCRIPTION
●●●●●	B-1	S-9	16.5-18.0	Brown and gray fine sand w/silt

GRAIN SIZE DISTRIBUTION CURVES

Geotest Engineering, Inc.

FIGURE B-3