GEOTECHNICAL INVESTIGATION
CLINTON DRIVE RECONSTRUCTION
PROJECT SEGMENT-1
WBS NO. N-000804-0001-3
HOUSTON, TEXAS

REPORT NO. 1140187001

Reported to:

TRANSYSTEMS

Houston, Texas

Submitted by:

GEOTEST ENGINEERING, INC.

Houston, Texas

January 4, 2013

Key Map No. 494 K & L

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Report No. 1140187001
January 4, 2013

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Reference: Geotechnical Investigation
Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000801-0001-3
Houston, Texas

Dear Mr. Gunkle:

Presented herein is our final geotechnical investigation report for the Clinton Drive Reconstruction Project – Segment 1. The recommended pavement sections were submitted to you on July 20, 2012. Boring logs were forwarded to you on October 26, 2012. A draft report was submitted to you on December 4, 2012. This final report supercedes the previously submitted reports, transmittals, e-mails, etc. for the referenced project. This study was authorized by Subcontract Agreement Between Transystems and Geotest Engineering, Inc. dated March 21, 2012 and by your email authorization on May 10, 2012.

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,
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EXECUTIVE SUMMARY

A geotechnical investigation was performed for the design and construction of the proposed Clinton Drive Reconstruction Project – Segment 1 in Houston, Texas. The project is comprised of approximately 3,600 linier feet of Clinton Drive Reconstruction with utility improvements from Hirsch to Lockwood. The utilities include storm sewer and a water line. The depth of utilities ranges from 8 to 20 feet. The existing Clinton Drive is an asphalt overlay over concrete pavement. The new pavement will consists of concrete pavement section with curb and gutter. The project also includes reconstruction of Hirsch Street approximately 190 feet north over the existing Union Pacific Railroad (UPRR) track and Clinton Drive approximately 615 feet west of Hirsch Street to Ingraham Gully. It is understood that proposed roadway improvements along Hirsch Street north of Clinton Drive will not reconstruct the existing UPRR crossing. The proposed pavement will stop on both sides of crossing and use asphalt pavement to transition to match the existing asphalt pavement crossing the UPRR tracks. The proposed utilities will be constructed by open cut method of construction along Clinton Drive except near Hirsch Street where it crosses the existing 8-inch water line will be installed by trenchless method of construction.

The investigation included drilling and sampling ten (10) soil borings to depths ranging from 13 to 42 feet, installing piezometers in two (2) existing borings, 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 the Clinton Drive Reconstruction Project – Segment 1 investigation is as follows:

- The subsurface soil beneath pavement as encountered in borings GB-1 through GB-10 along Clinton Drive consists of predominantly cohesive soils to the explored depths of 13 to 42 feet except in borings GB-1, GB-3A, GB-4B, GB-5 and GB-7. In borings GB-1, GB-4B, GB-5 and GB-7 the subsurface soil beneath pavement consists of cohesive with intermittent or underlain by cohesionless soil to the explored boring depths of 29 to 42

feet. In boring GB-3A, cohesionless soils were encountered below pavement to the depth of 13 feet, the explored depth of boring. The cohesive soils consist of medium stiff to hard dark gray, gray, brown, yellowish brown and reddish brown sandy lean clay, lean clay with sand, lean clay, silty clay and fat clay. The cohesionless soil consists of loose to medium dense brown and gray fine sand w/silt and silty sand.

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

- Groundwater was encountered in borings GB-1, GB-4B, GB-5, GB-6, and GB-7 to depths ranging from 12.0 to 25.0 feet during drilling. The groundwater level, measured 10 to 15 minutes after water was first encountered, ranged from 8.9 to 20.7 feet in these borings. No groundwater was encountered in other borings GB-2, GB-3A, GB-8, GB-9 and GB-10 drilled for this study. In piezometer borings GB-1P and GB-16P, the water level measured ranges from 13.1 to 14.6 feet on August 2, 2012.

- The existing paving as obtained in the soil borings GB-1 through GB-10 consists of 0 to 7.5 inches of asphalt underlain by 6 to 10.5 inches of Concrete over 0 to 9 inches of cement stabilized sand except in boring GB-3. In boring GB-3 the existing pavement consists of 3 inches of asphalt over 11.5 inches of limestone.

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

- In general, excavation and backfill for utilities should be designed and constructed in accordance with City of Houston Standard Specification No. 02317.

- The bedding and backfill for storm sewer should be in accordance with City of Houston Standard Specification Section 02317 and Drawing No. 02317-03.

- The recommendations for trenchless installation are provided in Section 5.3 of this report.

- The mat foundation for supporting the new manholes placed at depths ranging from 8 to 20 feet, should be designed for an allowable (net) bearing pressure of 2.0. These allowable bearing pressures include a safety factor of 4.0. The details are provided in Section 5.4 of this report.

- The recommended pavement sections for Clinton Drive are given below:

Rigid Pavement

Street	Limits	Concrete Pavement Thickness (inches)	Lime Stabilized Subgrade (inches)	Lime Flyash Stabilized Subgrade at Boring GB-3A, GB-4 and GB-4A
Clinton Drive	Hirsch to Lockwood	10	8	8
Clinton Drive	Lockwood to Harvey Wilson Drive	9	8	--

The details of pavement section are provided in Section 5.5 of this report.

Flexible Pavement Transition

Street	Asphaltic Cement Concrete (inch)	Asphaltic Treated Aggregate Base (Black Base) (inch)	Lime Stabilized Subgrade (inch)
Hirsch Road at UPRR Crossing	3	6	8

1.0 INTRODUCTION

1.1 General

The City of Houston selected Transystems to perform engineering services for design and construction of Clinton Drive Reconstruction Project – Segment 1 in Houston, Texas. Transystems 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 Subcontract Agreement Between Transystems and Geotest Engineering, Inc. dated March 21, 2012 and by your email authorization on May 10, 2012.

1.3 Location and Description of Project

The project is comprised of approximately 3,600 linier feet of Clinton Drive Reconstruction with utility improvements from Hirsch to Lockwood. The utilities include storm sewer and a water line. The depth of utilities ranges from 8 to 20 feet. The existing Clinton Drive is an asphalt overlay over concrete pavement. The new pavement will consists of concrete pavement section with curb and gutter. The project also includes reconstruction of Hirsch Street approximately 190 feet north over the existing Union Pacific Railroad (UPRR) track and Clinton Drive approximately 615 feet west of Hirsch Street to Ingraham Gully. It is understood that proposed roadway improvements along Hirsch Street north of Clinton Drive will not reconstruct the existing UPRR crossing. The proposed pavement will stop on both sides of crossing and use asphalt pavement to transition to match the existing asphalt pavement crossing the UPRR tracks. The proposed utilities will be constructed by open cut method of construction along Clinton Drive except near Hirsch Street where it crosses the existing 8-inch water line will be installed by trenchless method of construction. The vicinity map of the project area 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 design and construction of the proposed paving reconstruction with utility improvements. The scope of this investigation for Clinton Drive Reconstruction Project consisted of the following:

- Concrete coring at all ten (10) boring locations for borings access.
- Drilling and sampling ten (10) borings each to depths ranging from 13 to 42 feet.
- Converting two (2) borings into piezometers to monitor long term ground water level.
- Performing appropriate laboratory tests in accordance with ASTM methods on selected samples to develop engineering properties of the soil.
- Reviewing available fault information to evaluate the potential for known active faults that may impact the project.
- Performing engineering analyses in accordance with the City of Houston Design Manual (July 2012) to develop geotechnical recommendations for Clinton Drive Reconstruction –Segment 1 with utilities improvements.
- Preparing a geotechnical report that will include all field data, laboratory test data and geotechnical recommendations.
- Preparing a separate soil type report for trench (open cut) excavation.

2.0 FIELD INVESTIGATION

2.1 General

After obtaining the utilities clearance of proposed ten (10) marked borings in the field, existing concrete pavement was cored at all boring locations for boring access and ten (10) borings were drilled to the explored depths utilizing a truck mounted drilling rig. Traffic control devices and personnel were utilized during coring and drilling to maintain safety of drill crew and people driving in the streets. All the drilling and sampling were performed in accordance with appropriate ASTM procedures.

2.2 Geotechnical Borings

Subsurface conditions for Clinton Drive Reconstruction project area were explored by drilling and sampling ten (10) soil borings (designated as GB-1 through GB-10) each to depths ranging from 13 to 42 feet. The borings GB-3 and GB-4 were offset due to encounter of hard obstructions at 6 to 7 feet. The approximate boring locations are shown on Figures 2.1 through 2.7, Plan of Borings. Survey information (Northing and Easting coordinates and ground surface elevation) of completed borings was provided to us by Transystems. The survey information of completed borings is summarized in Table 1.

The existing concrete pavement was cored to provide access to all ten (10) borings. In general, samples were obtained continuously to depths of 13 to 20 feet, and intermittent sampling at 5 foot intervals to the termination depths of all borings. Cohesive soils were obtained with a 3-inch thin-walled tube sampler in general accordance with ASTM Method D 1587 and samples of granular soils were obtained with a 2-inch diameter split-barrel sampler in general accordance with ASTM Method D 1586. 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 sampler were recorded as "Blows per Foot" on the boring logs. All the borings, except the ones

converted to piezometers, 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 through A-10 in Appendix A. A key to symbols and terms used on boring logs is given on Figure A-11 in Appendix A.

2.3 Piezometer Installation

During the field investigation, a piezometer was installed in the open borehole of borings GB-1 and GB-6. The location of the piezometers, designated as GB-1P and GB-6P, are shown on Figures 2.1 and 2.5. The piezometer installation report showing the details of the construction of the piezometers are provided on Figures A-12 and A-13 in Appendix A.

After taking the 30-day water level, the piezometers were abandoned in-place. The piezometer abandonment reports are presented in Appendix C.

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 unconfined compression (ASTM D 2166) tests and unconsolidated undrained (UU) triaxial compression tests (ASTM D 2850). The results of the unconfined compression tests and UU triaxial compression tests are plotted on the boring logs as solid circles and solid squares, respectively. 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 unconfined compression test and 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 sieve (ASTM D 1140) were performed on appropriate samples. Sieve analysis (ASTM D 422) was also performed on selected cohesionless soil samples.

The result of all tests are tabulated or summarized on the boring logs presented on Figures A-1 through A-10 in Appendix A. The summary of laboratory tests is also presented in a tabular form on Figures B-1 through B-10 in Appendix B. The grain size distribution curves are presented on Figures B-11 and B-12 in Appendix B.

A California Bearing Ratio (CBR) test was performed on the bulk sample from borings GB-2, GB-6, GB-7, GB-8, GB-9 and GB-10 between the depths of 0 to 6 feet. The results of the CBR test are presented on Figures B-13a through B-13c in Appendix B. The relationship between dry density versus CBR is presented on Figure B-13d 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 area, the nearest fault is the Clinton Fault and it is located approximately 2 miles east of the project area. Hence, a Phase I Geological Fault Study is not required for this project.

4.3 Existing Paving

The existing paving as obtained in the soil borings GB-1 through GB-10 consists of 0 to 7.5 inches of asphalt underlain by 6 to 10.5 inches of Concrete over 0 to 9 inches of cement stabilized sand except in boring GB-3. In boring GB-3 the existing pavement consists of 3 inches of asphalt over 11.5 inches of limestone.

The details of the existing pavement thickness at each of the boring locations for Clinton Drive are summarized below:

Boring Nos.	**Asphalt (in.)**	**Concrete Thickness (in.)**	**Subbase Thickness (in.)**	**Total (in.)**
GB-1 (GB-1P)	7.5	7.5	--	15.0
GB-2	4.0	7.5	--	11.5
GB-3A	3.0	--	11.5	14.5
GB-4B	2.5	7.5	--	10.0
GB-5	--	7.5	7.0	14.5
GB-6 (GB-6P)	3.0	6.5	--	9.5
GB-7	4.5	6.0	--	10.5
GB-8	4.0	6.0	--	10.0
GB-9	2.25	10.5	9.0	21.75
GB-10	--	7.0	--	7.0

Note: The subbase includes Cement Stabilized Sand and Limestone.

4.4 Soils Stratigraphy

Based on the subsurface soils encountered in the boreholes, two (2) boring log profiles were developed and are presented on Figures 3.1 and 3.2. To the left of each boring shown on the profile is an indication of the consistency of each stratum. More than one consistency for an individual stratum indicates that the consistency 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 relative 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 as encountered in borings GB-1 through GB-10 along Clinton Drive consists of predominantly cohesive soils to the explored depths of 13 to 42 feet except in borings GB-1, GB-3A, GB-4B, GB-5 and GB-7. In borings GB-1, GB-4B, GB-5 and GB-7 the subsurface soil beneath pavement consists of cohesive with intermittent or underlain by cohesionless soil to the explored boring depths of 29 to 42 feet. In boring GB-3A, cohesionless soils were encountered below pavement to the depth of 13 feet, the explored depth of boring. The cohesive soils consist of medium stiff to hard dark gray, gray, brown, yellowish brown and reddish brown sandy lean

clay, lean clay with sand, lean clay, silty clay and fat clay. The cohesionless soil consists of loose to medium dense brown and gray fine sand w/silt and silty sand.

The Fat Clay and Fat Clay with sand is of high to very high plasticity with a liquid limits ranging from 53 to 87 and plasticity indices ranging from 31 to 56. The Sandy Lean Clay, Lean Clay w/sand and Silty Clay is of slight to medium plasticity with a liquid limits ranging from 20 to 37 and plasticity indices ranging from 6 to 19. The percent fines (percent passing No. 200 sieve) of Fat Clay with sand and Fat Clay ranges from 81 to 99 percent. The percent fines of Lean Clay and Lean Clay with sand ranges from 73 to 99 percent and the percent fines of Sandy Lean Clay and Silty Clay ranges from54 to 69 percent. The percent fines of Fine Sand w/silt is about 9 percent. The percent fines of Silty Sand ranges from 13 to 20 percent. The percent fines of Sandy Silt is about 61 percent.

4.5 Water Levels

Groundwater was encountered in borings GB-1, GB-4B, GB-5, GB-6, and GB-7 to depths ranging from 12.0 to 25.0 feet during drilling. The groundwater level, measured 10 to 15 minutes after water was first encountered, ranged from 8.9 to 20.7 feet in these borings. No groundwater was encountered in other borings GB-2, GB-3A, GB-8, GB-9 and GB-10 drilled for this study. In piezometer borings GB-1P and GB-16P, the water level measured ranges from 13.1 to 14.6 feet on August 2, 2012. The water level encountered in borings is summarized below.

Boring No.	Location/Street Name	Groundwater Depth During Drilling (ft)	Groundwater Depth 30 Days After Drilling (ft)
GB-1 (GB-1P)	**Clinton Drive**	**15.3**	**14.6**
GB-2	Clinton Drive	Dry	N/A
GB-3A	Clinton Drive	Dry	N/A
GB-4B	Clinton Drive	10.0	N/A
GB-5	Clinton Drive	8.9	N/A
GB-6 (GB-6P)	**Clinton Drive**	**20.7**	**13.1**
GB-7	Clinton Drive	13.8	N/A
GB-8	Clinton Drive	Dry	N/A
GB-9	Clinton Drive	Dry	N/A
GB-10	Hirsch Road	Dry	N/A

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.

4.6 Environmental Concerns

No environmental concerns were observed or noticed in any of the borings (GB-1 through GB-10) drilled for this study.

5.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

5.1 General

The project is comprised of approximately 3,600 linier feet of Clinton Drive Reconstruction with utility improvements from Hirsch to Lockwood. The utilities include storm sewer and a water line. The depth of utilities ranges from 8 to 20 feet. The existing Clinton Drive is an asphalt overlay over concrete pavement. The new pavement will consists of concrete pavement section with curb and gutter. The project also includes reconstruction of Hirsch Street approximately 190 feet north over the existing Union Pacific Railroad (UPRR) track and Clinton Drive approximately 615 feet west of Hirsch Street to Ingraham Gully. It is understood that proposed roadway improvements along Hirsch Street north of Clinton Drive will not reconstruct the existing UPRR crossing. The proposed pavement will stop on both sides of crossing and use asphalt pavement to transition to match the existing asphalt pavement crossing the UPRR tracks. The proposed utilities will be constructed by open cut method of construction along Clinton Drive except near Hirsch Street where it crosses the existing 8-inch water line will be installed by trenchless method of construction.

5.2 Trench Excavation

Based on the information provided by Transystems, it is understood that the storm sewer and water line will be by installed by open cut method of construction except on Hirsch Street at existing UPRR track crossing where it will be trenchless (bore and jack) method of construction. The following subsections provide information for the design and construction of the utilities by open cut method of excavations including open excavation for access shafts.

5.2.1 Geotechnical Parameters. Based on the soil conditions revealed by the borings GB-1 through GB-10, geotechnical parameters were developed for the design of open cut construction for utilities installation. The design parameters are provided in Table 2. 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 laid back to a stable slope 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 laid back to a stable slope or supported by some other equivalent means or protection such that workers are not exposed to moving ground or cave-ins. The slopes and 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 diagrams were developed and are presented on Figures 5.1 through 5.3. These pressure diagrams 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 91 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 predominantly encountered for the proposed excavation depths in most of the borings, the bottom stability can be evaluated as outlined on Figure 6. However, at locations near boring GB-3A, GB-4B and GB-5 where cohesionless soils (such as fine sand w/silt and silty sand) were encountered at the invert or within 3 feet of bottom of excavation, dewatering will be necessary to avoid bottom stability problems.

5.2.3 Groundwater Control. Excavations for the utilities 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 borings for the proposed water line and storm sewer installation, all the excavations will be in cohesive soils except at borings GB-3A, GB-4B and GB-5 where water line and storm sewer will be in cohesionless soils.

In general for cohesive soils as predominantly encountered for most of the borings for the excavation depths, the groundwater if encountered may be managed by collection in excavation bottom sumps for pumped disposal. However, in borings GB-3A, GB-4B and GB-5 where cohesionless soils were encountered at the invert or within 3 feet of bottom of the excavation; dewatering will be required. Dewatering such as vacuum well points up to 15 feet or deep wells with submersible pumps for excavation greater than 15 feet may be required to lower the groundwater level to at least 5 feet below the bottom of the excavation. It is recommended that the actual groundwater conditions should be verified by the contractor at the time of construction and that groundwater control should be performed in general accordance with the City of Houston Standard Specifications, Section 01578.

5.2.4 Bedding and Backfill for Utilities. In general, excavation and backfill for utilities should be designed and constructed in accordance with the City of Houston Standard Specification No. 02317, Subsection 3.09 and 3.10 "Excavation and Backfill for Utilities."

Bedding and backfill for storm sewer should be in accordance with City of Houston Standard Specification Section 02317 and Drawing No. 02317-02 and 02317-03.

The bedding and backfill for water line should be in accordance with City of Houston Standard Specification Section 02317 and Drawing No. 02317-04.

5.2.5 Live Loads on Pipe Due to Traffic. Loads on pipe due to traffic should be considered. A graph providing calculated vertical stress on pipe due to traffic loads is given on Figure 7.

5.3 Trenchless Installation

The storm sewer along Clinton Drive near the Hirsch Street where it crosses the existing 8-inch water line will be installed by trenchless method of construction.

5.3.1 Geotechnical Parameters for Trenchless Installation. Based on the soil conditions revealed by borings GB-1, GB-2 and GB-10 and laboratory test data, geotechnical design parameters were developed for cohesive soils and cohesionless soils and are provided in Table 3. The cohesive soils include sandy lean clay, silty clays and fat clays.

5.3.2 Earth Pressure on Tunnel Liner. The earth pressures on the tunnel liner should be determined from Figure 8. Equations to calculate the tunnel liner loads are also shown in Figure 8. For pipe crossing under the major roads, the stress due to traffic loads should be considered. The relationship between the depth of pipe and the vertical stress on the pipe due to traffic live loads is provided on Figure 7.

5.3.3 Access Shafts. The access shafts may be constructed by retained excavation. Retained excavations generally require less ground surface area than open cut excavation with laid back

slopes. The retention system can consist of driven sheetpile, liner plates, solider pile/lagging, driven planking, or ring beams and timber lagging. The items pertaining to design criteria for retained excavation stability should be in accordance with guidelines as outlined in Section 5.2.2 of this report.

5.3.4 Access Shafts Backfill. The excavated access shafts for pipe jacking should be back filled in accordance with City of Houston Standard Specification Section 02441, "Microtunneling And Pipe-Jacked Tunnels."

5.3.5 Carrier Pipe Design Parameters. Carrier pipe must be sufficiently strong to withstand anticipated long-term ground loads and must not be subject to deterioration by substance either in the ground or in the tunnel. The carrier pipe design should include consideration of not only the loads applied to the pipe but also factors other than soil loading. These factors could include minimum structural code requirements, loading from pipe jacking operations and other construction loads. The drained geotechnical design parameters given in Table 3 should be used in analyzing the soil structure intersection of the carrier pipe.

5.3.6 Influence of Tunneling on Adjacent Structures. Surface and near-surface structures near the tunnel alignment consist primarily of private property, city streets, and public and private utilities.

Ground movement, in terms of loss of ground or ground lost, is commonly associated with soft ground tunneling. If such ground movement is excessive, it may cause damage to the structures, roads and services located above the tunnel. While ground movement cannot be eliminated, it can be controlled within certain limits by the use of proper construction techniques and good quality workmanship. These include, but are not limited to, prevention of excessive ground loss during tunneling with the use of grouting and filling the annular space between the pipe or casing and the surrounding soil.

The selection and execution of tunneling methods that are best suited to anticipated ground conditions along the proposed tunnel are, in fact, the contractor's primary contribution to successful completion of the proposed tunnel. On review of the boring logs, the ground conditions for bore and jacking will be primarily through cohesive soils. In these areas, the ground may be expected to behave as firm to raveling near the invert. Close monitoring of ground movement should be carried out during the trenchless operation.

The proposed rehabilitation are parallel with or cross beneath a number of water, gas, power, telephone, and storm sewer lines. The largest potential problems from utilities may result from:

- Leaking water pipes
- Gas pipe breakage leading to a potential explosion
- Breakage of storm or sanitary sewers

In general, it is the contractor's responsibility to investigate these and other possible third party interactions along the proposed rehabilitation and to accommodate all of these interactions with the use of good construction methods.

5.4 Structures

5.4.1 Description. The structure associated with this project will be new manholes. The new manholes for storm sewer will be placed at depths ranging from 8 to 20 feet.

5.4.2 Foundation Conditions. Based on the soil conditions revealed by the borings GB-1 through GB-10, the manholes bottom will be in medium stiff to very stiff lean clay, fat clay, sandy lean clay except at borings GB-3A, GB-4B, GB-5 borings where storm sewer manholes will be in loose fine sand with silt and/or silty sand.

5.4.3 Foundation Design Recommendations. The following items provide recommendations and design criteria for construction of the new manholes.

- Allowable Bearing Pressures. The mat foundation for supporting the new manholes placed at a depths ranging from 8 to 20 feet [into medium stiff to very stiff fat clay, lean clay and sandy lean clay except at borings GB-3A, GB-4B, GB-5 (loose to medium dense fine sand w/silt and silty sand)] should be designed for an allowable (net) bearing pressure of 2,000 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.

- Bottom Stability. Bottom stability is described earlier in Section 5.2.2 under Excavation Stability.

- Lateral Earth Pressure. The pressure diagram presented on Figures 5.1 through 5.3 can be used for the design of braced excavation. The lateral earth pressure diagram presented on Figures 9.1 through 9.3 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 10. In determining the configuration and dimensions of the structure using one of the approaches presented on Figure 10, 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.

5.4.4 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.

5.4.5 Groundwater Control During Construction. Excavations may encounter groundwater seepage to varying degrees depending upon groundwater conditions at the time of construction and the location and depth of excavation. In cohesive soils, as predominantly encountered in most of the borings, groundwater may be managed by collection in trench bottom sumps for pumped disposal. However, at locations near boring GB-3A, GB-4B and GB-5 where cohesionless soils (such as fine sand w/silt and silty sand) were encountered at the invert or within 3 feet of bottom of excavation, dewatering will be necessary to avoid bottom stability problems.

The contractor should verify the groundwater level at the time of construction and should provide an adequate dewatering system, where required.

5.4.6 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."

5.5 Pavement Structure Design

It is understood that approximately 3,600 linear feet of Clinton Drive from Hirsch to Lockwood will be reconstructed with a rigid pavement. The pavement design presented below was

developed in accordance with "AASHTO Guide for Design of Pavement Structures," 1993 Edition.

5.5.1 Design Parameters

Subgrade Soil Properties. Based on the laboratory test data obtained from the natural subgrade soils, the effective roadbed soil resilient modulus (M_R) is estimated to be about 1,941 psi. Based on an estimated resilient modulus of the 8-inch lime-stabilized subgrade, the effective modulus of subgrade reaction (k) is estimated to be about 49 pci.

Traffic Data. Traffic data is provided by to us by Transystems. The details were given below.

Street	**Limits**	**Design ESAL's**
Clinton Drive	Hirsch to Lockwood	8,307,608
Clinton Drive	Lockwood to Harvey Wilson Drive	6,608,518

Other Design Parameters. Other design parameters used in the development of rigid pavement thickness are given below:

Material Properties of Concrete:

Modulus of Elasticity of Concrete (E_c): 3,256,266 psi

Mean value of Modulus of Rupture of Concrete after 28 days (S'_c): 640 psi (based on compressive strength of 3,500 psi)

Load Transfer coefficient (J): 3.3

Drainage coefficient (C_d): 1.2

Overall Standard Deviation (S_o): 0.35

Reliability Level (R): 95%

Serviceability Index

Initial (P_o): 4.5

Terminal (P_t): 2.25

Reinforcement Variables

Allowable Working Stress (f_s): 45,000 psi (grade 60 steel)

Friction Factor (F): 1.8

5.5.2 Recommended Pavement Section

Based on the design parameters described above and the AASHTO design procedures, the thickness of rigid pavement was determined. The recommended pavement section is given below:

Rigid Pavement

Street	**Limits**	**Concrete Pavement Thickness (inches)**	**Lime Stabilized Subgrade (inches)**	**Lime Flyash Stabilized Subgrade at Boring GB-3A, GB-4 and GB-4A**
Clinton Drive	Hirsch to Lockwood	10	8	8
Clinton Drive	Lockwood to Harvey Wilson Drive	9	8	--

Based on the reinforcement variables and recommended pavement section, the required longitudinal and transverse reinforcing steel (No. 4, Grade 60 Steel) can be determined for 9-inch and 10-inch concrete pavement per Table 1 of City of Houston Drawing No. 02751-01 (Revised July 1, 2009).

Asphalt Pavement Transition

Street	**Asphaltic Cement Concrete (inch)**	**Asphaltic Treated Aggregate Base (Black Base) (inch)**	**Lime Stabilized Subgrade (inch)**
Hirsch Road at UPRR Crossing	3	6	8

5.5.3 Preparation of Pavement Subgrade

Based on the field and laboratory test data, the subgrade soils consists of fat clay of high to very high plasticity except near locations GB-3 and GB-4. These soils have a high volume change potential. Hence, lime stabilization of the clay subgrade will be required to reduce the swell potential of clay subgrade due to volume changes and to accelerate the construction and provide a stable subgrade on which to construct the pavement section. The subgrade soils should be stabilized

with approximately 6 percent lime to a depth of at least 8 inches. This corresponds to approximately 31 pounds of hydrated lime per square yard based upon a soil dry unit weight of 103 pcf. The actual percentage of lime must be confirmed by laboratory tests at the time of construction. However, at locations near borings GB-3A, GB-4 and GB-4A, where the subgrade soils consists of silty sand, the subgrade should be stabilized with 3 percent lime and 9 percent fly ash to a depth of 8 inches, this corresponds to 19 pounds of hydrated lime and 56 pounds of fly ash per square yard based upon soil dry unit weight of 103 pcf.

Subgrade preparation for the proposed pavement after removing the existing pavement should consist of stripping, proof-rolling, and stabilization. The following procedures for subgrade preparation are recommended:

1. Strip the surficial soils to a suitable depth to remove all surficial vegetation and achieve grade. In isolated areas where soft, compressible, or very loose soils are encountered, additional stripping may be required. Stripping should extend to a minimum of 2 feet beyond the edge of the proposed pavement.

2. After stripping, the exposed surface should be proof-rolled with a minimum of 3 passes of a 30-ton pneumatic-tired roller or a partially 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 so these weak materials can be removed and replaced.

3. Lime stabilization of cohesive subgrade (fat clay) should be performed in accordance with City of Houston Standard Specification No. 02336, "Lime-Stabilized Subgrade," Lime fly ash stabilization of cohesionless subgrade (silty sand) should be performed in accordance with City of Houston Standard Specification N. 02337, “ Lime Fly ash Stabilized Subgrade”.

6.0 CONSTRUCTION CONSIDERATIONS

Excavations for the sanitary sewer and storm sewer 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 borings for the proposed utility installation, all the excavations will be in cohesive soils except at boring GB-3A, GB-4B and GB-5 where cohesionless soils were encountered at the invert of the excavation.

In general for cohesive soils as predominantly encountered for most of the borings for the excavation depths, the groundwater if encountered may be managed by collection in excavation bottom sumps for pumped disposal. However, in borings GB-3A, GB-4B and GB-5 where cohesionless soils were encountered at the invert of the excavation; dewatering will be required. Dewatering such as vacuum well points up to 15 feet or deep wells with submersible pumps for excavation greater than 15 feet may be required to lower the groundwater level to at least 5 feet below the bottom of the excavation. It is recommended that the actual groundwater conditions should be verified by the contractor at the time of construction and that groundwater control should be performed in general accordance with the City of Houston Standard Specifications, Section 01578.

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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ILLUSTRATIONS

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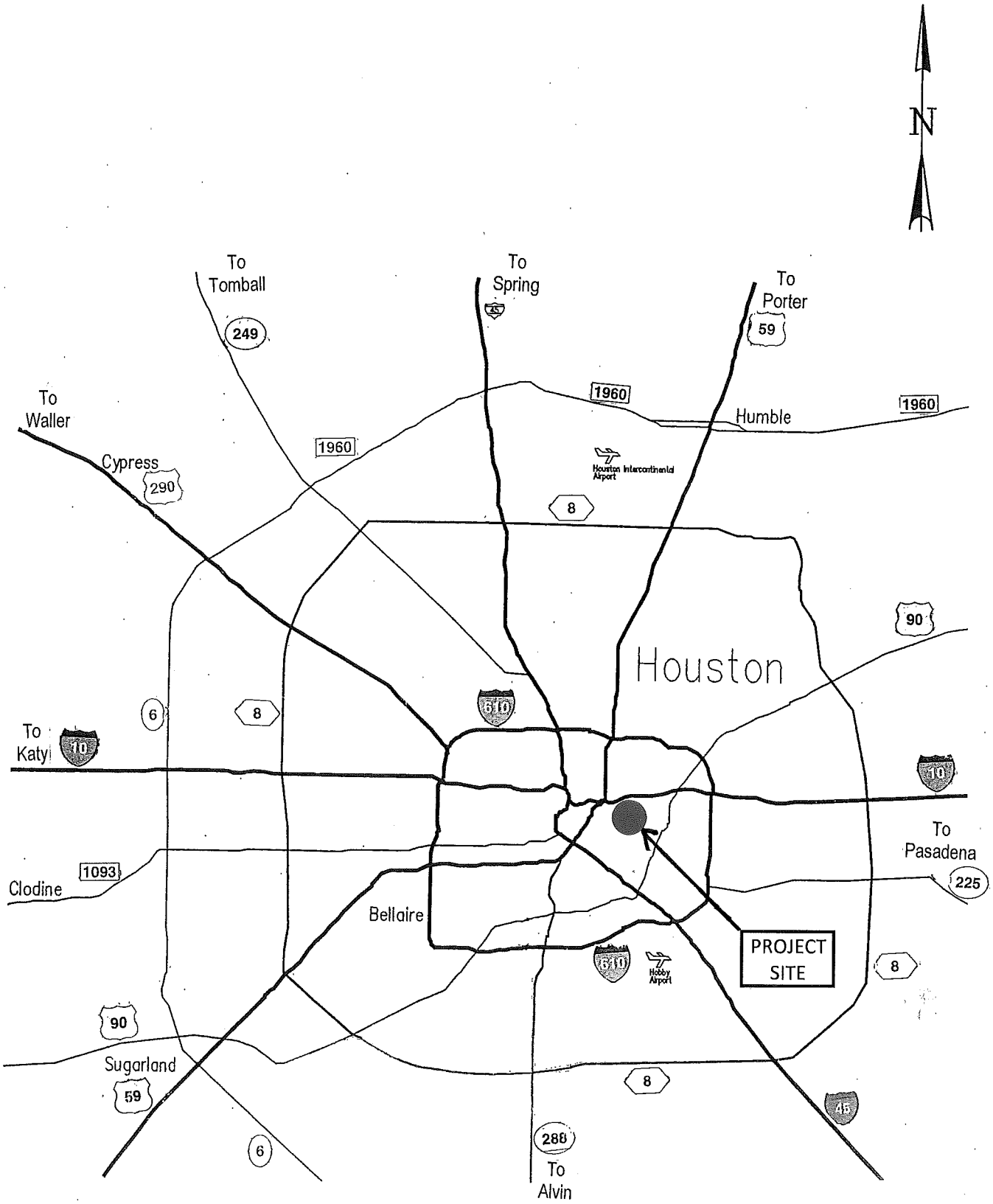
Vertical Stress on Pipes Due to Traffic Loads	7
Tunnel Liners Loads	8
Lateral Earth Pressure Diagram for Permanent Wall	9.1 thru 9.3
Uplift Pressure and Resistance	10

Job No. 1140187001



PROJECT LOCATION MAP
(NOT TO SCALE)

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FIGURE 1

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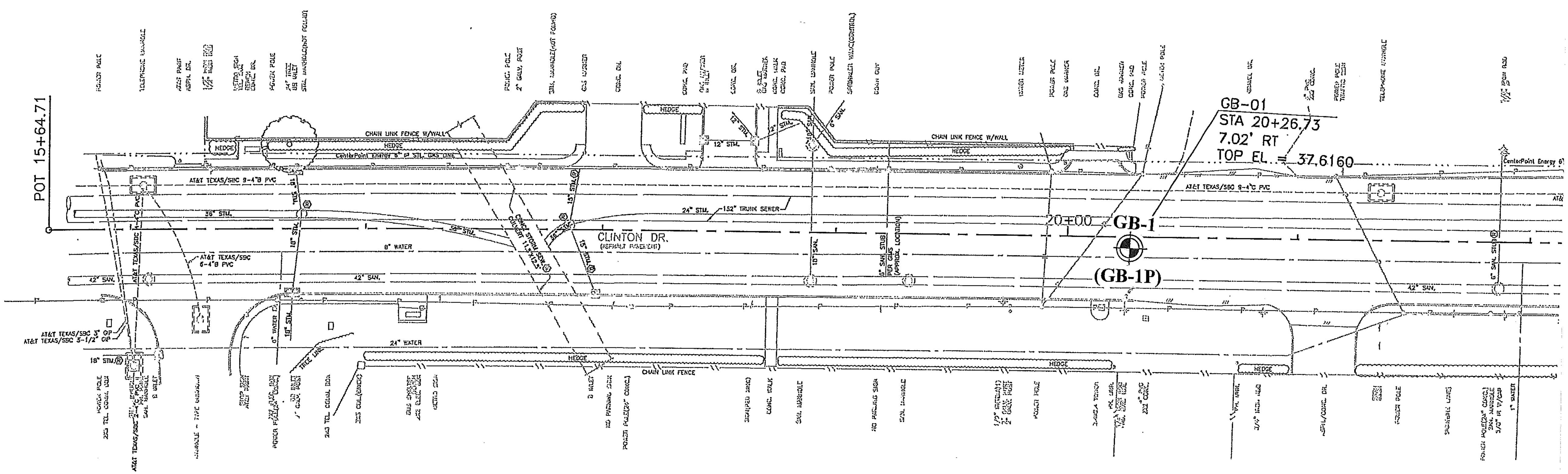


Legend	Geotest Engineering, Inc	1140187001
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

FIGURE 2.1

Job No.: 1140187001

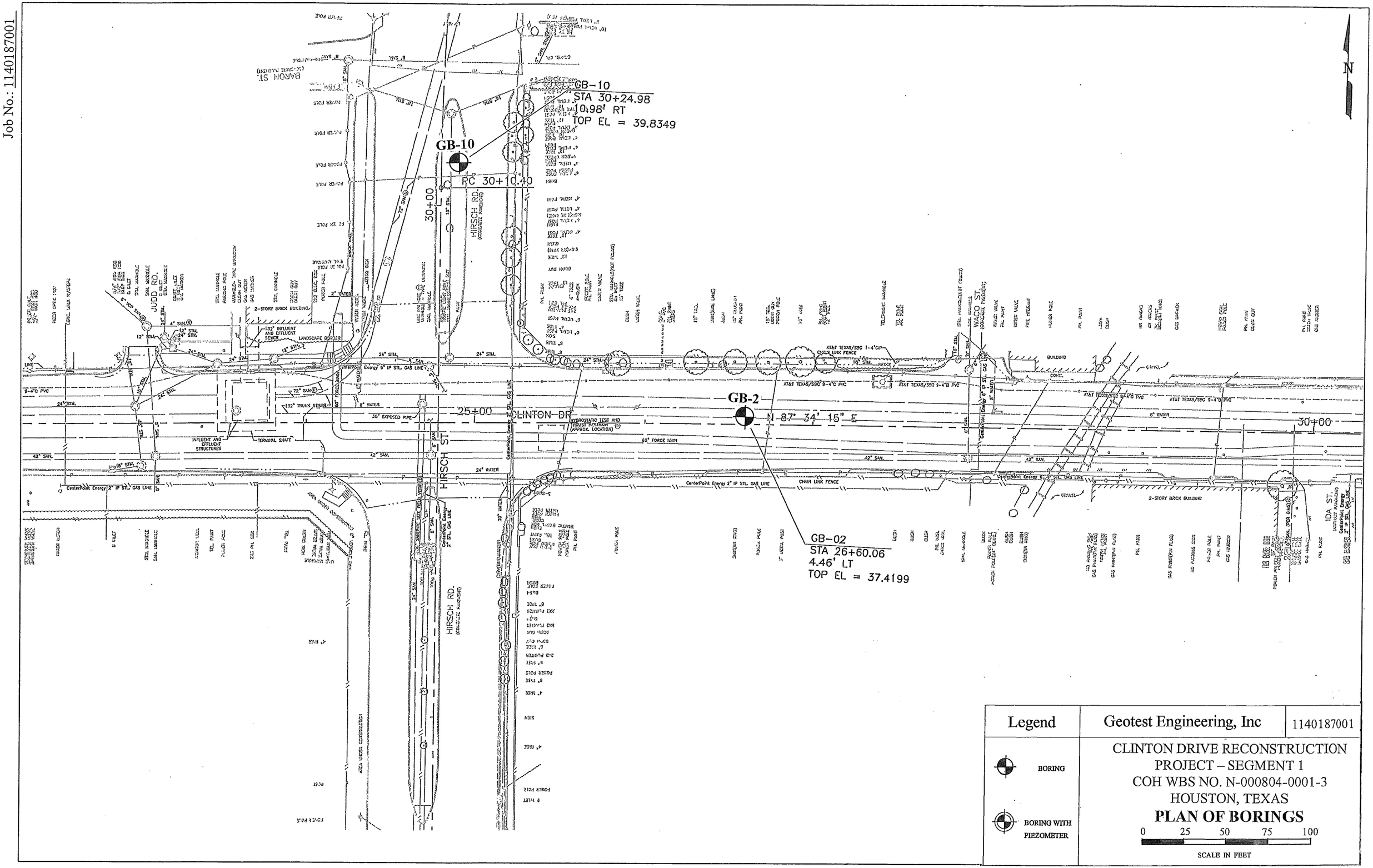
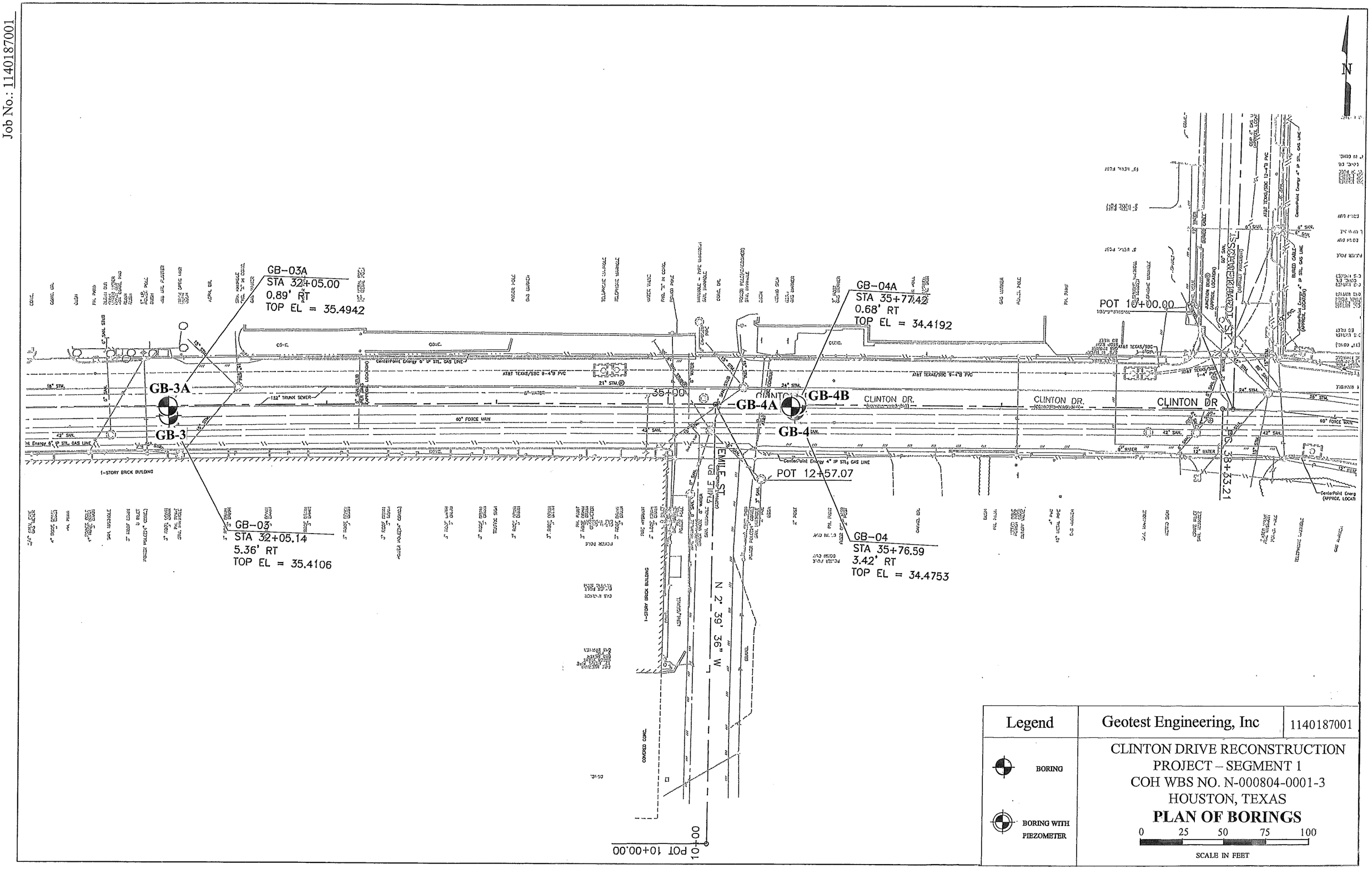


FIGURE 2.2

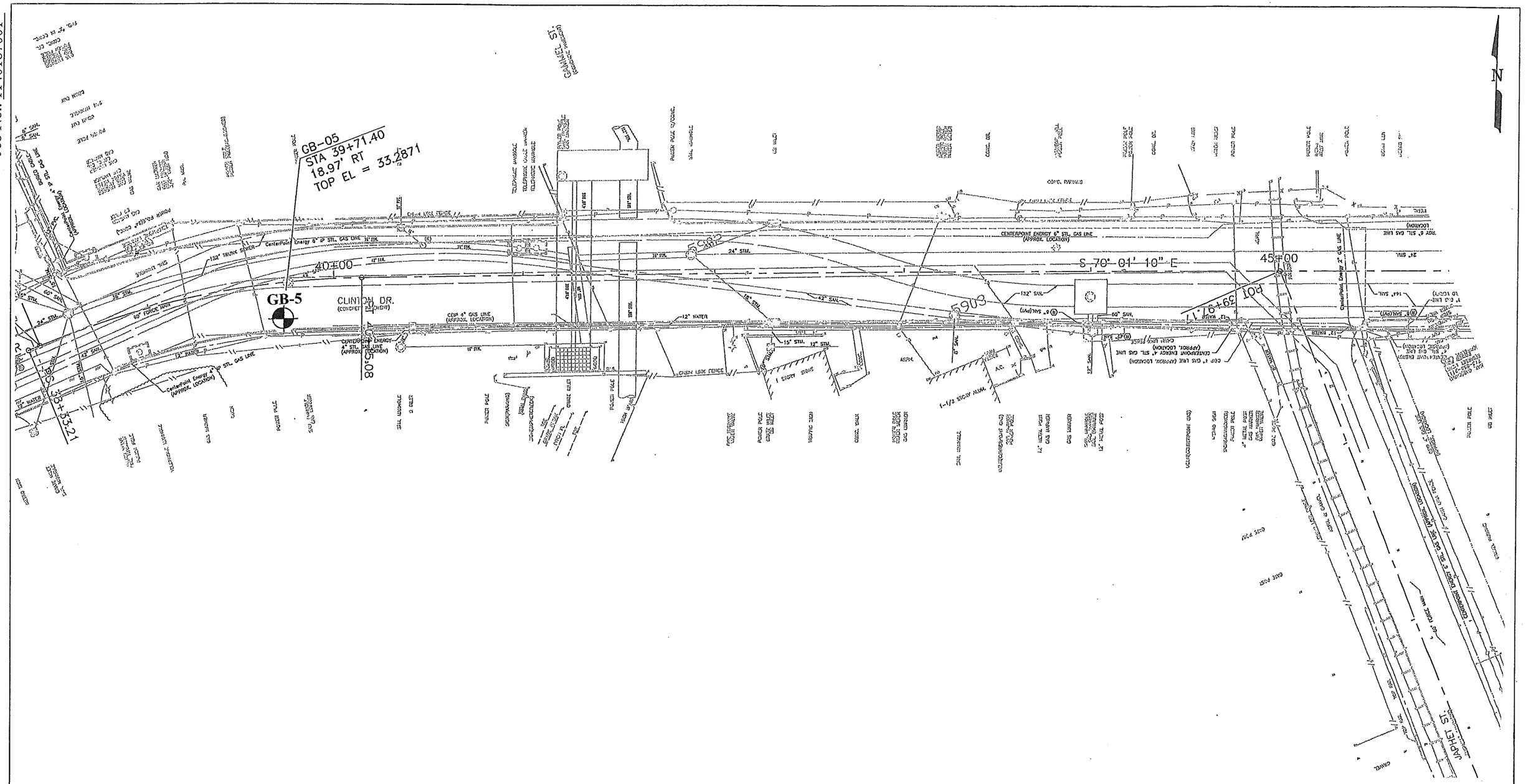
Job No.: 1140187001



Legend	Geotest Engineering, Inc	1140187001
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

FIGURE 2.3

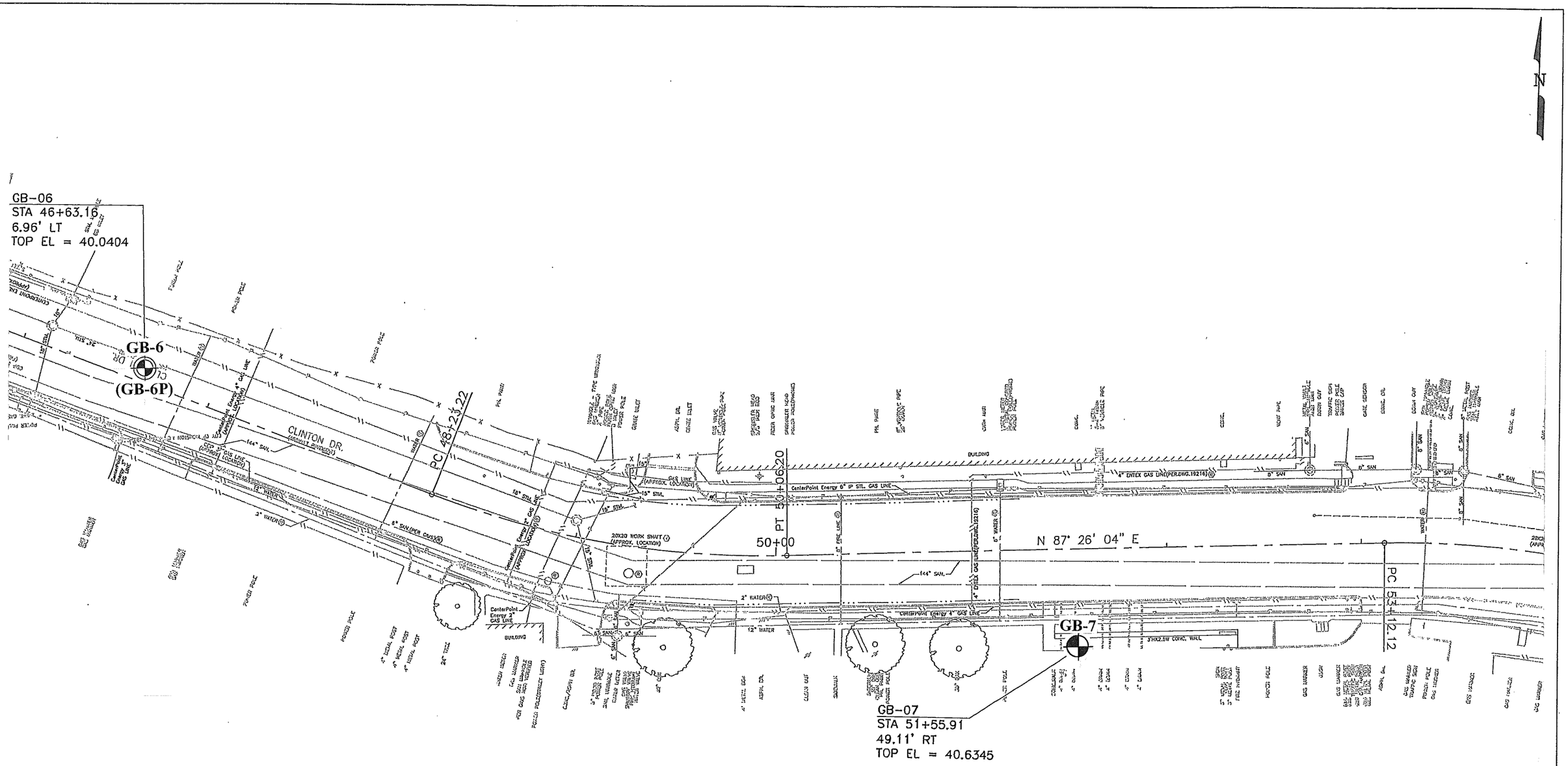
Job No.: 1140187001



Legend	Geotest Engineering, Inc	1140187001
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

FIGURE 2.4

Job No.: 1140187001



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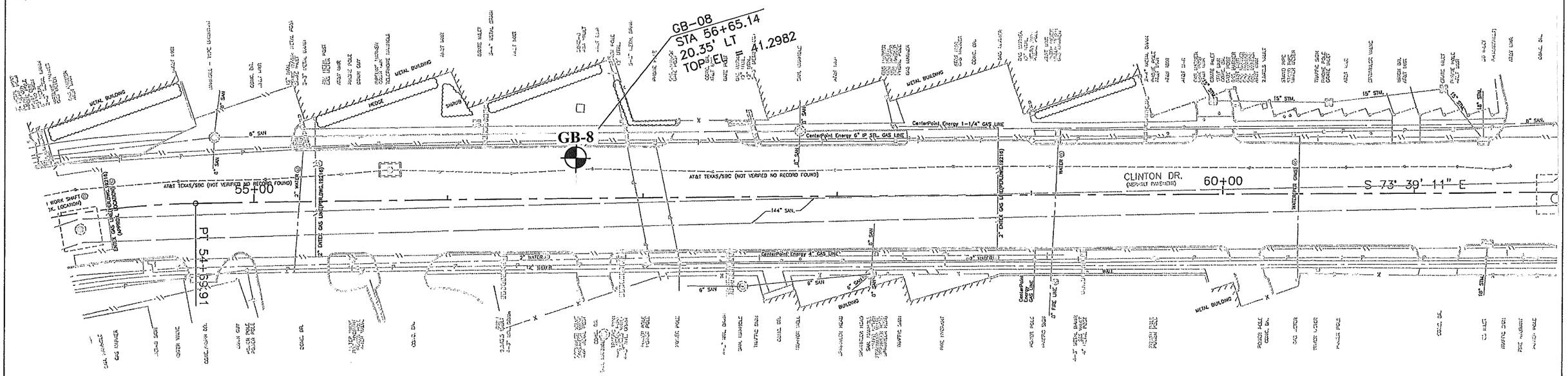
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

FIGURE 2.5

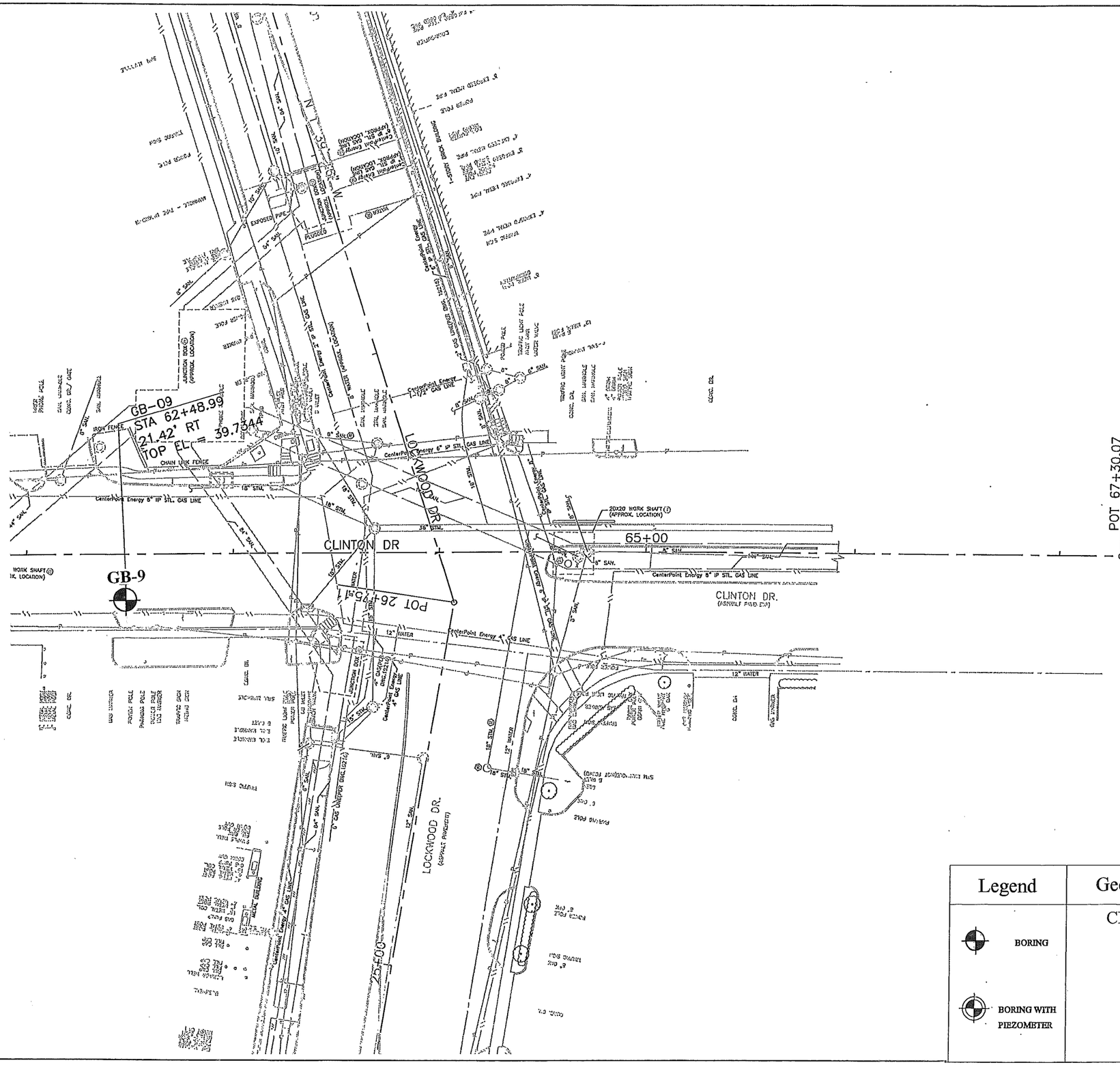
Job No.: 1140187001



Legend	Geotest Engineering, Inc	1140187001
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

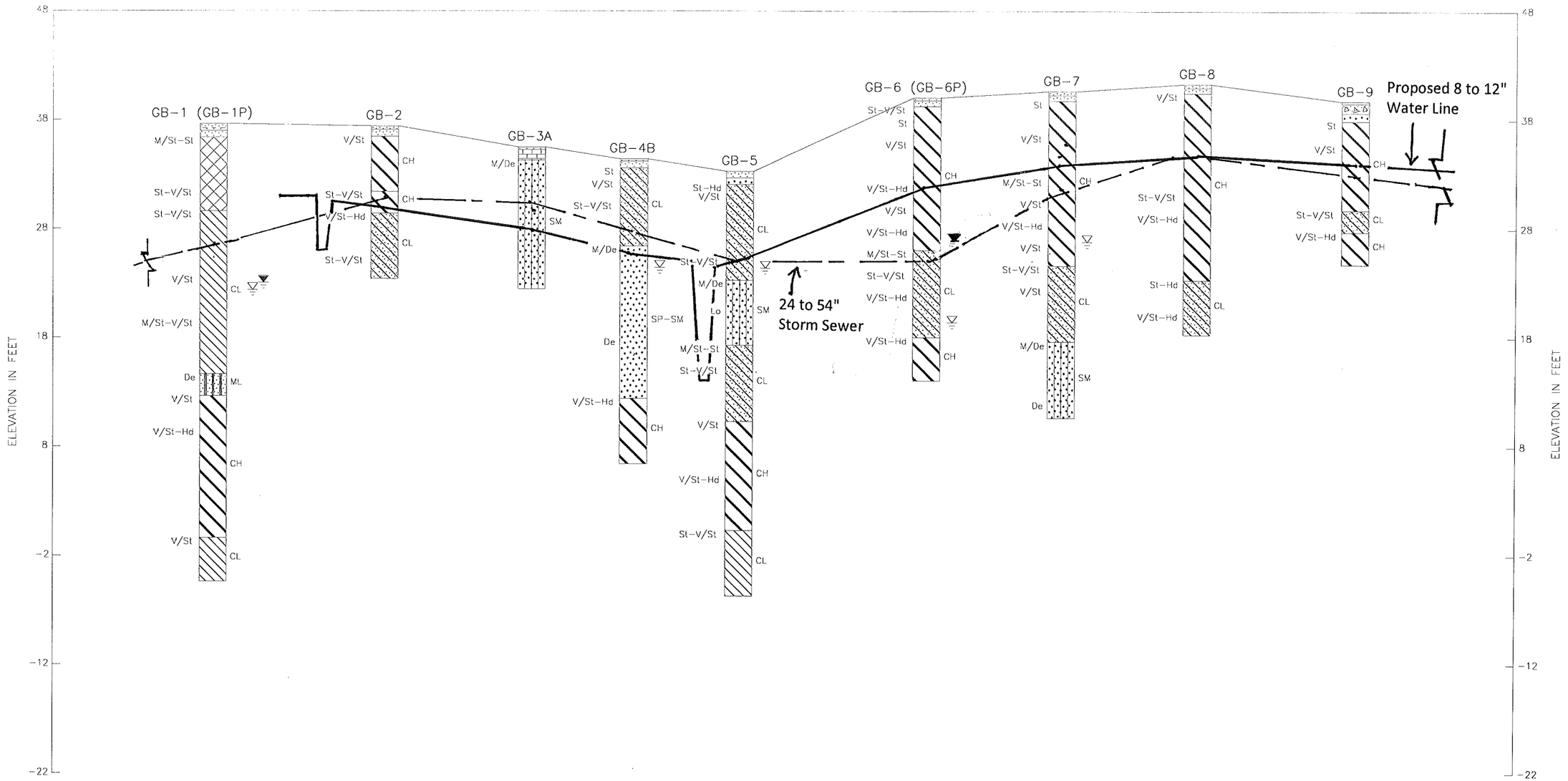
FIGURE 2.6

Job No.: 1140187001



Legend	Geotest Engineering, Inc	1140187001
BORING BORING WITH PIEZOMETER	CLINTON DRIVE RECONSTRUCTION PROJECT – SEGMENT 1 COH WBS NO. N-000804-0001-3 HOUSTON, TEXAS **PLAN OF BORINGS** 0 25 50 75 100 SCALE IN FEET	

FIGURE 2.7



GENERAL NOTES:

1. See Figures 2.1 THRU 2.7 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.
5. Ground surface elevation at each boring location was based on survey data provided to us by TRANSYSTEMS

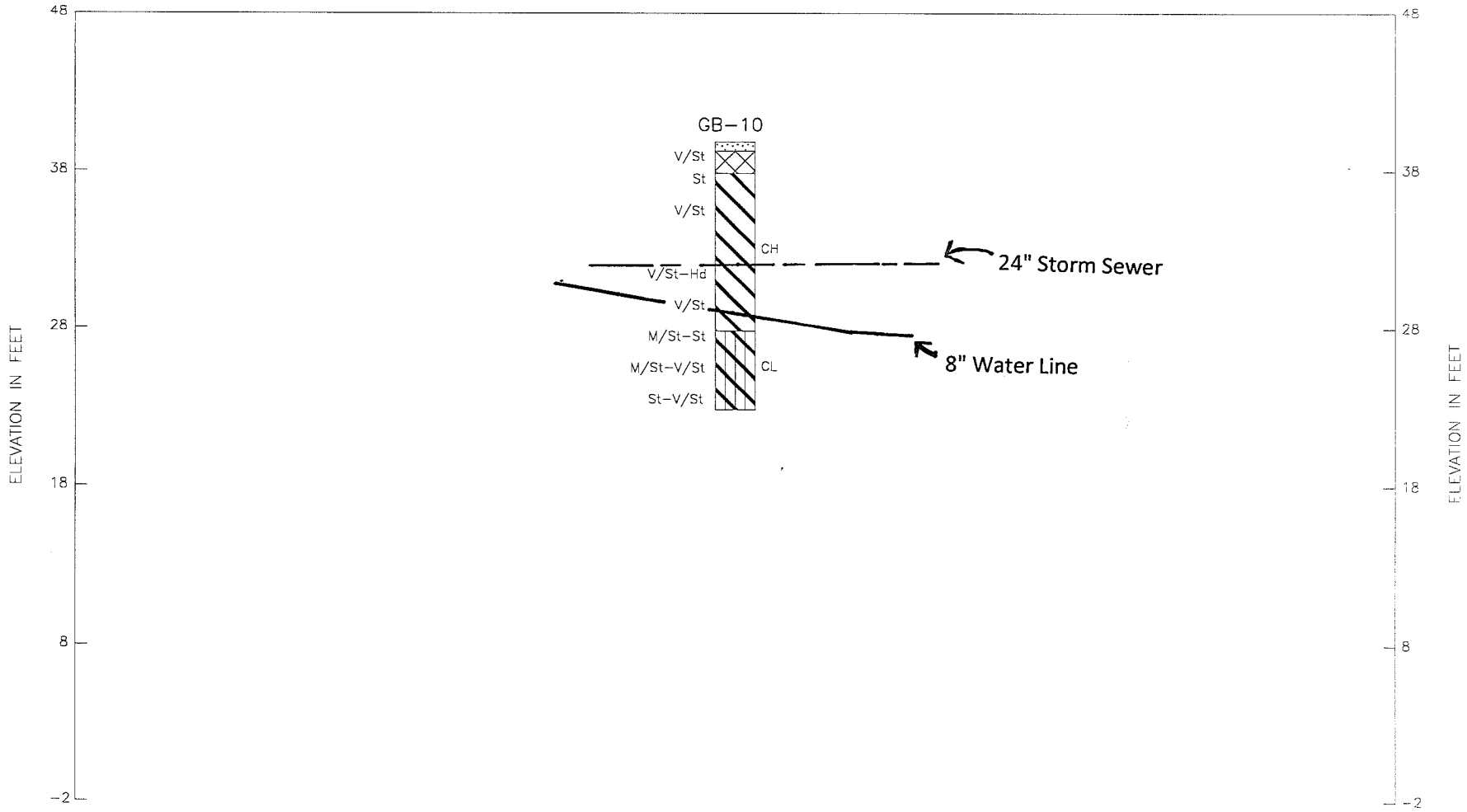
BORING LOG PROFILE

CLINTON DRIVE

0 200 400 600 800

HORIZONTAL SCALE IN FEET

Job No. 114018/001



GENERAL NOTES:

1. See Figure 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.
5. Ground surface elevation at each boring location was based on survey data provided to us by TRANSYSTEMS

BORING LOG PROFILE
HIRSCH STREET

0 200 400 600 800

HORIZONTAL SCALE IN FEET

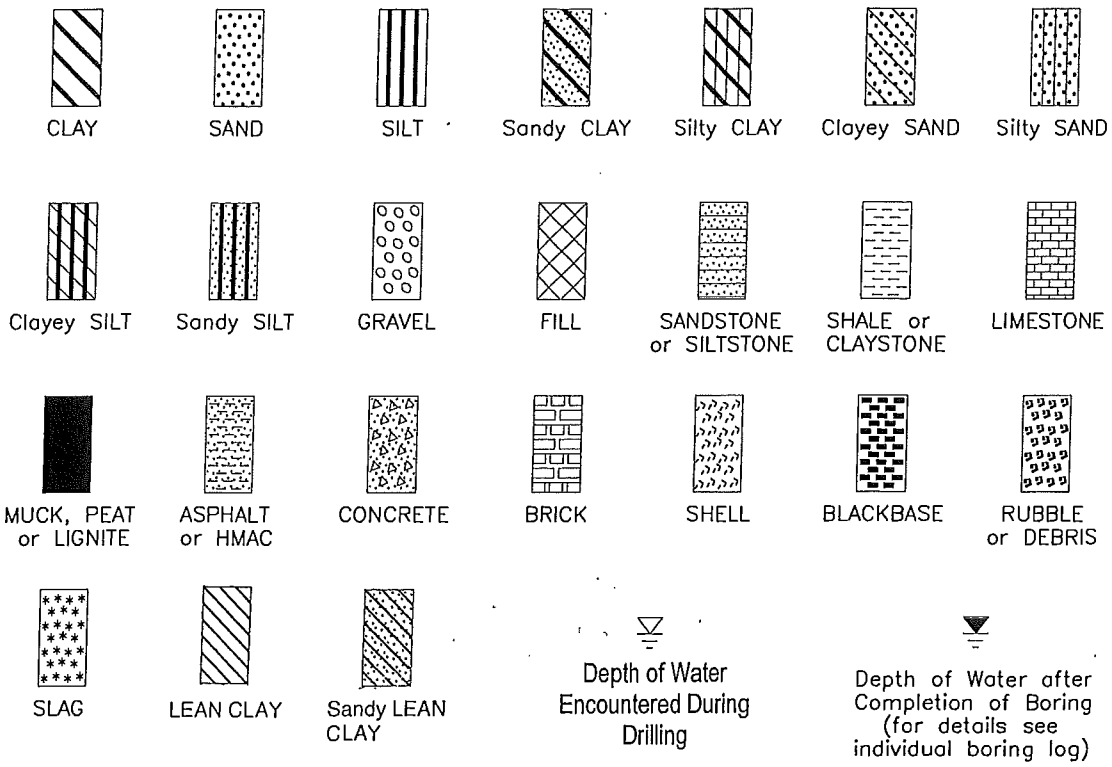
Geotest Engineering, Inc.

FIGURE 3.2

Job No. 1140187001

SYMBOLS AND ABBREVIATIONS USED ON BORING LOG PROFILE

LEGEND



ABBREVIATIONS USED FOR CONSISTENCY/DENSITY

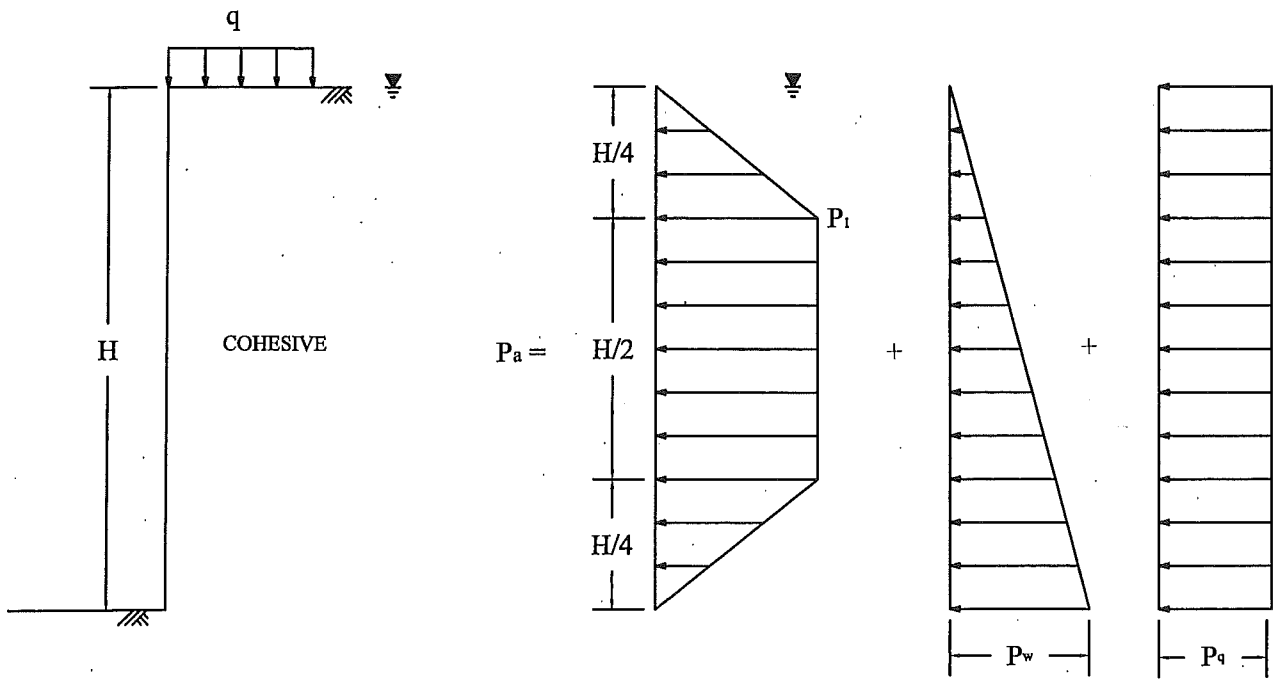
COHESIVE SOILS

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. 1140187001



TYPICAL SOIL PARAMETERS

BRACED WALL

See Table 2 for typical values of soil parameters

For $\gamma H/c \leq 4$

$P_1 = 0.3\ \gamma_c' H$
$P_w = \gamma_w H = 62.4\ H$
$P_q = 0.5\ q$

Where:

γ_c' = Submerged unit weight of cohesive soil, pcf;
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 cohesion soil, psf;

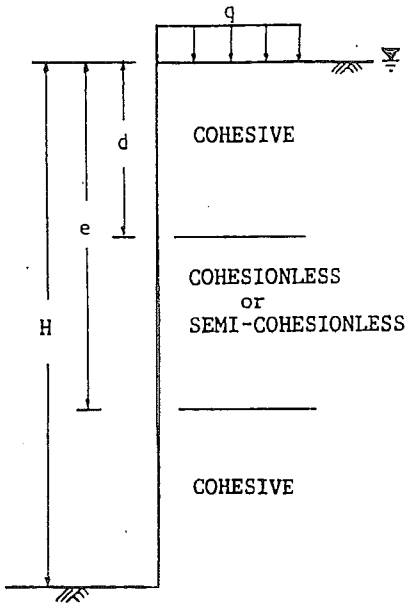
TRENCH SUPPORT EARTH PRESSURE

SUBMERGED COHESIVE SOIL

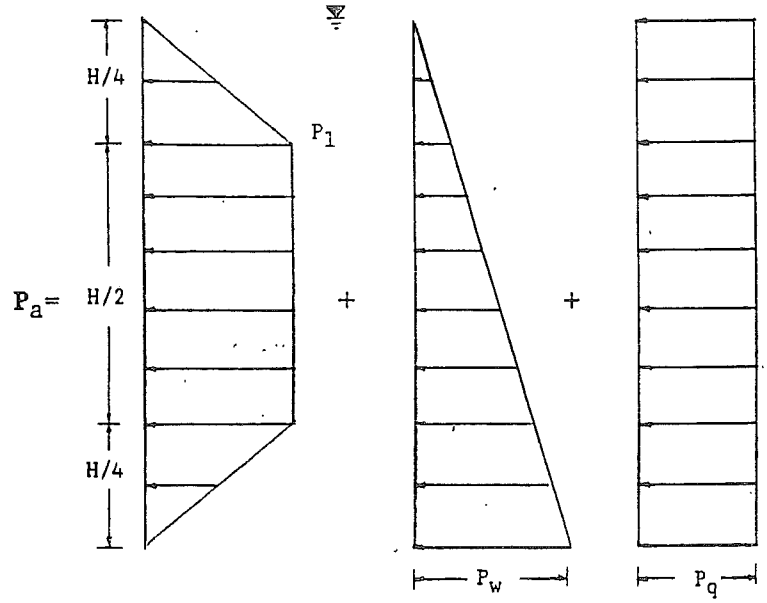
Geotest Engineering, Inc.

FIGURE 5.1

Job No. 1140187001



TYPICAL SOIL PARAMETERS



BRACED WALL

See Table 2 for typical values of soil parameters

$P_1 = 0.3\ \gamma'_{avg}\ H$

$P_w = \gamma_w\ H = 62.4\ H$

$P_q = 0.5q$

$$\gamma'_{avg} = \frac{\gamma_c'\ d + \gamma_s'\ (e-d) + \gamma_c'\ (H-e)}{H}$$

$\gamma_w = 62.4$ pcf

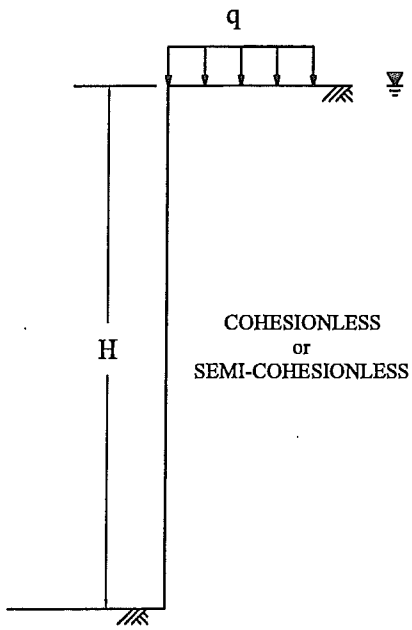
Where:

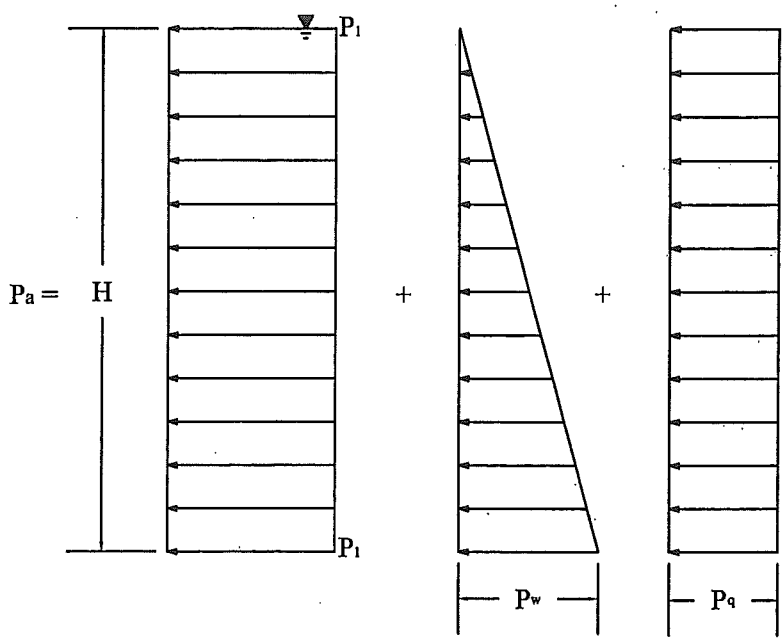
- γ_c' = Submerged unit weight of cohesive soil, pcf ;
- γ_s' = Submerged unit weight of cohesionless or semi-cohesionless soil, pcf ;
- γ_w = Unit weight of water, pcf;
- γ'_{avg} = Average submerged unit weight of soil, pcf ;
- q = Surcharge load at surface, psf;
- P_a = Lateral pressure, psf;
- P_1 = 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

TRENCH SUPPORT EARTH PRESSURE

SUBMERGED COHESIVE SOIL INTERBEDDED WITH COHESIONLESS OR SEMI-COHESIONLESS SOIL

Job No. 1140187001





TYPICAL SOIL PARAMETERS

BRACED WALL

See Table 2 for typical values of soil parameters

$P_1 = 0.65\ K_a\ \gamma_s'\ H$
$P_w = 62.4\ H$
$P_q = 0.5\ q$

$K_a = \tan^2(45 - \phi/2)$

Where:

γ_s' = Submerged unit weight of cohesionless or semi-cohesionless soil, 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;
K_a = Coefficient of active earth pressure;
ϕ = Angle of internal friction

TRENCH SUPPORT EARTH PRESSURE

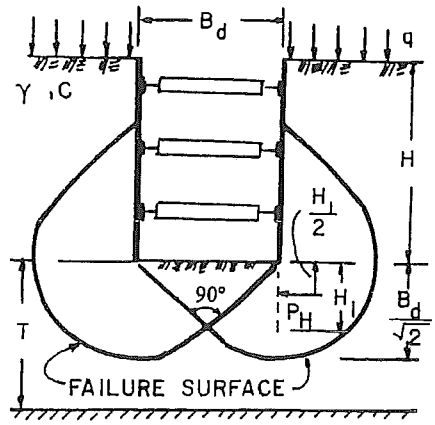
SUBMERGED COHESIONLESS OR SEMI-COHESIONLESS SOIL

Geotest Engineering, Inc.

FIGURE 5.3

Job No. 1140187001

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 2)

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)

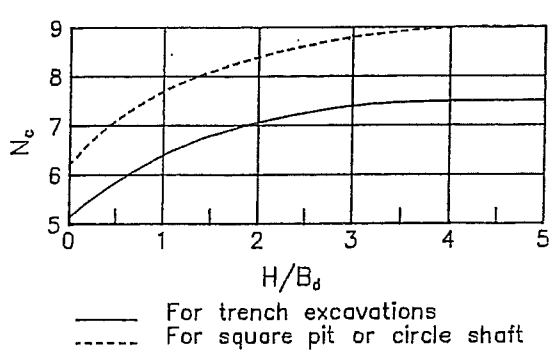
$$H_1 = \text{Buried length} = \frac{B_d}{2} \geq 5 \text{ feet}$$

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:

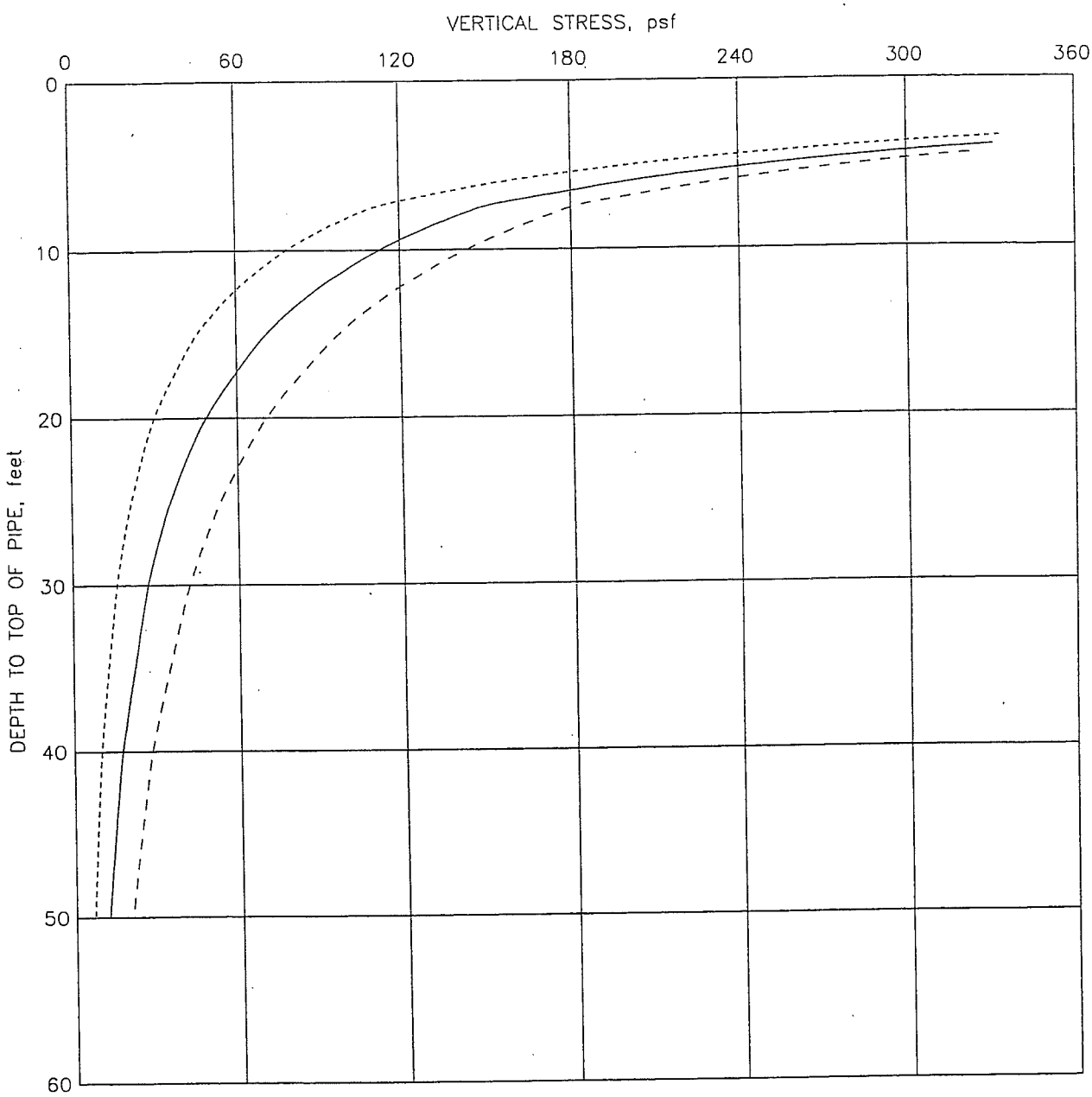
$$\text{If } H_1 > \frac{2}{3}\frac{B_d}{\sqrt{2}}, \quad P_H = 0.7\,(\gamma\, HB_d - 1.4CH - \pi CB_d) \text{ in lbs/ linear foot}$$

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



STABILITY OF BOTTOM FOR BRACED CUT

Job No. 1140187001



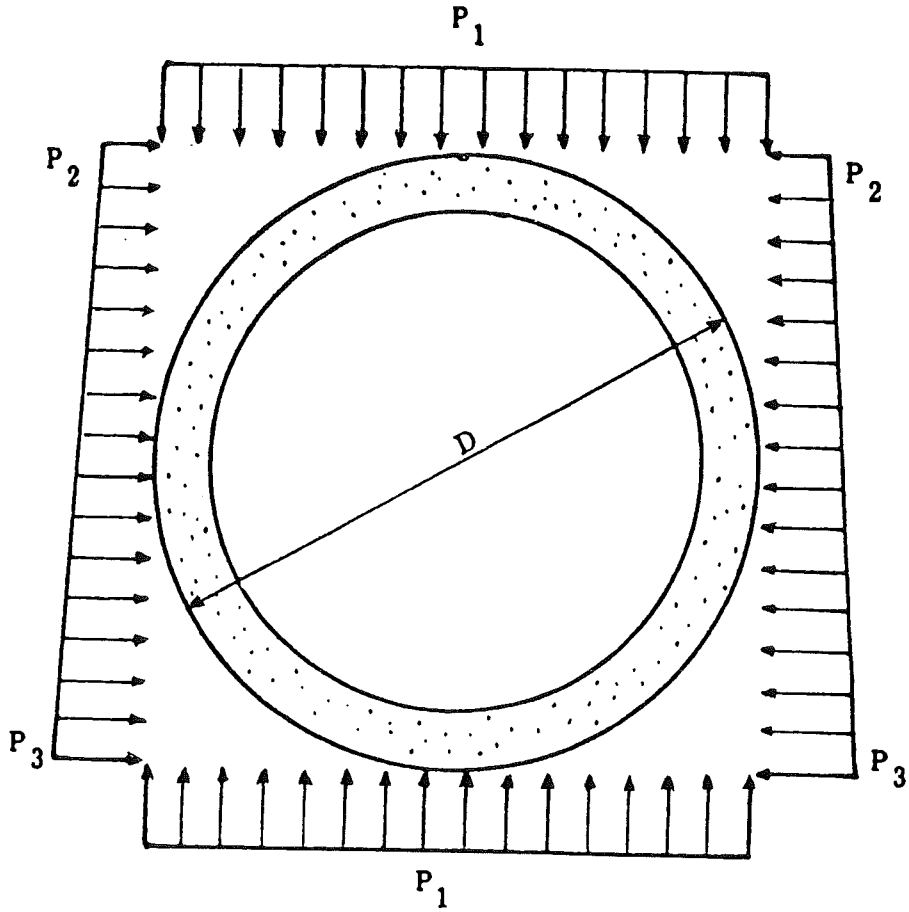
Legend:

------ One passing truck
——— Two passing trucks
– – – Four passing trucks

Notes: 1. The vertical stress was estimated using AASHTO H20 or HS20 truck axle loadings on paved surfaces.
2. Impact factor was included in the vertical stress.

VERTICAL STRESS ON PIPES
DUE TO TRAFFIC LOADS

Job No. 1140187001



$$P_1 = [(H + \frac{D}{2}) \times (\gamma - \gamma_w) + D_w \times \gamma_w] + q_s, \text{ for } D_w < H + \frac{D}{2}$$

$$P_1 = [(H + \frac{D}{2}) \times \gamma] + q_s \quad , \text{ for } D_w \geq H + \frac{D}{2}$$

$$P_2 = (H \times \gamma) + q_s$$

$$P_3 = [(H + D) \times \gamma] + q_s$$

Where: P_1, P_2, P_3 = Tunnel liner load, psf.

D = Tunnel outside diameter, ft.

H = Depth to top of tunnel; ft.

D_w = Depth to ground water level; ft.

γ = Wet unit weight of soil, pcf

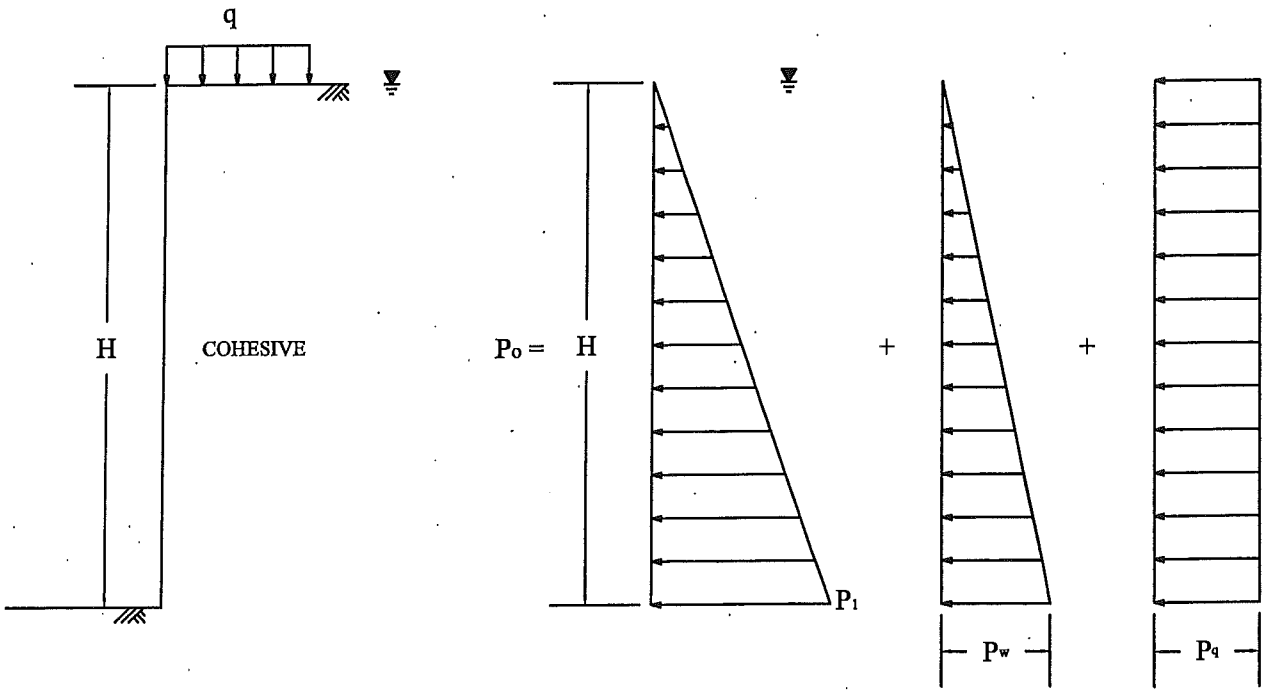
γ_w = Unit weight of water, 62.4 pcf

q_s = Surcharge load, psf.

K_o = Coefficient of Lateral Earth Pressure at rest (see Table 3)

TUNNEL LINER LOADS

Job No. 1140187001



TYPICAL SOIL PARAMETERS

See Table 2 for typical values of soil parameters

$K_{oc} = 1.0$

PERMANENT WALL

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

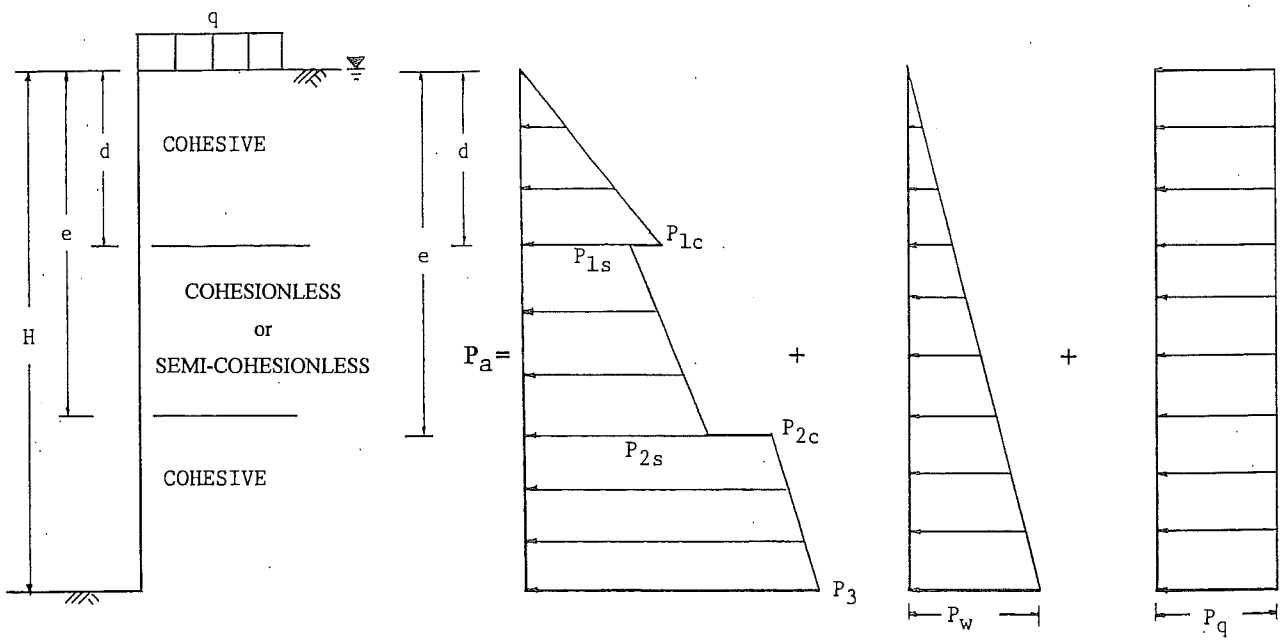
Where:

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. 1140187001



TYPICAL SOIL PARAMETERS

See Table 2 for typical values of soil parameters

$K_{oc} = 1.0$

$K_{os} = 1 - \sin\phi_s$

$\gamma_w = 62.4$ psf

PERMANENT WALL

$P_{1c} = \gamma'_c\, d\, K_{oc}$

$P_{1s} = \gamma'_c\, d\, K_{os}$

$P_{2s} = P_{1s} + \gamma'_s\, (e-d)\, K_{os}$

$P_{2c} = [\gamma'_c\, d + \gamma'_s\, (e-d)]\, K_{oc}$

$P_3 = [\gamma'_c\, d + \gamma'_s\, (e-d) + \gamma'_c\, (H-e)\,]\, K_{oc}$

$P_w = \gamma_w H = 62.4\, H$

$P_q = 0.5\, q$

Where:

γ_c' = Effective unit weight of cohesive soil, pcf;
γ_s' = Effective unit weight of cohesionless or semi-cohesionless soil, pcf;
ϕ_s = Internal friction angle of cohesionless or semi-cohesionless soil, degree;
K_{oc} = Coefficient of earth pressure at rest in cohesive soils;
Ko_{as} = Coefficient of earth pressure at rest in cohesionless or semi-cohesionless soil;
γ_w = Unit weight of water, pcf;
q = Surcharge load at surface, psf;
P_a = Lateral pressure, psf;
P_i, P_{ic}, P_{is} = Earth pressure at rest, psf; i = 1, 2, 3;
P_q = Horizontal pressure due to surcharge, psf;
P_w = Hydrostatic pressure due to groundwater, psf;
H = Height of wall, feet

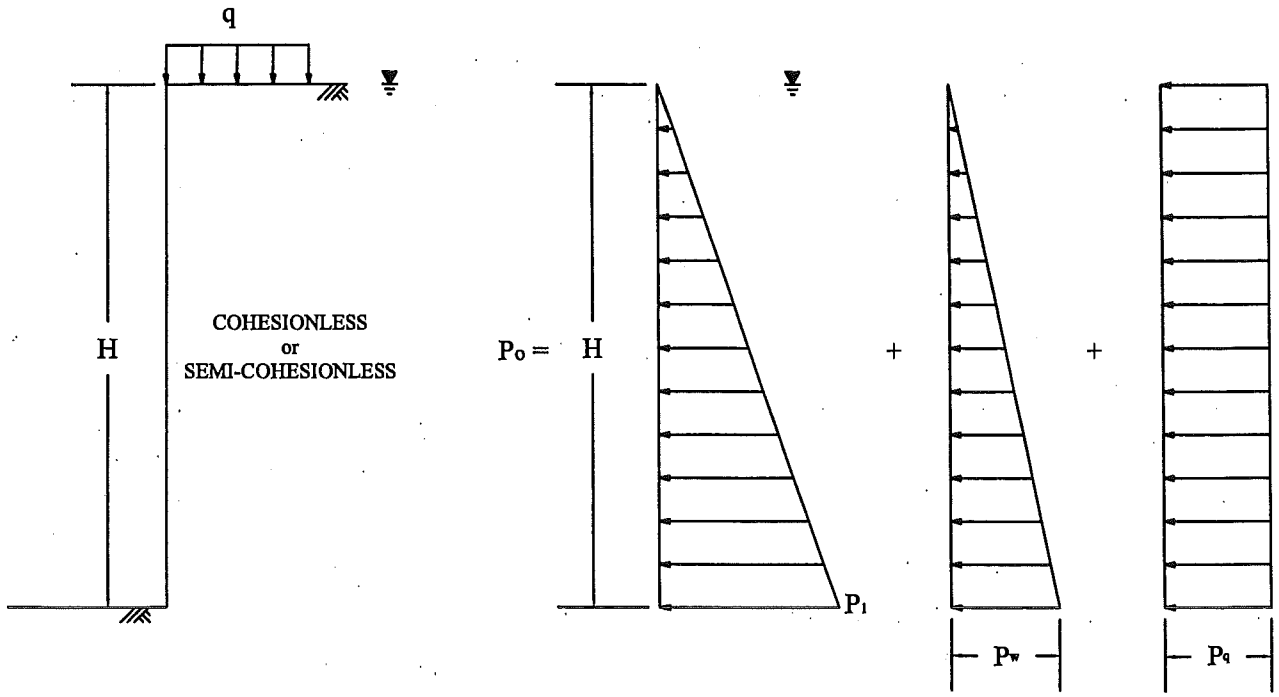
LATERAL EARTH PRESSURE DIAGRAM FOR PERMANENT WALL

SUBMERGED COHESIVE SOIL INTERBEDDED WITH COHESIONLESS OR SEMI-COHESIONLESS SOIL

Geotest Engineering, Inc.

FIGURE 9.2

Job No. 1140187001



TYPICAL SOIL PARAMETERS

PERMANENT WALL

See Table 2 for typical values of soil parameters

$K_{os} = 1 - \sin \phi$

$P_1 = K_{os} \gamma_s' H$
$P_w = \gamma_w H = 62.4 H$
$P_q = 0.5 q$

Where:

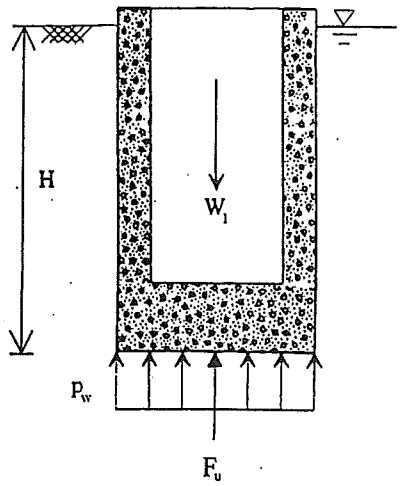
γ_s' = Submerged unit weight of cohesionless or semi-cohesionless soil, pcf;
K_{os} = Coefficent of earth pressure at-rest in cohesionless or semi-cohesionless soil;
γ_w = Unit weight of water, pcf;
q = Surcharge load at surface, psf;
P_o = Lateral pressure, psf;
P_1 = Earth pressure at-rest, psf;
P_q = Horizontal pressure due to surcharge, psf;
P_w = Hydrostatic pressure due to groundwater, psf;
H = Depth of wall, feet;
ϕ = Angle of internal friction, degree

LATERAL EARTH PRESSURE DIAGRAM FOR PERMANENT WALL

SUBMERGED COHESIONLESS OR SEMI-COHESIONLESS SOIL

Job No. 1140187001

(a) DEAD WEIGHT OF STRUCTURE

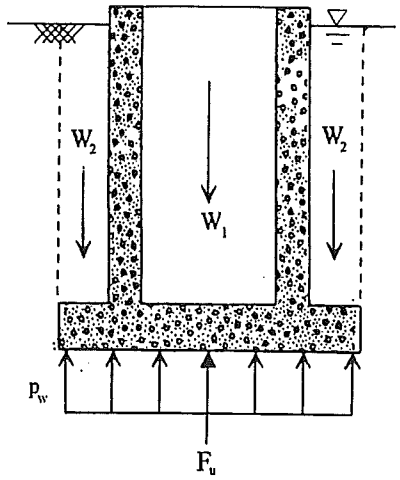


$$p_w = H\gamma_w$$

$$F_u = A_b p_w$$

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

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

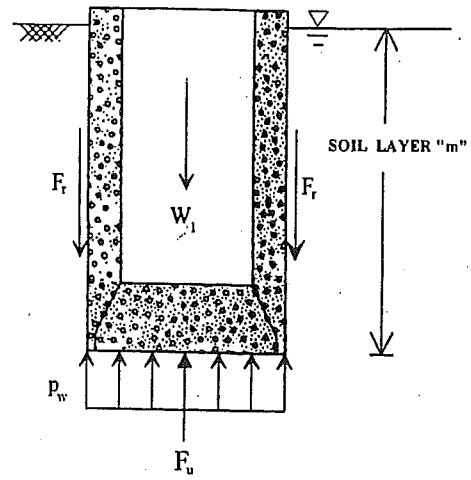


$$p_w = H\gamma_w$$

$$F_u = A_b p_w$$

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

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



$$p_w = H\gamma_w$$

$$F_u = A_b p_w$$

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

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

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

See Table 2 for typical values of soil parameters

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

TABLES

	Table
Summary of Boring Information	1
Geotechnical Design Parameter Summary – Open Cut Excavation	2
Geotechnical Design Parameter Summary – Trenchless Installation	3

TABLE 1

SUMMARY OF BORING INFORMATION

Boring No.	Depth (feet)	Street	Northing	Easting	Elevation (feet)
GB-1 (GB-1P)	42	Clinton	13845656.37	3133350.66	37.62
GB-2	14	Clinton	13845694.68	3133982.94	37.42
GB-3	7	Clinton	13845707.97	3134527.94	35.41
GB-3A	13	Clinton	13845712.43	3134527.61	35.49
GB-4	16	Clinton	13845725.65	3134898.97	34.48
GB-4B	28	Clinton	13845728.43	3134899.69	34.42
GB-5	39	Clinton	13845707.00	3135287.35	33.29
GB-6 (GB-6P)	26	Clinton	13845497.47	3135945.27	40.04
GB-7	30	Clinton	13845366.35	3136424.78	40.63
GB-8	23	Clinton	13845368.14	3136927.64	41.30
GB-9	15	Clinton	13845163.73	3137476.13	39.73
GB-10	17	Hirsch	13845835.23	3133805.50	39.84

TABLE 2

GEOTECHNICAL DESIGN PARAMETER SUMMARY
OPEN-CUT EXCAVATION

Alignments	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
24" to 54" Storm Sewer and 12" Water Along Clinton Drive and Hirsch	GB-1 (GB-1P)	Cohesive	0-8	123	62	800	--
			8-15	130	65	1,000	--
			15-18	130	65	2,000	--
			18-23	130	65	800	--
		Cohesionless	23-25	110	55	--	30
		Cohesive	25-42	125	63	3,000	--
	GB-2	Cohesive	0-6	125	63	2,500	--
			6-13	125	63	1,600	--
			13-15	120	60	1,000	--
	GB-3A	Cohesionless	0-13	104	52	--	30
	GB-4B	Cohesive	0-8	125	63	1,000	--
		Cohesionless	8-22	110	55	--	30
		Cohesive	22-27	120	60	3,500	--
	GB-5	Cohesive	0-10	130	65	1,600	--
		Cohesionless	10-16	110	55	--	28
		Cohesive	16-23	130	65	800	--
			23-33	130	65	2,500	--
			33-39	120	60	1,200	--
	GB-6 (GB-6P)	Cohesive	0-6	125	63	1,500	--
			6-14	130	65	3,000	--
			14-22	130	65	1,500	--
			22-26	120	60	3,500	--
	GB-7	Cohesive	0-10	125	63	1,400	--
			10-16	120	60	3,000	--
			16-23	130	65	1,500	--
		Cohesionless	23-30	114	57	--	30
	GB-8 and GB-9	Cohesive	0-6	115	58	1,000	--
			6-10	125	63	2,000	--
			10-17	125	63	500	--
			17-23	125	63	1,800	--
24" Storm Sewer and 8" Water Along Hirsch	GB-10	Cohesive	0-6	115	58	1,000	--
			6-10	125	63	2,000	--
			10-17	125	63	1,000	--
							--

Note: 1) Cohesive soils include Fat Clay, Fat Clay w/sand, Lean Clay, Lean Clay with sand and Sandy Lean Clay.
2) Cohesionless soils include Silty Sand, Fine Sand w/silt and Sandy Silt.

TABLE 3

GEOTECHNICAL DESIGN PARAMETER SUMMARY
TRENCHLESS INSTALLATION (Borings GB-1, GB-2 & GB-10)

PROPERTY		COHESIVE SOILS[1]	COHESIONLESS SOILS[2]
Wet Unit Weight, γ, pcf	0-8	123	--
	8-15	130	--
	15-17	130	--
	17-23	130 (GB-1 only)	--
	23-25	--	110 (GB-1 only)
	25-42	125 (GB-1 only)	--
Submerged Unit Weight, γ', pcf	0-8	63	--
	8-15	65	--
	15-17	65	--
	17-23	65 (GB-1 only)	--
	23-25	--	55 (GB-1 only)
	25-42	63 (GB-1 only)	--
Moisture Content (%)	0-8	20	--
	8-15	13	--
	15-17	15	--
	17-23	19 (GB-1 only)	--
	23-25	--	21 (GB-1 only)
	25-42	21 (GB-1 only)	--
UNDRAINED PROPERTIES			
Undrained Cohesion, C_u, psf	4-8*	800	--
	8-15*	1,000	--
	15-18*	2,000	--
Angle of Internal Friction, ϕ, degrees	4-8*	--	--
	8-15*	--	--
	15-18*	--	--
Elastic Modulus, E, psf	4-8*	240,000	--
	8-15*	300,000	--
	15-18*	600,000	--
Coefficient of Lateral Earth Pressure at Rest, K_o,	4-8*	1.2	--
	8-15*	1.2	--
	15-18*	1.2	--
Poisson's Ratio, μ		0.45	0.3
DRAINED PROPERTIES			
Drained Cohesion, C', psf	4-8*	0	--
	8-15*	0	--
	15-18*	0	--
Angle of Internal Friction, ϕ', degrees	4-8*	20	--
	8-15*	20	--
	15-18*	21	--
Elastic Modulus, E, psf	4-8*	140,000	--
	8-15*	180,000	--
	15-18*	360,000	--

Notes: 1. Cohesive soils include Fat Clay, Fat Clay with sand, Lean Clay and Lean Clay with sand.
2. Cohesionless soil includes Sandy Silt.
* Tunnel zone which includes invert depth plus 6 feet above invert plus 6 feet below invert.

APPENDIX A

	Figure
Log of Borings	A-1 thru A-10
Symbols and Terms Used on Boring Logs	A-11
Piezometer Installation Report	A-12 and A-13

LOG OF BORING NO. GB-1 (GB-1P)

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845656.37, E 3133350.66
Clinton Drive; See Plan of Borings (Figure 2.1)

COMPLETION DEPTH : 42.0 FT.

SURFACE ELEVATION : 37.62 FT.

DATE : 05-23-12

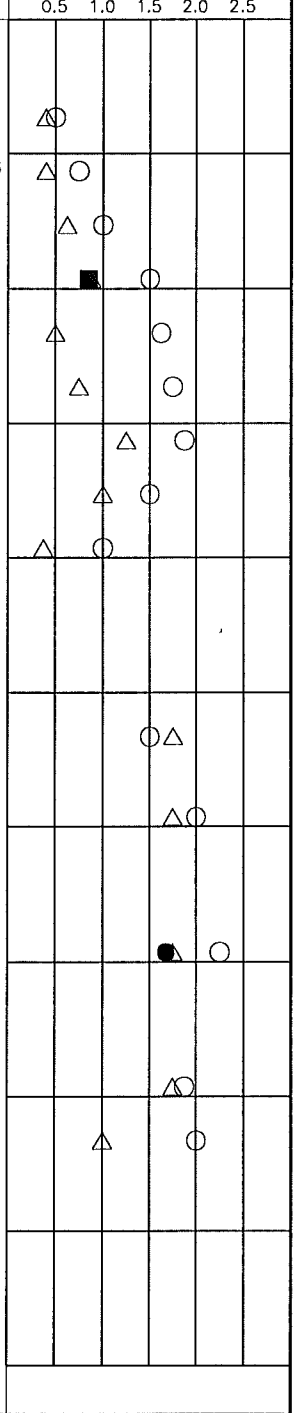
SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 23.0 FT.
WET ROTARY : 23.0 TO 42.0 FT.

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, %
37.6	0	7.5" Asphalt							
37.0		7.5" Concrete							
36.4		FILL: medium stiff to stiff brown and gray lean clay w/sand and calcareous nodules			102	21			
	5			73		19	32	16	16
29.6		–w/ferrous nodules 4'–8' –stiff to very stiff 6'–8'				16			
	10	Stiff to very stiff gray LEAN CLAY (CL) w/sand, ferrous stains and calcareous nodules			117	14			
						14			
		–gray and brown 12'–14'		80		15	37	18	19
	15	–very stiff gray and yellowish brown 14'–18'				19			
		–medium stiff to very stiff 18'–20'				19			
	20	–gray and brown 18'–23'		74		19	24	15	9
14.6		Dense brown and gray SANDY SILT (ML)							
12.6	25		34	61		21	22	20	2
		Very stiff gray and yellowish brown FAT CLAY (CH) –very stiff to hard 28'–35'				18			
	30					19			
		–reddish brown and gray 32'–42'							
	35			99	107	24	68	26	42
-0.4		Very stiff reddish brown and gray LEAN CLAY (CL) w/calcareous and ferrous nodules							
	40					20			
-4.4						21			
	45								
	50								

SYMBOL

SAMPLES

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



NOTE :
See Piezometer GB-1P for water level readings.

DEPTH TO WATER IN BORING :
▽: FREE WATER 1st ENCOUNTERED AT 23.0 FT. DURING DRILLING; AFTER 15.0 MIN. AT 15.3 FT.
▼: WATER DEPTH AT 14.6 FT., HOLE OPEN TO 42.0 FT. ON 08-02-12.

Geotest Engineering, Inc.

FIGURE A-1

LOG OF BORING NO. GB-2

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845694.68, E 3133982.94
Clinton Drive; See Plan of Borings (Figure 2.2)

COMPLETION DEPTH : 14.0 FT.

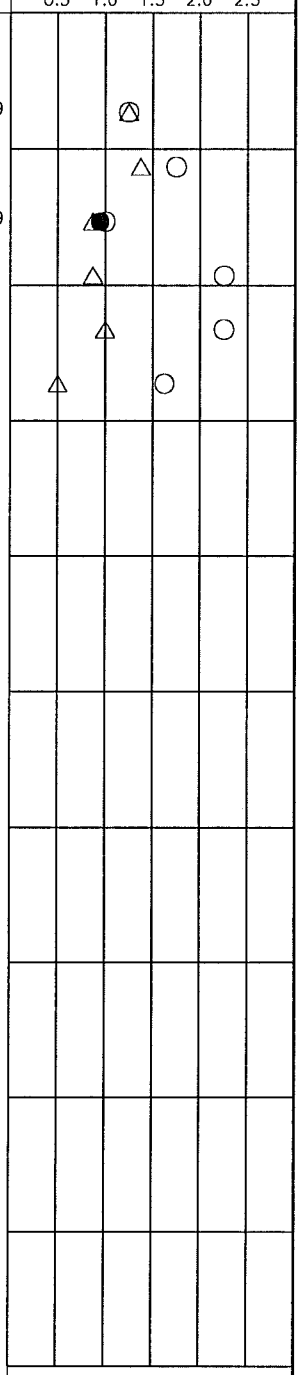
SURFACE ELEVATION : 37.42 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 14.0 FT.
WET ROTARY : -- TO -- FT.

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, %
37.4	0	4" Asphalt							
37.1		7.5" Concrete							
36.5		Very stiff dark gray FAT CLAY (CH) w/sand and ferrous nodules		83	105	21	64	25	39
	5								
31.4		-gray 4'-6'				20			
29.4		Stiff to very stiff brown and gray FAT CLAY (CH) w/ferrous and calcareous nodules		91	109	20	64	25	39
	10					14			
						13			
23.4		Very stiff to hard gray SANDY LEAN CLAY (CL) w/sand seams and ferrous nodules				15			
	15	-stiff to very stiff 12'-14'							
	20								
	25								
	30								
	35								
	40								
	45								
	50								

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 14.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-2

LOG OF BORING NO. GB-3

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845707.97, E 3134527.94
Clinton Drive; See Plan of Borings (Figure 2.3)

COMPLETION DEPTH : 7.0 FT.

SURFACE ELEVATION : 35.41 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 7.0 FT.
WET ROTARY : -- TO -- FT.

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

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, %	UNDRAINED SHEAR STRENGTH, TSF (0.5 1.0 1.5 2.0 2.5)
35.4	0			3" Asphalt								
35.2												
34.2				11.5" Limestone								
				Medium dense brown SILTY SAND (SM)	22	14		7				
	5				23	13		8				
28.4												
	10			NOTE : Hit hard obstruction at 7 feet. Offset to boring GB-3A.								
	15											
	20											
	25											
	30											
	35											
	40											
	45											
	50											

DEPTH TO WATER IN BORING :
NO GROUNDWATER ENCOUNTERED DURING DRILLING.
HOLE OPEN TO 7.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-3

LOG OF BORING NO. GB-3A

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845712.43, E 3134527.61
Clinton Drive; See Plan of Borings (Figure 2.3)

COMPLETION DEPTH : 13.0 FT.

SURFACE ELEVATION : 35.49 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 13.0 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, %	UNDRAINED SHEAR STRENGTH, TSF (○ HAND PENETROMETER, ● UNCONFINED COMPRESSION, ■ UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION, △ TORVANE) 0.5 1.0 1.5 2.0 2.5
35.5	0			3" Asphalt								
35.2				11.5" Limestone								
34.3				Medium dense brown and gray SILTY SAND (SM)								
					22	14		7				
	5				23	13		8				
					19			8				
	10				22	19		13				
					20			13				
22.5					17			14				
	15											
	20											
	25											
	30											
	35											
	40											
	45											
	50											

DEPTH TO WATER IN BORING :
NO GROUNDWATER ENCOUNTERED DURING DRILLING.
HOLE OPEN TO 13.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-3A

LOG OF BORING NO. GB-4

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845725.65, E 3134898.97
Clinton Drive; See Plon of Borings (Figure 2.3)

COMPLETION DEPTH : 16.0 FT.

SURFACE ELEVATION : 34.48 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 13.0 FT.
WET ROTARY : 13.0 TO 16.0 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, %	UNDRAINED SHEAR STRENGTH, TSF (○ HAND PENETROMETER, ● UNCONFINED COMPRESSION, ■ UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION, △ TORVANE) 0.5 1.0 1.5 2.0 2.5
34.5	0			1.5" Asphalt								
33.8				5.75" Limestone								
				3.5" Asphalt	17							
	5			FILL: medium dense brown silty sand	21							
	10				25							
					21							
					18							
	15											
18.5												
	20											
	25											
	30											
	35											
	40											
	45											
	50											

NOTE :
Hit hard obstruction at 16 feet and offset to boring GB-4A. Boring GB-4A was augered to 6 feet, hit hard obstruction at 6 feet, and offset to boring GB-4B. 0.895 0 0.895

DEPTH TO WATER IN BORING :
▽: FREE WATER ENCOUNTERED AT 13.0 FT. DURING DRILLING.
HOLE OPEN TO 15.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-4

LOG OF BORING NO. GB-4B

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845728.43, E 3134899.69
Clinton Drive; See Plan of Borings (Figure 2.3)

COMPLETION DEPTH : 28.0 FT.

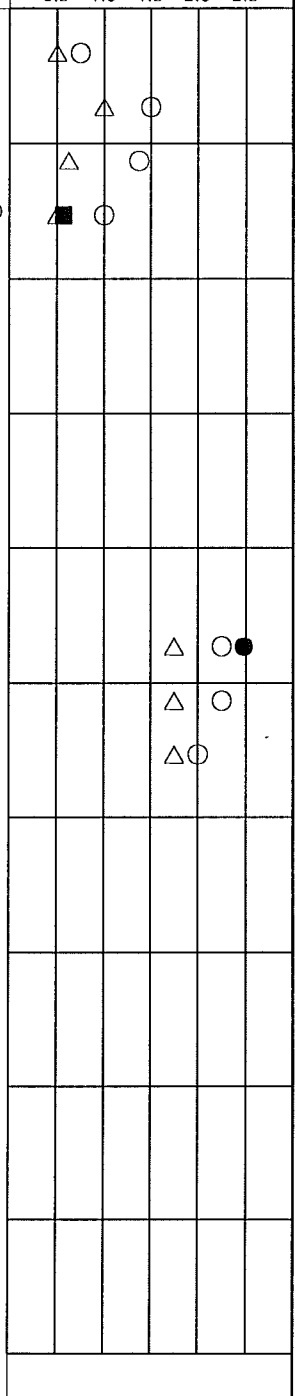
SURFACE ELEVATION : 34.42 FT.

DATE : 05-25-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 14.0 FT.
WET ROTARY : 14.0 TO 28.0 FT.

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, %
34.4	0	2.5" Asphalt							
34.2		7.5" Concrete							
33.6		Stiff dark gray SANDY LEAN CLAY (CL) w/sand seams				18			
	5	–very stiff gray and yellowish brown 2'–4'				11			
26.4		–stiff to very stiff gray 4'–8'		63	113	14	25	15	10
	10	Medium dense brown and gray FINE SAND (SP-SM) w/silt	15			8			
			15	9		9			
			14			20			
	15	–dense 16.5'–20'	12			22			
			36			19			
	20		39			19			
12.4		Very stiff to hard gray and yellowish brown FAT CLAY (CH), slickensided		93	114	18	53	22	31
	25					19			
6.4						23			
	30								
	35								
	40								
	45								
	50								

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



DEPTH TO WATER IN BORING :
▽: FREE WATER 1st ENCOUNTERED AT 14.0 FT. DURING DRILLING; AFTER 10.0 MIN. AT 10.0 FT.
HOLE OPEN TO 28.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-4B

LOG OF BORING NO. GB-5

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845707.00, E 3135287.35
Clinton Drive; See Plan of Borings (Figure 2.4)

COMPLETION DEPTH : 39.0 FT.

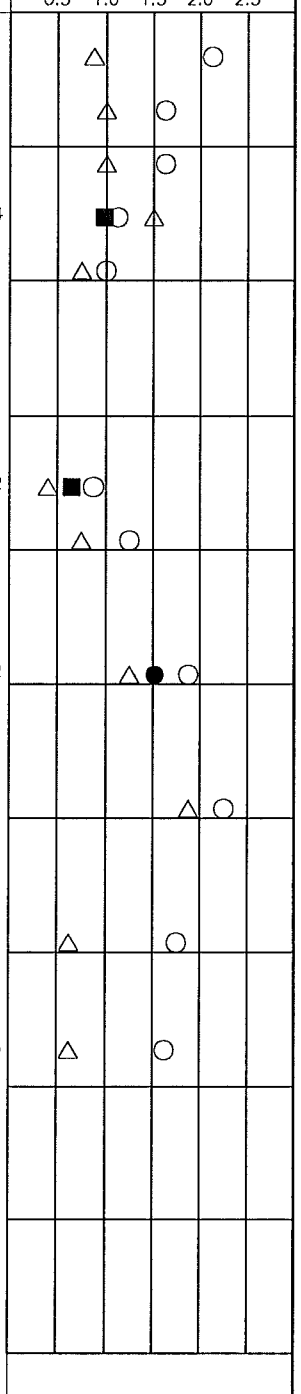
SURFACE ELEVATION : 33.29 FT.

DATE : 05-23-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 12.0 FT.
WET ROTARY : 12.0 TO 39.0 FT.

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, %
33.3	0	7.5" Concrete							
32.7		7" Cement Stabilized Sand							
32.1		Stiff to hard gray and brown SANDY LEAN CLAY (CL) w/sand seams				16			
					118	16			
	5	–very stiff 2'–8'				14			
		–stiff to very stiff 8'–10'		66	117	15	29	15	14
						17			
23.3	10	Medium dense brown and gray SILTY SAND (SM) –loose 12.5'–16'	13	13		17			
			7			18			
	15		9			21			
17.3		Medium stiff to stiff gray SANDY LEAN CLAY (CL) w/sand seams		64	116	17	27	15	12
	20	–stiff to very stiff 18'–20'				17			
10.3		Very stiff gray FAT CLAY (CH)		94	112	18	54	22	32
	25								
		–very stiff to hard 28'–30' –reddish brown and gray w/ferrous stains 28'–33'				20			
	30								
0.3		Stiff to very stiff reddish brown and gray LEAN CLAY (CL) –w/sand stone seams 35'–37'				22			
	35								
-5.7				92		23	25	17	8
	40								
	45								
	50								

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



DEPTH TO WATER IN BORING :
▽: FREE WATER 1st ENCOUNTERED AT 12.0 FT. DURING DRILLING; AFTER 10.0 MIN. AT 8.9 FT.
HOLE OPEN TO 39.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-5

LOG OF BORING NO. GB-6 (GB-6P)

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845497.47, E 3135945.27
Clinton Drive; See Plan of Borings (Figure 2.5)

COMPLETION DEPTH : 26.0 FT.

SURFACE ELEVATION : 40.04 FT.

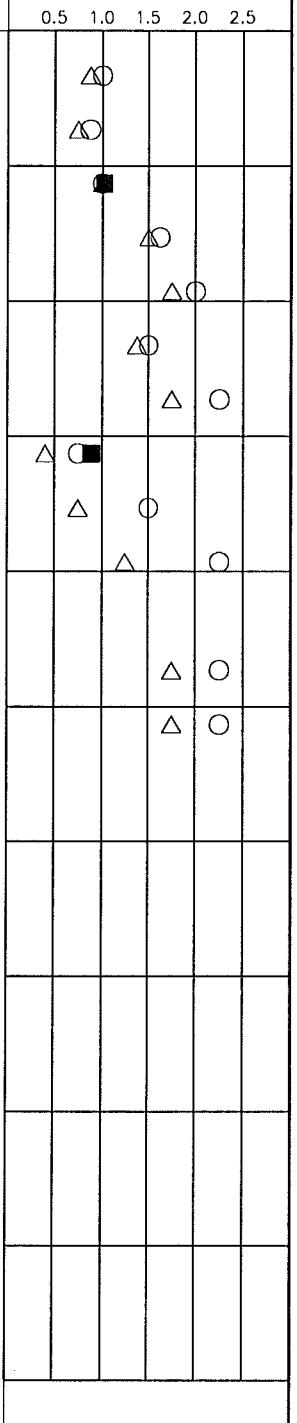
DATE : 05-23-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 26.0 FT.
WET ROTARY : -- TO -- FT.

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, %
40.0	0	3" Asphalt							
39.8		6.5" Concrete							
39.2		Stiff to very stiff gray FAT CLAY (CH) w/sand and calcareous nodules -stiff 2'-4' -w/ferrous nodules 2'-6' -very stiff 4'-8' -reddish brown and gray 6'-14' -very stiff to hard 8'-10' -very stiff 10'-12' -very stiff to hard 12'-14'			101	22			
	5					22			
				82	106	21	53	22	31
	10					31			
						24			
						25			
26.0						20			
	15	Medium stiff to stiff gray and brown SANDY LEAN CLAY (CL) w/sand seams -stiff to very stiff 16'-18' -very stiff to hard gray and yellowish brown 18'-22'		59	115	17	28	15	13
						15			
	20					12			
18.0		Very stiff to hard reddish brown and gray FAT CLAY (CH) w/calcareous nodules							
	25					23			
14.0						25			
	30								
	35								
	40								
	45								
	50								

NOTE :
See Piezometer GB-6P for water level readings.

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



DEPTH TO WATER IN BORING :
▽ : FREE WATER 1st ENCOUNTERED AT 25.0 FT. DURING DRILLING; AFTER 15.0 MIN. AT 20.7 FT.
▼ : WATER DEPTH AT 13.1 FT., HOLE OPEN TO 26.0 FT. ON 08-02-12.

Geotest Engineering, Inc.

FIGURE A-6

LOG OF BORING NO. GB-7

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845366.35, E 3136424.78
Clinton Drive; See Plan of Borings (Figure 2.5)

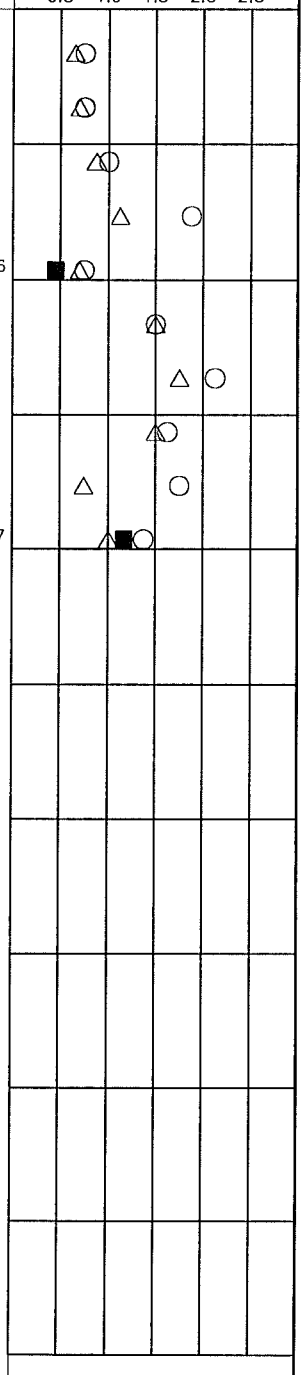
COMPLETION DEPTH : 30.0 FT.

SURFACE ELEVATION : 40.63 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 23.0 FT.
WET ROTARY : 23.0 TO 30.0 FT.

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, %
40.6	0	4.5" Asphalt							
40.3		6" Concrete							
39.8		Stiff dark gray FAT CLAY (CH)			102	20			
						23			
	5	–dark gray and gray 4'–6'				24			
		–very stiff 4'–8'							
		–reddish brown and gray w/calcoreous nodules 6'–16'				26			
	10			95	87	35	87	31	56
		–medium stiff to stiff 8'–10'				27			
		–very stiff 10'–12'							
		–very stiff to hard 12'–14'				19			
	15	–very stiff 14'–16'							
24.6		Stiff to very stiff gray SANDY LEAN CLAY (CL) w/sand seams and calcareous nodules				16			
						15			
	20	–very stiff 18'–20'		59	115	18	34	17	17
17.6		Medium dense gray SILTY SAND (SM)							
	25		24	20		21			
		–dense 28.5'–30'							
10.6	30		36			19			
	35								
	40								
	45								
	50								



DEPTH TO WATER IN BORING :
▽: FREE WATER 1st ENCOUNTERED AT 23.0 FT. DURING DRILLING; AFTER 15.0 MIN. AT 13.8 FT.
HOLE OPEN TO 30.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-7

LOG OF BORING NO. GB-8

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845368.14, E 3136927.64
Clinton Drive; See Plan of Borings (Figure 2.6)

COMPLETION DEPTH : 23.0 FT.

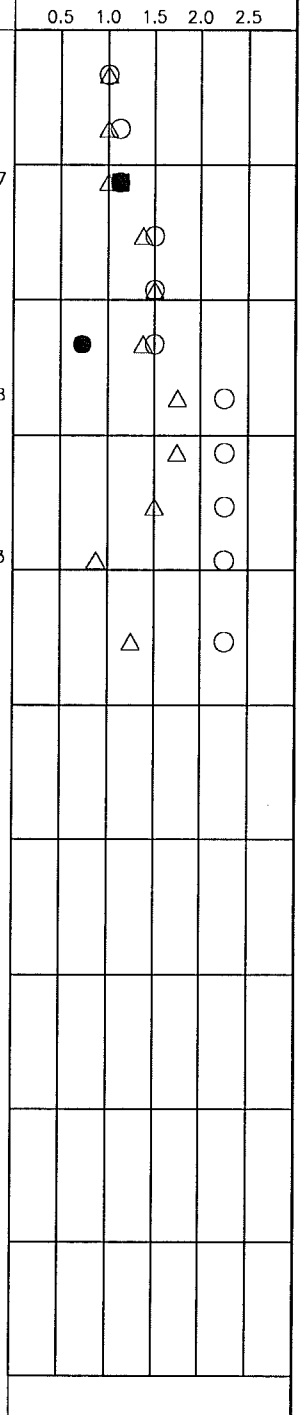
SURFACE ELEVATION : 41.30 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 23.0 FT.
WET ROTARY : -- TO -- FT.

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, %
41.3	0	4" Asphalt							
41.0		6" Concrete							
40.5		Very stiff dark gray FAT CLAY (CH)				25			
					102	24			
	5	–gray 2'–7'		87	103	22	61	24	37
		–brown and gray w/calcareous and ferrous nodules 7'–18'				20			
	10	–stiff to very stiff, slickensided 10'–12'				20			
		–very stiff to hard 12'–18'			104	22			
				99		23	62	24	38
	15					24			
23.3		Stiff to hard gray and yellowish brown SANDY LEAN CLAY (CL) w/sand seams				23			
	20			54		15	28	15	13
		–very stiff to hard w/calcareous nodules 21'–23'				14			
18.3									
	25								
	30								
	35								
	40								
	45								
	50								

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 23.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-8

LOG OF BORING NO. GB-9

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845163.73, E 3137476.13
Clinton Drive; See Plan of Borings (Figure 2.7)

COMPLETION DEPTH : 15.0 FT.

SURFACE ELEVATION : 39.73 FT.

DATE : 05-25-12

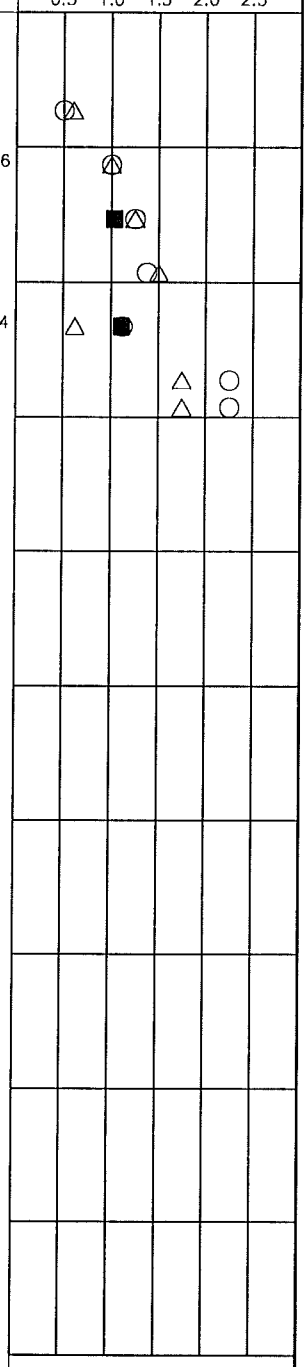
SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 15.0 FT.
WET ROTARY : -- TO -- FT.

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, %
39.7	0	2.25" Asphalt							
39.5									
38.7		10.5" Concrete							
37.9		9" Cement Stabilized Sand			88	25			
	5	Stiff gray FAT CLAY (CH) w/calcareous and ferrous nodules		91		23	60	24	36
		–very stiff gray and yellowish brown 4'–10'			96	29			
29.7	10	–w/sand and silt seams 8'–10'				20			
27.7		Stiff to very stiff brown and gray SANDY LEAN CLAY (CL) w/sand seoms		69	111	19	31	17	14
						25			
24.7	15					25			
		Very stiff to hard reddish brown and gray FAT CLAY (CH) w/colcareous nodules							
	20								
	25								
	30								
	35								
	40								
	45								
	50								

SYMBOL

SAMPLES

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 15.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-9

LOG OF BORING NO. GB-10

PROJECT : Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas

PROJECT NO. : 1140187001

LOCATION : N 13845835.23, E 3133805.50
Hirsch Street; See Plan of Borings (Figure 2.2)

COMPLETION DEPTH : 17.0 FT.

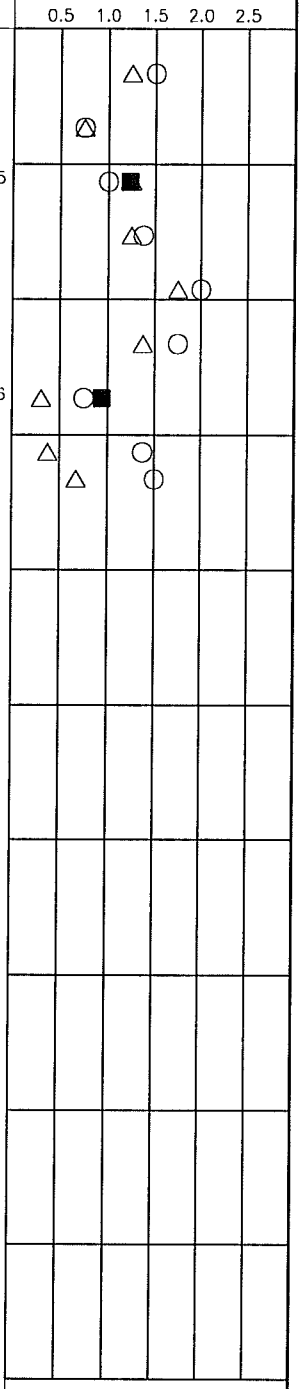
SURFACE ELEVATION : 39.84 FT.

DATE : 05-24-12

SAMPLER : Shelby Tube/Split Spoon
DRY AUGER : 0.0 TO 17.0 FT.
WET ROTARY : -- TO -- FT.

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, %
39.8	0	7" Concrete							
39.3		FILL: very stiff brown and gray fat clay w/gravel, shell and			84	32			
37.8					98	26			
	5	Stiff brown and gray FAT CLAY (CH) w/sand and calcareous nodules		81	109	20	58	23	35
		–very stiff 4'–8'				26			
	10	–reddish brown and gray 6'–12'				24			
27.8		–very stiff to hard 8'–10'				25			
		–very stiff 10'–12'		66	118	15	20	14	6
	15	Medium stiff to stiff gray and brown SILTY CLAY (CL–ML) w/sand seams, ferrous nodules and ferrous stains				16			
22.8						15			
	20	–medium stiff to very stiff 14'–16'							
		–stiff to very stiff 16'–17'							
	25								
	30								
	35								
	40								
	45								
	50								

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



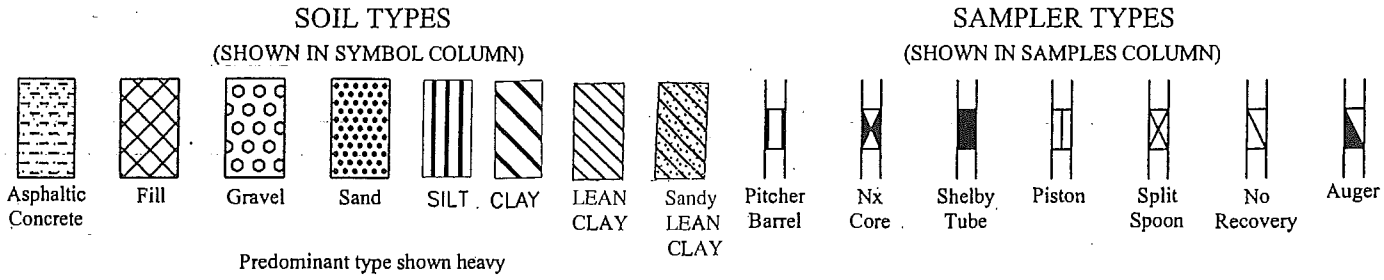
DEPTH TO WATER IN BORING :
NO GROUNDWATER ENCOUNTERED DURING DRILLING.
HOLE OPEN TO 17.0 FT. AT END OF DRILLING.

Geotest Engineering, Inc.

FIGURE A-10

Job No. 1140187001

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.

Job No. 1140187001

PIEZOMETER INSTALLATION REPORT

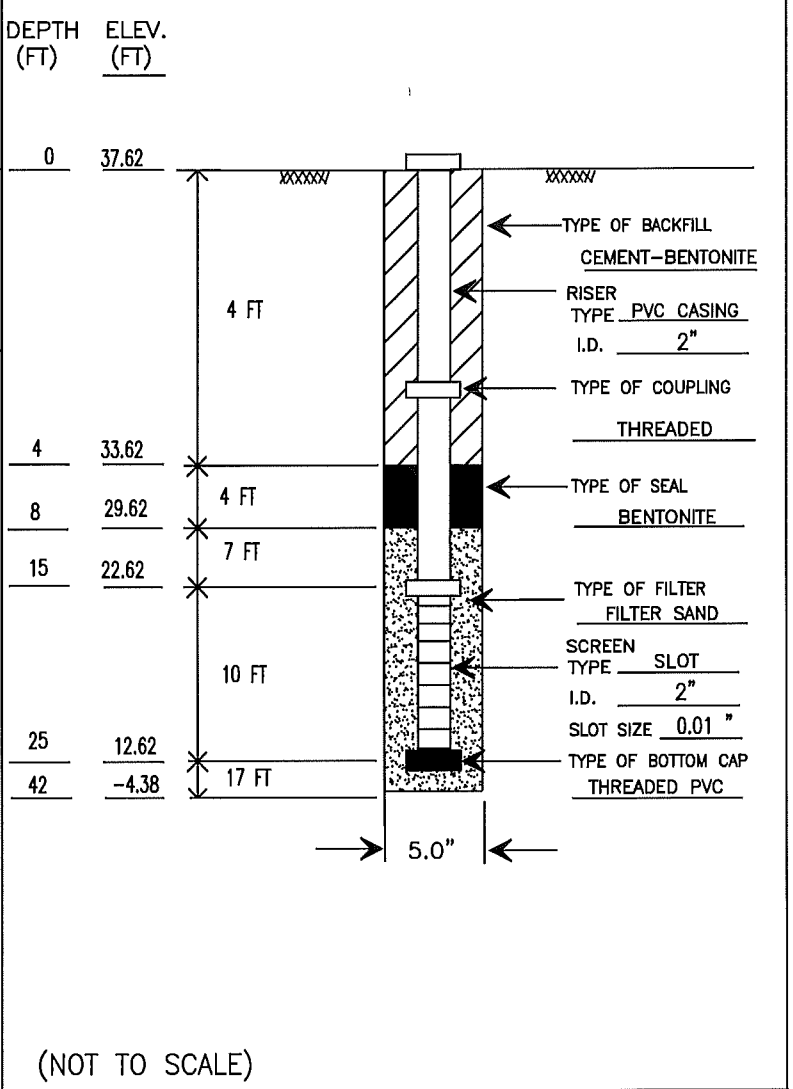
PROJECT NAME: CLINTON DRIVE RECONSTRUCTION WBS NO. N-000804-0001-3		PIEZOMETER NUMBER: GB-1P
GEOTECHNICAL CONSULTANT GEOTEST ENGINEERING, INC.	DESIGN CONSULTANT TRANSYSTEMS	HOUSTON, TEXAS

COMPLETION DATE 5-23-12
DRY AUGERED 0 TO 23 FT
WASH BORED 23 TO 42 FT
DRILLING FLUID: WATER

DEVELOPMENT DATE: 5-23-12
METHOD OF DEVELOPMENT:
BAILING

WATER LEVEL READINGS:

DATE	DEPTH (TOG)	ELEVATION
5-24-12	14.2	23.4 FT
8-2-12	14.6	23.0 FT



REMARKS:

NOTES:
1. DIMENSIONS NOMINAL UNLESS OTHERWISE NOTED
2. TOG = TOP OF GROUND

DRILLED BY: DG	STARTED: 5-23-12	NORTHING: 13845656.37 EASTING: 3133350.66
LOGGED BY: JG	COMPLETED: 5-23-12	GROUND LEVEL (MSL): 37.62 FT
CHECKED BY: NK	APPROVED BY: MB	SHEET 1 OF 1

Job No. 1140187001

PIEZOMETER INSTALLATION REPORT

PROJECT NAME: CLINTON DRIVE RECONSTRUCTION WBS NO. N-000804-0001-3		PIEZOMETER NUMBER: GB-6P
GEOTECHNICAL CONSULTANT GEOTEST ENGINEERING, INC.	DESIGN CONSULTANT TRANSYSTEMS	HOUSTON, TEXAS

COMPLETION DATE 5-23-12

DRY AUGERED 0 TO 26 FT

WASH BORED -- TO -- FT

DRILLING FLUID: WATER

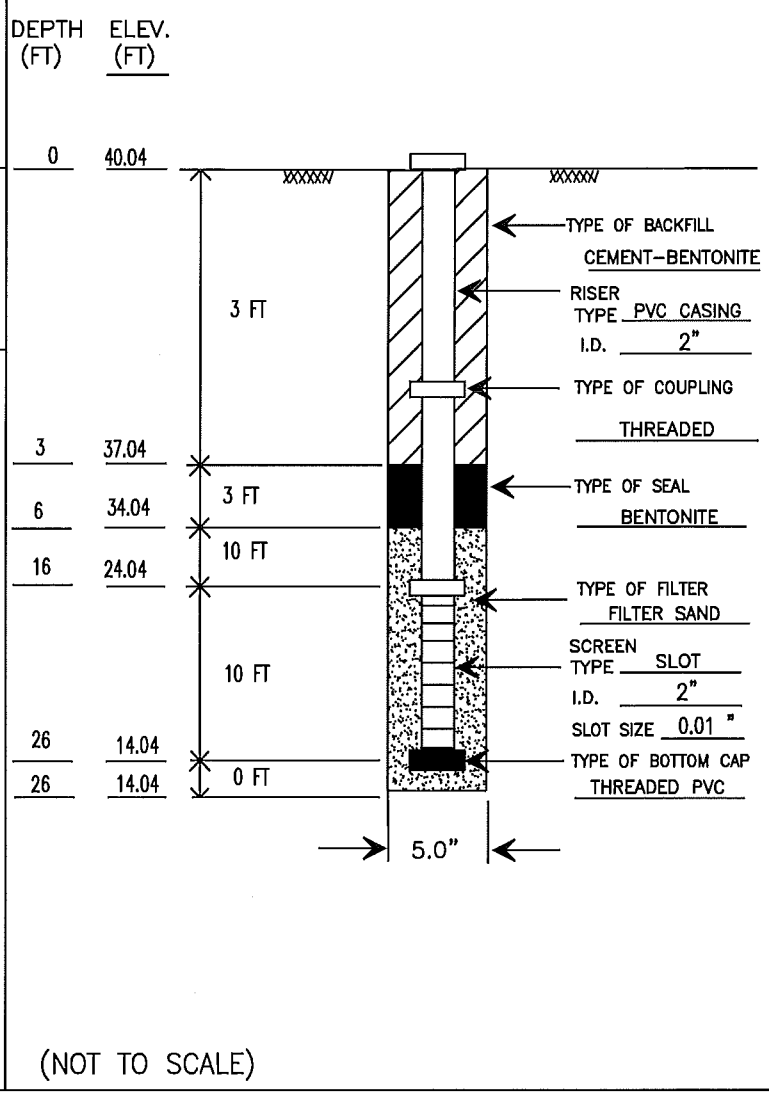
DEVELOPMENT DATE: 5-23-12

METHOD OF DEVELOPMENT:

BAILING

WATER LEVEL READINGS:

DATE	DEPTH (TOG)	ELEVATION
5-24-12	13.1	26.9 FT
8-2-12	13.1	26.9 FT



(NOT TO SCALE)

REMARKS:

NOTES:

1. DIMENSIONS NOMINAL UNLESS OTHERWISE NOTED
2. TOG = TOP OF GROUND

DRILLED BY: DG	STARTED: 5-23-12	NORTHING: 13845497.47 EASTING: 3135945.27
LOGGED BY: JG	COMPLETED: 5-23-12	GROUND LEVEL (MSL): 40.04 FT
CHECKED BY: NK	APPROVED BY: MB	SHEET 1 OF 1

APPENDIX B

	Figure
Summary of Laboratory Test Results	B-1 thru B-10
Grain Size Distribution Curves	B-11 and B-12
California Bearing Ratio Test Results	B-13a thru B-13c
Dry Density Versus CBR	B-13d

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N–000804–0001–3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB–1 (GB–1P)	3	2.0	4.0	UD		21	102								0.40	0.50	Fill
	4	4.0	6.0	UD		19		32	16	16	73				0.40	0.75	Fill
	5	6.0	8.0	UD		16									0.63	1.00	Fill
	6	8.0	10.0	UD		14	117						0.85	0.72	0.88	1.50	Lean Clay
	7	10.0	12.0	UD		14									0.50	1.63	Lean Clay
	8	12.0	14.0	UD		15		37	18	19	80				0.75	1.75	Lean Clay
	9	14.0	16.0	UD		19									1.25	1.88	Lean Clay
	10	16.0	18.0	UD		19									1.00	1.50	Lean Clay
	11	18.0	20.0	UD		19		24	15	9	74				0.38	1.00	Lean Clay
	12	23.5	25.0	SS	34	21		22	20	2	61						Sandy Silt
	13	25.0	27.0	UD		18									1.75	1.50	Fat Clay
	14	28.0	30.0	UD		19									1.75	2.00	Fat Clay
	15	33.0	35.0	UD		24	107	68	26	42	99	1.68			1.75	2.25	Fat Clay
	16	38.0	40.0	UD		20									1.75	1.88	Lean Clay
	17	40.0	42.0	UD		21									1.00	2.00	Lean Clay

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: Clinton Drive Reconstruction Project – Segment 1
WBS No. N–000804–0001–3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB–2	3	2.0	4.0	UD		21	105	64	25	39	83				1.25	1.25	Fat Clay
	4	4.0	6.0	UD		20									1.38	1.75	Fat Clay
	5	6.0	8.0	UD		20	109	64	25	39	91	0.95			0.88	1.00	Fat Clay
	6	8.0	10.0	UD		14									0.88	2.25	Sandy Lean Clay
	7	10.0	12.0	UD		13									1.00	2.25	Sandy Lean Clay
	8	12.0	14.0	UD		15									0.50	1.63	Sandy Lean Clay

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

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N–000804–0001–3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB–3A	4	2.5	4.0	SS	22	7					14						Silty Sand
	5	4.5	6.0	SS	23	8					13						Silty Sand
	6	6.5	8.0	SS	19	8											Silty Sand
	7	8.5	10.0	SS	22	13					19						Silty Sand
	8	10.0	11.5	SS	20	13											Silty Sand
	9	11.5	13.0	SS	17	14											Silty 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–3A

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB-4B	2	0.8	2.0	UD											0.50	0.75	Sandy Lean Clay
	3	2.0	4.0	UD		18									1.00	1.50	Sandy Lean Clay
	4	4.0	6.0	UD		11									0.63	1.38	Sandy Lean Clay
	5	6.0	8.0	UD		14	113	25	15	10	63		0.57	0.58	0.50	1.00	Sandy Lean Clay
	6	8.5	10.0	SS	15	8											Sand
	7	10.5	12.0	SS	15	9					9						Sand
	8	12.5	14.0	SS	14	20											Sand
	9	14.5	16.0	SS	12	22											Sand
	10	16.5	18.0	SS	36	19											Sand
	11	18.5	20.0	SS	39	19											Sand
	12	22.0	24.0	UD		18	114	53	22	31	93	2.48			1.75	2.25	Fat Clay
	13	24.0	26.0	UD		19									1.75	2.25	Fat Clay
	14	26.0	28.0	UD		23									1.75	2.00	Fat Clay

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-4B

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB-5	2	1.2	2.0	UD		16									0.88	2.13	Sandy Lean Clay
	3	2.0	4.0	UD		16	118								1.00	1.63	Sandy Lean Clay
	4	4.0	6.0	UD		14									1.00	1.63	Sandy Lean Clay
	5	6.0	8.0	UD		15	117	29	15	14	66		0.98	0.58	1.50	1.13	Sandy Lean Clay
	6	8.0	10.0	UD		17									0.75	1.00	Sandy Lean Clay
	7	10.5	12.0	SS	13	17					13						Silty Sand
	8	12.5	14.0	SS	7	18											Silty Sand
	9	14.5	16.0	SS	9	21											Silty Sand
	10	16.0	18.0	UD		17	116	27	15	12	64		0.65	1.30	0.40	0.88	Sandy Lean Clay
	11	18.0	20.0	UD		17									0.75	1.25	Sandy Lean Clay
	12	23.0	25.0	UD		18	112	54	22	32	94	1.52			1.25	1.88	Fat Clay
	13	28.0	30.0	UD		20									1.88	2.25	Fat Clay
	14	33.0	35.0	UD		22									0.63	1.75	Lean Clay
	15	37.0	39.0	UD		23		25	17	8	92				0.63	1.63	Lean Clay

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-5

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB-6 (GB-6P)	2	0.8	2.0	UD		22	101								0.88	1.00	Fat Clay
	3	2.0	4.0	UD		22									0.75	0.88	Fat Clay
	4	4.0	6.0	UD		21	106	53	22	31	82		1.02	0.43	1.00	1.00	Fat Clay
	5	6.0	8.0	UD		31									1.50	1.63	Fat Clay
	6	8.0	10.0	UD		24									1.75	2.00	Fat Clay
	7	10.0	12.0	UD		25									1.38	1.50	Fat Clay
	8	12.0	14.0	UD		20									1.75	2.25	Fat Clay
	9	14.0	16.0	UD		17	115	28	15	13	59		0.89	1.15	0.40	0.75	Sandy Lean Clay
	10	16.0	18.0	UD		15									0.75	1.50	Sandy Lean Clay
	11	18.0	20.0	UD		12									1.25	2.25	Sandy Lean Clay
	12	22.0	24.0	UD		23									1.75	2.25	Fat Clay
	13	24.0	26.0	UD		25									1.75	2.25	Fat Clay

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-6

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB-7	2	0.9	2.0	UD		20	102								0.65	0.75	Fat Clay
	3	2.0	4.0	UD		23									0.70	0.75	Fat Clay
	4	4.0	6.0	UD		24									0.88	1.00	Fat Clay
	5	6.0	8.0	UD		26									1.13	1.88	Fat Clay
	6	8.0	10.0	UD		35	87	87	31	56	95		0.45	0.72	0.70	0.75	Fat Clay
	7	10.0	12.0	UD		27									1.50	1.50	Fat Clay
	8	12.0	14.0	UD		19									1.75	2.13	Fat Clay
	9	14.0	16.0	UD		16									1.50	1.63	Fat Clay
	10	16.0	18.0	UD		15									0.75	1.75	Sandy Lean Clay
	11	18.0	20.0	UD		18	115	34	17	17	59		1.17	1.44	1.00	1.38	Sandy Lean Clay
	12	23.5	25.0	SS	24	21					20						Silty Sand
	13	28.5	30.0	SS	36	19											Silty 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-7

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N–000804–0001–3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB–8	2	0.8	2.0	UD		25									1.00	1.00	Fat Clay
	3	2.0	4.0	UD		24	102								1.00	1.13	Fat Clay
	4	4.0	6.0	UD		22	103	61	24	37	87		1.13	0.43	1.00	1.13	Fat Clay
	5	6.0	8.0	UD		20									1.38	1.50	Fat Clay
	6	8.0	10.0	UD		20									1.50	1.50	Fat Clay
	7	10.0	12.0	UD		22	104					0.73			1.38	1.50	Fat Clay
	8	12.0	14.0	UD		23		62	24	38	99				1.75	2.25	Fat Clay
	9	14.0	16.0	UD		24									1.75	2.25	Fat Clay
	10	16.0	18.0	UD		23									1.50	2.25	Fat Clay
	11	18.0	20.0	UD		15		28	15	13	54				0.88	2.25	Sandy Lean Clay
	12	21.0	23.0	UD		14									1.25	2.25	Sandy Lean Clay

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–8

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N–000804–0001–3
Houston, Texas
PROJECT NUMBER: 1140187001

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
GB–9	3	2.0	4.0	UD		25	88								0.60	0.50	Fat Clay
	4	4.0	6.0	UD		23		60	24	36	91				1.00	1.00	Fat Clay
	5	6.0	8.0	UD		29	96						1.03	0.58	1.25	1.25	Fat Clay
	6	8.0	10.0	UD		20									1.50	1.38	Fat Clay
	7	10.0	12.0	UD		19	111	31	17	14	69		1.11	0.86	0.63	1.13	Sandy Lean Clay
	8	12.0	14.0	UD		25									1.75	2.25	Fat Clay
	9	14.0	15.0	UD		25									1.75	2.25	Fat Clay

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–9

SUMMARY OF LABORATORY TEST RESULTS

GEOTEST ENGINEERING, INC.

PROJECT NAME: Clinton Drive Reconstruction Project – Segment 1
WBS No. N-000804-0001-3
Houston, Texas
PROJECT NUMBER: 1140187001

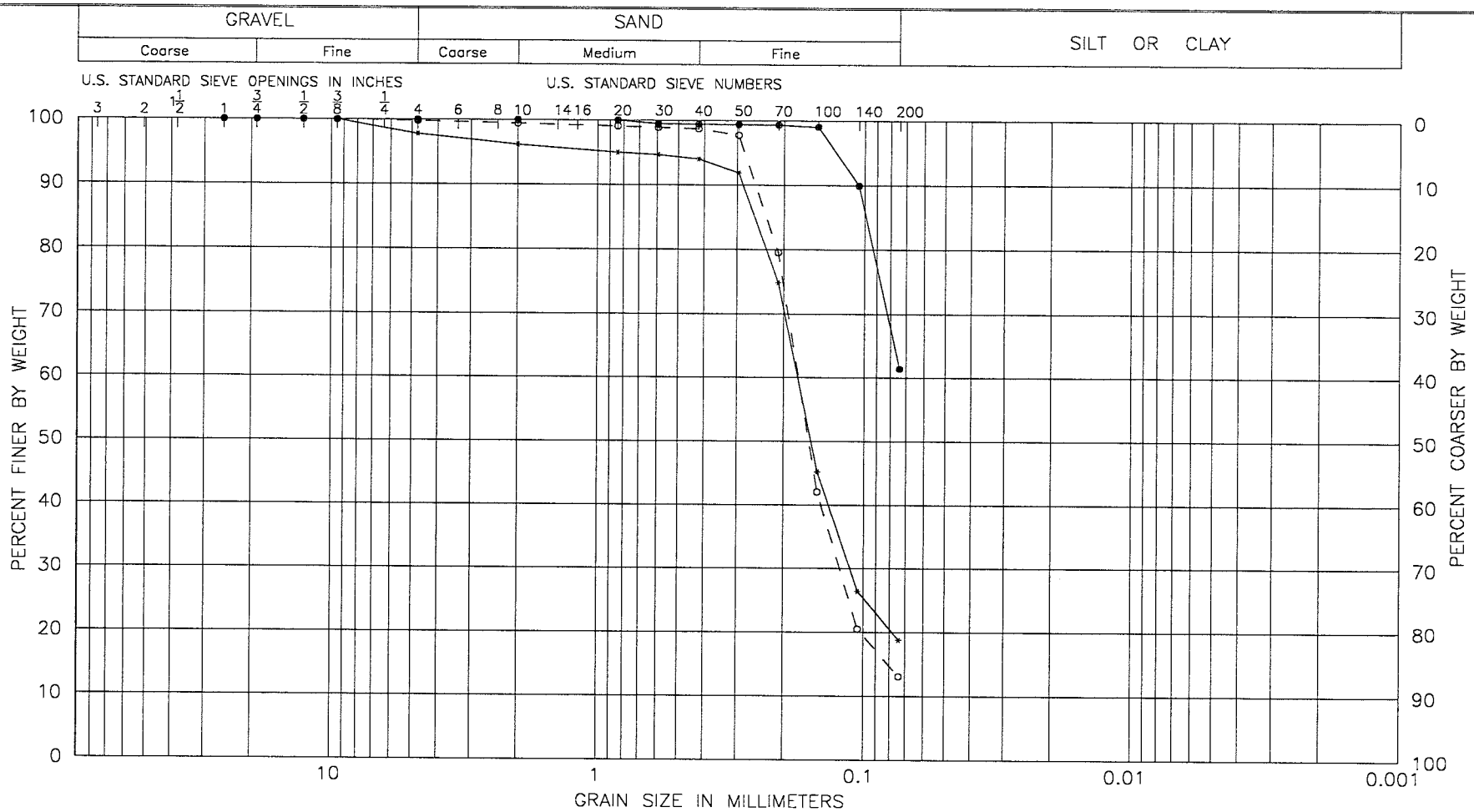
BORING NO.	SAMPLE No.	SAMPLE Depth (ft.) Top	SAMPLE Depth (ft.) Bottom	SAMPLE Type	SPT (blows/ft.)	WATER CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	PASSING NO. 200 SIEVE (%)	UNCONFINED COMPRESSION TEST Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Shear Strength (tsf)	TRIAXIAL COMPRESSION TEST (U-U) Conf. Press. (tsf)	TORVANE Shear Strength (tsf)	POCKET PENE-TROMETER Shear Strength (tsf)	TYPE OF MATERIAL
GB-10	2	0.6	2.0	UD		32	84								1.25	1.50	Fill
	3	2.0	4.0	UD		26	98								0.75	0.75	Fat Clay
	4	4.0	6.0	UD		20	109	58	23	35	81		1.23	0.43	1.25	1.00	Fat Clay
	5	6.0	8.0	UD		26									1.25	1.38	Fat Clay
	6	8.0	10.0	UD		24									1.75	2.00	Fat Clay
	7	10.0	12.0	UD		25									1.38	1.75	Fat Clay
	8	12.0	14.0	UD		15	118	20	14	6	66		0.94	1.01	0.30	0.75	Silty Clay
	9	14.0	16.0	UD		16									0.38	1.38	Silty Clay
	10	16.0	17.0	UD		15									0.68	1.50	Silty Clay

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-10

Job No. 1140187001



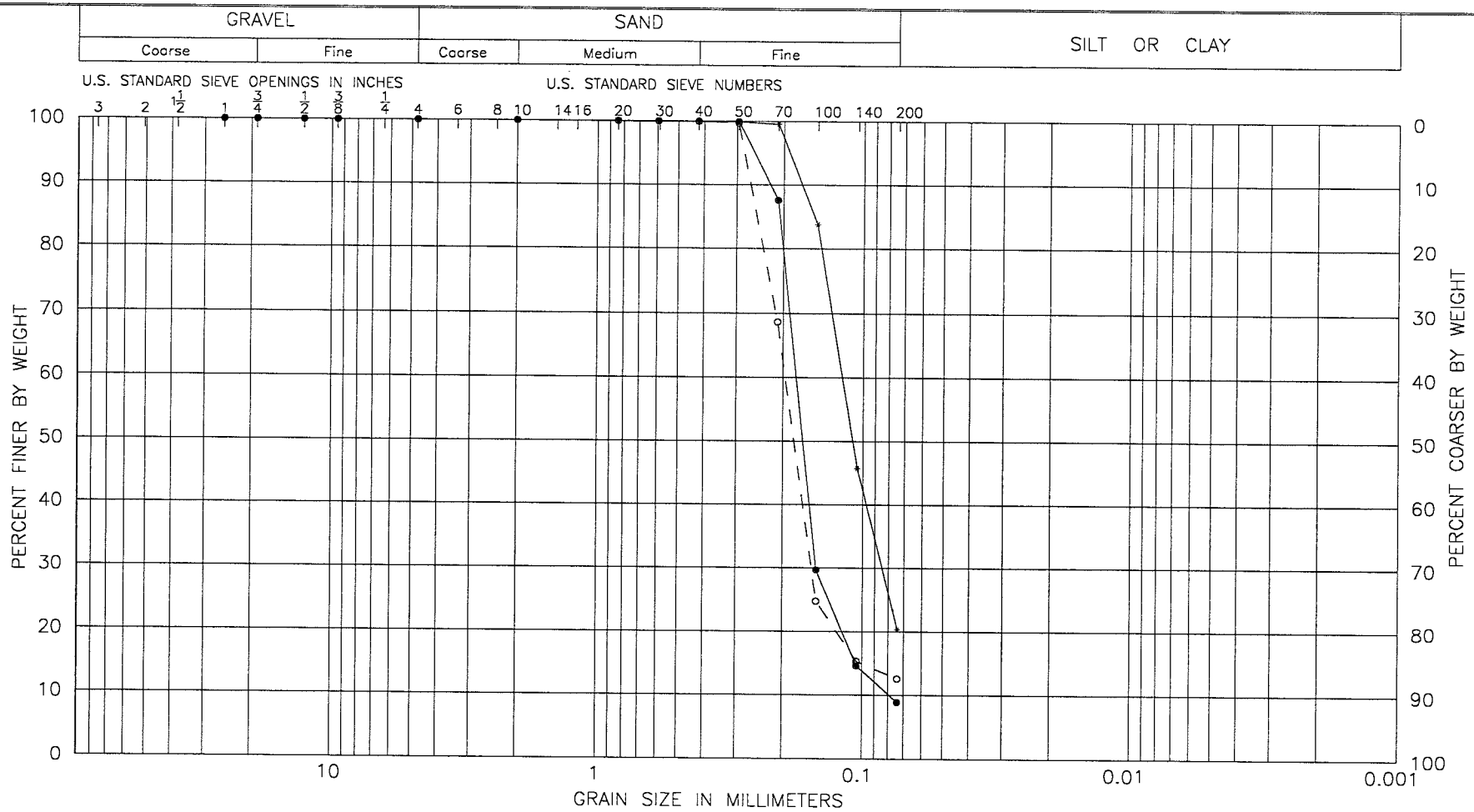
LEGEND	BORING NO.	SAMPLE NO.	DEPTH FT.	SAMPLE DESCRIPTION
●●●●●	GB−1	S−11	23.5−25.0	Brown and gray sandy silt
○⊖○⊖○	GB−3	S−3	4.5−6.0	Brown silty sand
+++++	GB−3A	S−2	8.5−10.0	Brown and gray silty sand

GRAIN SIZE DISTRIBUTION CURVES

Geotest Engineering, Inc.

FIGURE B−11

Job No. 1140187001



LEGEND	BORING NO.	SAMPLE NO.	DEPTH FT.	SAMPLE DESCRIPTION
●●●●●	GB-4B	S-6	10.5-12.0	Brown and gray fine sand w/silt
○-○-○-○	GB-5	S-6	10.0-12.0	Brown and gray silty sand
+-+-+-+	GB-7	S-11	23.5-25.0	Gray silty sand

GRAIN SIZE DISTRIBUTION CURVES

Geotest Engineering, Inc.

FIGURE B-12

CALIFORNIA BEARING RATIO (CBR) OF LABORATORY-COMPACTED SOILS

ASTM D1883

Project: : Clinton Drive Reconstruction
WBS No. N-000804-0001-3

Job No. 1140187001

Sample Location: Composite Sample GB-2, GB-6, GB-7, GB-8, GB-9 and GB-10 (0'-6')

Sample Description: Yellowish brown and gray Fat Clay w/sand seams, calcareous and ferrous nodules.

Liquid Limit 54 Plastic Limit 22 Plasticity Index 32

Method of Compaction: ☒ ASTM D698
☐ ASTM D1557

Blows per layer: 10

Sample Condition: ☒ soaked ☐ unsoaked

Dry Density before soaking 82.1 pcf

Dry Density after soaking 78.9 pcf

Moisture Content:

Before compaction	18.8 %
After compaction	19.0 %
Top 1-in layer after soaking	30.9 %
Average after soaking	30.0 %

Swell 4.04 %

Bearing Ratio 0.25 % (☒ soaked ☐ unsoaked)

Surcharge 10 lbs

CALIFORNIA BEARING RATIO (CBR) OF LABORATORY-COMPACTED SOILS

ASTM D1883

Project: : Clinton Drive Reconstruction
WBS No. N-000804-0001-3

Job No. 1140187001

Sample Location: Composite Sample GB-2, GB-6, GB-7, GB-8, GB-9 and GB-10 (0'-6')

Sample Description: Yellowish brown and gray Fat Clay w/sand seams, calcareous and ferrous nodules.

Liquid Limit 54 Plastic Limit 22 Plasticity Index 32

Method of Compaction: ☒ ASTM D698
☐ ASTM D1557

Blows per layer: 25

Sample Condition: ☒ soaked ☐ unsoaked

Dry Density before soaking 100.4 pcf

Dry Density after soaking 96.2 pcf

Moisture Content:

Before compaction	18.6 %
After compaction	18.9 %
Top 1-in layer after soaking	29.3 %
Average after soaking	26.4 %

Swell 4.23 %

Bearing Ratio 0.76 % (☒ soaked ☐ unsoaked)

Surcharge 10 lbs

CALIFORNIA BEARING RATIO (CBR) OF LABORATORY-COMPACTED SOILS

ASTM D1883

Project: : Clinton Drive Reconstruction
WBS No. N-000804-0001-3

Job No. 1140187001

Sample Location: Composite Sample GB-2, GB-6, GB-7, GB-8, GB-9 and GB-10 (0'-6')

Sample Description: Yellowish brown and gray Fat Clay w/sand seams, calcareous and ferrous nodules.

Liquid Limit 54 Plastic Limit 22 Plasticity Index 32

Method of Compaction: ☒ ASTM D698
☐ ASTM D1557

Blows per layer: 56

Sample Condition: ☒ soaked ☐ unsoaked

Dry Density before soaking 104.3 pcf

Dry Density after soaking 102.4 pcf

Moisture Content:

Before compaction	20.3 %
After compaction	19.9 %
Top 1-in layer after soaking	25.1 %
Average after soaking	21.8 %

Swell 2.16 %

Bearing Ratio 1.89 % (☒ soaked ☐ unsoaked)

Surcharge 10 lbs

Job No. 1140187001

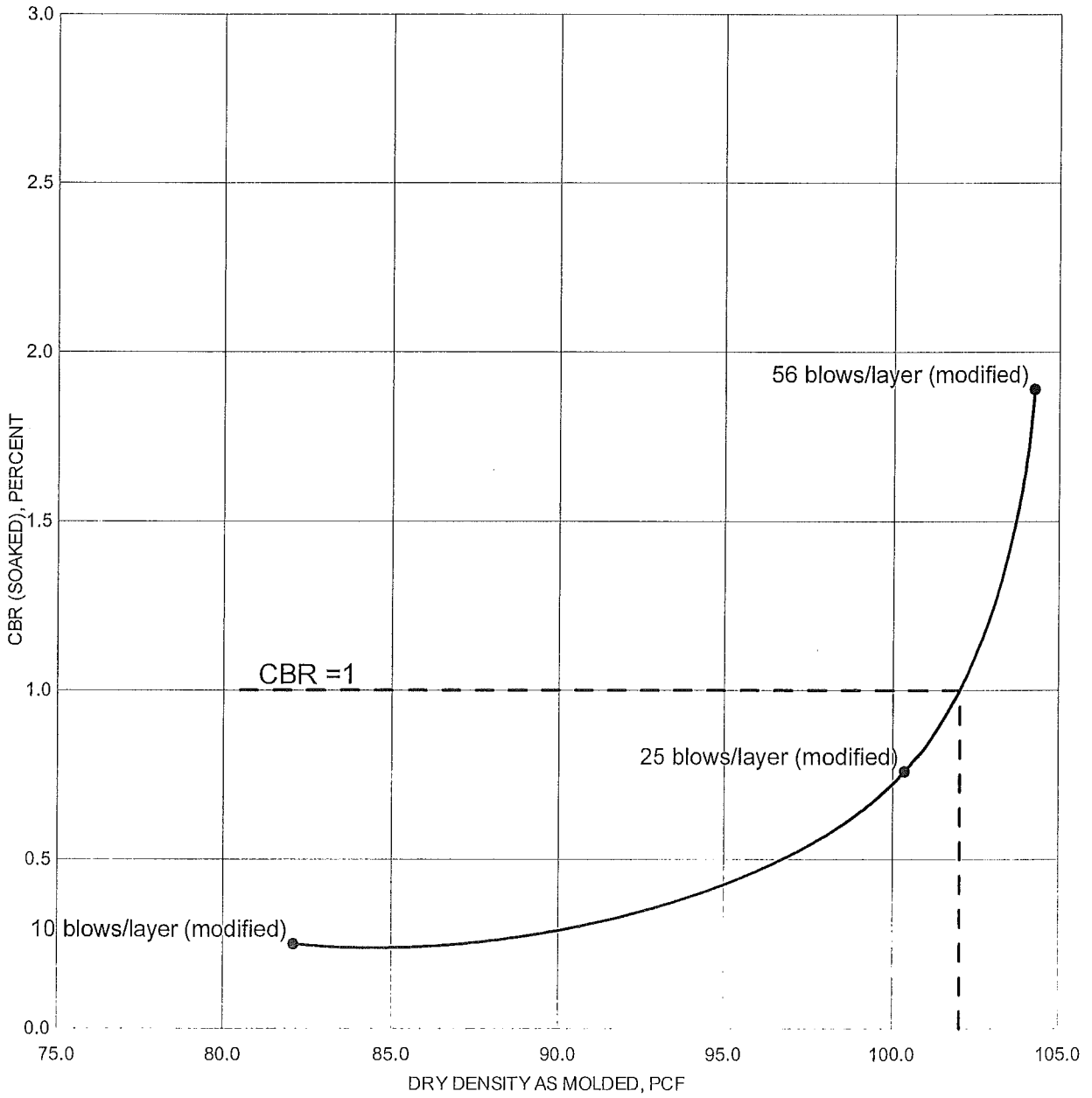
Project: Cliton Drive Reconstruction

Sample Location: Composite Sample GB-2, GB-6, GB-7, GB-8, GB-9 and GB-10

Sample Description: Yellowish brown and gray Fat Clay w/sand seams, calcareous and ferrous nodules.

Liquid Limit 54 **Plastic Limit** 22 **Plasticity Index** 32

Avarage Dry Density (pcf): 102.0



DRY DENSITY VERSUS CBR
(ASTM D 1883)
Geotest Engineering, Inc.

FIGURE B-13d

APPENDIX C

Piezometer Abandonment Reports

STATE OF TEXAS PLUGGING REPORT for Tracking #84409

Owner:	**Geotest Engineering, Inc.**	Owner Well #:	**P - 1**
Address:	**5600 Bintliff Dr. Houston , TX 77036**	Grid #:	**65-14-8**
Well Location:	**Clinton Dr. Houston , TX 77020**	Latitude:	**29° 45' 56" N**
Well County:	**Harris**	Longitude:	**095° 19' 44" W**
		GPS Brand Used:	**Lowrance XOG**

Well Type: **Monitor**

HISTORICAL DATA ON WELL TO BE PLUGGED

Original Well Driller:	**Dempsey Gearen Jr.**
Driller's License Number of Original Well Driller:	**2836**
Date Well Drilled:	**5/23/2012**
Well Report Tracking Number:	**291987**
Diameter of Borehole:	**5 inches**
Total Depth of Borehole:	**42 feet**

Date Well Plugged:	**10/20/2012**
Person Actually Performing Plugging Operation:	**Dempsey Gearen Jr.**
License Number of Plugging Operator:	**2836**
Plugging Method:	**Tremmie pipe cement from bottom to top.**
Plugging Variance #:	**No Data**
Casing Left Data:	1st Interval: **0 inches diameter, (No Data) ft to (No Data) ft** 2nd Interval: **No Data** 3rd Interval: **No Data**
Cement/Bentonite Plugs Placed in	1st Interval: **From 0 ft to 42 ft; Sack(s)/type of cement used: 3** 2nd Interval: **No Data**

Well:	3rd Interval: **No Data** 4th Interval: **No Data** 5th Interval: **No Data**

Certification Data:	The plug installer certified that the plug installer plugged this well (or the well was plugged under the plug installer's direct supervision) and that each and all of the statements herein are true and correct. The plug installer understood that failure to complete the required items will result in the log(s) being returned for completion and resubmittal.
Company Information:	**Gearen Drilling** **32126 Rochen Rd.** **Waller , TX 77484**
Plug Installer License Number:	**2836**
Licensed Plug Installer Signature:	**Dempsey Gearen Jr.**
Registered Plug Installer Apprentice Signature:	**No Data**
Apprentice Registration Number:	**No Data**
Plugging Method Comments:	**No Data**

Please include the plugging report's tracking number (Tracking #**84409**) on your written request.

Texas Department of Licensing & Regulation
P.O. Box 12157
Austin, TX 78711
(512) 463-7880

STATE OF TEXAS PLUGGING REPORT for Tracking #84408

Owner:	**Geotest Engineering, Inc.**	Owner Well #:	**P - 6**
Address:	**5600 Bintliff Dr.** **Houston , TX 77036**	Grid #:	**65-14-8**
Well Location:	**Clinton Dr.** **Houston , TX 77020**	Latitude:	**29° 45' 53" N**
Well County:	**Harris**	Longitude:	**095° 19' 15" W**
		GPS Brand Used:	**Lowrance XOG**

Well Type: **Monitor**

HISTORICAL DATA ON WELL TO BE PLUGGED

Original Well Driller:	**Dempsey Gearen Jr.**
Driller's License Number of Original Well Driller:	**2836**
Date Well Drilled:	**5/23/2012**
Well Report Tracking Number:	**291989**
Diameter of Borehole:	**5 inches**
Total Depth of Borehole:	**26 feet**

Date Well Plugged:	**10/20/2012**
Person Actually Performing Plugging Operation:	**Dempsey Gearen Jr.**
License Number of Plugging Operator:	**2836**
Plugging Method:	**Tremmie pipe cement from bottom to top.**
Plugging Variance #:	**No Data**
Casing Left Data:	1st Interval: **0 inches diameter, (No Data) ft to (No Data) ft** 2nd Interval: **No Data** 3rd Interval: **No Data**
Cement/Bentonite Plugs Placed in	1st Interval: **From 0 ft to 26 ft; Sack(s)/type of cement used: 1 1/2** 2nd Interval: **No Data**

Well:	3rd Interval: **No Data** 4th Interval: **No Data** 5th Interval: **No Data**

Certification Data:	The plug installer certified that the plug installer plugged this well (or the well was plugged under the plug installer's direct supervision) and that each and all of the statements herein are true and correct. The plug installer understood that failure to complete the required items will result in the log(s) being returned for completion and resubmittal.
Company Information:	**Gearen Drilling** **32126 Rochen Rd.** **Waller , TX 77484**
Plug Installer License Number:	**2836**
Licensed Plug Installer Signature:	**Dempsey Gearen Jr.**
Registered Plug Installer Apprentice Signature:	**No Data**
Apprentice Registration Number:	**No Data**
Plugging Method Comments:	**No Data**

Please include the plugging report's tracking number (Tracking #**84408**) on your written request.

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