FINAL
GEOTECHNICAL INVESTIGATION
FULTON ROAD IMPROVEMENTS
FROM TIDWELL ROAD TO PARKER ROAD
WBS NO. N-000542-0003-3
HOUSTON, TEXAS

SUBMITTED TO
HNTB CORPORATION
2950 NORTH LOOP WEST, SUITE 900
HOUSTON, TEXAS 77092

BY
HVJ ASSOCIATES, INC.
HOUSTON, TEXAS
AUGUST 20, 2010

REPORT NO. HG0614605
KEY MAP NOS. 413 X & 453 B

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August 20, 2010

Mr. Gregory D. Denby
HNTB Corporation
2950 North Loop West, Suite 900
Houston, Texas 77092

Re: Geotechnical Investigation
Fulton Road Improvements from Tidwell to Parker
WBS No. N-000542-0003-3
Owner: City of Houston
HVJ Proposal No. HG0614605

Dear Mr. Denby:

Submitted herein is the final report of our geotechnical investigation for the above referenced project. The study was conducted in general accordance with our proposal number HG0614605 dated October 13, 2006 (revised on March 09, 2009) and is subject to the limitations presented in this report.

We appreciate the opportunity of working with you on this project. Please read the entire report and notify us if there are questions concerning this report or if we may be of further assistance.

Sincerely,

HVJ ASSOCIATES, INC.
Texas Firm Registration No. F-000646

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The seal appearing on this document was authorized by Diaa Mounir, PE 106189 on August 20, 2010. Alteration of a sealed document without proper notification to the responsible engineer is an offense under the Texas Engineering Practice Act.

The following lists the pages which complete this report:

- Main Text – 30 pages
- Plates – 7 pages
- Appendix A – 14 pages
- Appendix B – 3 pages
- Appendix C – 4 pages
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1 EXECUTIVE SUMMARY

HVJ Associates, Inc. was retained by HNTB Corporation to provide geotechnical services for the proposed Fulton Road improvements between Tidwell Road and Parker Road in Houston, Texas. The project involves widening of approximately 1 mile of two lane roadway with deep side ditches to two 24 foot wide concrete roadway with curbs, sidewalks and underground utilities. We understand that the invert depth of the utilities will be ranging between 5 to 25 feet below existing grade. We also understand that a box culvert and signal poles will be installed near Tidwell Road intersection.

The purpose of this study is to provide design and construction recommendations for the proposed pavement replacement, underground utilities, box culvert and signal poles. This study was performed in general accordance with the City of Houston Department of Public Works and Engineering Infrastructure Design Manual, July 2009.
Based on the subsurface conditions revealed by the soil borings, the findings and recommendations of this report are summarized below:

1. The subsurface soils at the site generally comprise of soft to hard cohesive silty clays, sandy lean clays, lean clays and fat clays to the termination depth of the borings. Soft cohesive soils are encountered in borings B-2, B-8 and B-11. Cohesionless sandy silts and silty sands were encountered at the top 8 feet and below 33 feet in boring B-9; and at the top 4 feet in boring B-11. Fill material comprising of sandy lean clay and lean clay with rocks was encountered at the top 4 feet in boring B-12. Boring B-12 was moved 200 feet to the north due to existing underground utilities at a depth of 4 feet. Ferrous and calcareous nodules were encountered at various depths in most of the borings.

2. A review of surface faults was made from geologic literature and available in-house records. Based on our review, Eureka Heights fault is located at about 1 mile southwest of the project site. As the fault is not crossing the project alignment, we believe that faulting may not impact the project site. However, detailed fault study is beyond the scope of our work.

3. Groundwater was encountered in borings B-6, B-7, B-8, B-9, B-11 and B-12 at a depth ranging between 16 and 26 feet during the drilling operations. No Groundwater was encountered in other borings during drilling. Two piezometers were installed at boring locations B-9 (PZ-1) and B-11 (PZ-2). The 24-hour water level readings in the piezometers B-9 and B-11 were observed at a depth of 5 and 5.5 feet, and the 30-day water level readings were observed at a depth of 5.5 and 6.9 feet below existing grade, respectively. Piezometer installation records and groundwater level data are provided in Appendix B.

4. Recommendations for installation of utilities using both open cut and trenchless techniques are presented in this report. Augering operations should generally be in accordance with City of Houston Standard Specification, 02447.

5. The existing pavement were cored at all boring locations prior to drilling and the core data revealed that the existing pavement comprises of 1.5 to 3.5 inches of asphaltic concrete over 6 to 15 inches of stabilized base material except at boring location B-1 where the asphaltic concrete was over an 8-inch thick concrete pavement. Details of existing pavement thickness at each boring location are presented in the report.

6. We have estimated a traffic loading of 2.5 million 18-Kip equivalent single axle load (ESAL) assuming 3% trucks and 2% growth rate. This estimate is based on the 24 hour traffic volume counts for Fulton Road from Tidwell to Parker (8,052 as on September 2009) provided to us by HNTB Corporation. Based on our calculations, a 9.0-inch thick concrete pavement is appropriate for the project.

7. We have provided geotechnical foundation recommendations for box culvert and light poles at the intersection of Fulton Road and Tidwell Road as it has been requested by HNTB Corporation. Detailed design discussion is presented in section 8 and section 10 of this report.

Please note that this executive summary does not fully relate our findings and opinions. Those findings and opinions are only presented through our full report.

2 INTRODUCTION

2.1 Project Description

HVJ Associates, Inc. was retained by HNTB Corporation to provide geotechnical services for the proposed Fulton Road improvements between Tidwell Road and Parker Road in Houston, Texas. The project involves widening of approximately 1 mile of two lane roadway with deep side ditches to two 24 foot wide concrete roadway with curbs, sidewalks and underground utilities. We understand that the invert depth of the utilities will be ranging between 5 to 25 feet below existing grade. We also understand that a box culvert and signal poles will be installed near Tidwell Road intersection.

The purpose of this study is to provide design and construction recommendations for the proposed pavement replacement, underground utilities, box culvert and signal poles. This study was performed in general accordance with the City of Houston Department of Public Works and Engineering Infrastructure Design Manual, July 2009.

2.2 Geotechnical Investigation Program

The primary objectives of this study were to gather information on subsurface conditions at the site and to provide recommendations for the proposed utilities. The objectives were accomplished by:

1. Drilling twelve soil borings to depths ranging from 15 to 35 feet below the existing grade to determine soil stratigraphy and to obtain samples for laboratory testing;

2. Installing two piezometers to gain an understanding of the groundwater conditions at the site and to evaluate the potential need for dewatering during construction;

3. Performing laboratory tests to determine physical and engineering characteristics of the soils; and

4. Performing engineering analyses to develop design guidelines and recommendations.

Subsequent sections of this report contain descriptions of the field exploration, laboratory-testing program, general subsurface conditions, design recommendations, and construction considerations.

3 FIELD INVESTIGATION

3.1 Geotechnical Borings

The field exploration program undertaken at the project site was performed between March 19 and March 23, 2010. Subsurface conditions were investigated by drilling twelve soil borings to depths ranging from 15 to 35 feet below the existing grade. Boring B-12 was moved 200 feet north from its original location near Tidwell Road due to existing underground utilities at a depth of 4 feet below existing grade. The pavement was cored at all the boring locations prior to drilling and pavement thickness information was obtained.

All boreholes including the piezometers were backfilled with grout in accordance with the City guidelines and patched at the surface where applicable. The piezometers were backfilled after obtaining the 30 day water level readings. Approximate boring and piezometer locations are presented on Plates 2 of the report.

3.2 Survey Data

Based on the survey information provided to us by HNTB Corporation the location, northing, easting, ground surface elevation, station number, offset and termination depth for all the borings is presented below.

Boring No.	Location	Northing (ft)	Easting (ft)	Elevation (ft)	Station	Offset	Drilling Depth (ft)
B-1	Fulton rd	13,878,802.06	3,114,831.91	71.40	51+27.18	-15.99	15
B-2	Fulton rd	13,878,802.94	3,115,025.56	71.36	46+65.55	-7.827	15
B-3	Fulton rd	13,878,142.91	3,115,142.24	70.92	43+98.86	2.139	15
B-4	Fulton rd	13,877,653.06	3,115,346.21	70.39	38+68.35	-8.77	25
B-5	Fulton rd	13,877,208.51	3,115,536.68	70.04	33+84.74	-13.76	25
B-6	Fulton rd	13,876,749.35	3,115,742.94	69.82	28+81.39	-10.19	30
B-7	Fulton rd	13,876,273.95	3,115,966.48	69.09	23+56.22	2.618	30
B-8	Fulton rd	13,875,937.33	3,116,103.47	68.77	19+92.93	-7.785	30
B-9	Fulton rd	13,875,529.57	3,116,278.40	68.42	15+49.26	-12.16	35
B-10	Fulton rd	13,875,203.88	3,116,426.97	67.95	11+91.17	2.012	35
B-11	Fulton rd	13,874,816.99	3,116,611.37	67.40	7+62.67	16.32	35
B-12	Fulton rd	13,874,469.98	3,116,726.32	65.59	3+99.45	-19.71	35

*Notes: (-) sign indicates left offset from baseline

3.3 Sampling Methods

Soil samples were obtained continuously to a depth of 10 feet and then at 5 foot intervals thereafter to the termination depth of the borings. Cohesive soil samples were obtained with a three-inch thin-walled (Shelby) tube sampler in general accordance with ASTM D-1587 standard. Each sample was removed from the sampler in the field, carefully examined and then classified. The shear strength of the cohesive soils was estimated by a hand penetrometer in the field. Cohesionless soils were sampled with the split spoon sampler in accordance with ASTM D 1586 standard. Suitable portions of each sample were sealed and packaged for transportation to our laboratory.

Detailed descriptions of the soils encountered in the borings are given on the boring logs presented in Appendix A. A key to the soils classification and symbols used in the boring logs is also presented in Appendix A.

3.4 Water Level Measurements

Groundwater was encountered in borings B-6, B-7, B-8, B-9, B-11 and B-12 at a depth ranging between 16 and 26 feet during the drilling operations. No Groundwater was encountered in other borings during drilling. Two piezometers were installed at boring locations B-9 (PZ-1) and B-11 (PZ-2) to observe long term water level readings. Piezometer installation records and groundwater level data are provided in Appendix B.

4 LABORATORY TESTING

Selected soil samples were tested in the laboratory to determine applicable physical and engineering properties. All tests were performed according to the relevant ASTM Standards. These tests consisted of moisture content measurements; pocket penetrometer, Percent Passing No. 200 Sieve, Atterberg Limits, unconfined compression, unconsolidated undrained compression and unit dry weight tests.

The Atterberg limits and percent passing number 200 sieve tests were utilized to verify field classification by the Unified Soils Classification System, and the unconfined compression tests were performed to obtain the undrained shear strength of the soil. The type and number of tests performed for this investigation are summarized below:

Type of Test	Number of Tests
Moisture Content (ASTM D2216)	100
Atterberg Limits (ASTM D4318)	32
Percent Passing No. 200 Sieve (ASTM D1140)	32
Pocket Penetrometer	83
Unconfined Compression (UC) (ASTM D 2166)	35
Unconsolidated Undrained (UU) (ASTM D2850)	2
CBR (ASTM D 1883)	1
Proctor (ASTM D698)	1
Unit Dry Weight (ASTM D 2166/2850)	37

The laboratory test results are presented on the boring logs in Appendix A. A summary of laboratory test results except CBR and Proctor tests are provided in Appendix D. Standard Proctor and CBR test results are presented in Appendix C.

5 SITE CHARACTERIZATION

5.1 General Geology

There are two major surface geological formations that exist in the Houston area: the Beaumont formation and the Lissie formation. The Beaumont formation is a relatively younger formation generally found to the southeast of the Lissie formation. The Beaumont formation dips southeastward and extends beneath beach sand and waters of the Gulf of Mexico as far as the continental shelf. The north part of the project alignment is located in Beaumont formation (B-1 through B-8) while the south part of the project alignment (B-9 through B-12) is located in upper Lissie formation.

The upper Lissie formation is sometimes denoted as the Montgomery formation. The upper Lissie formation is heterogeneous, containing interbedded layers of clay, sand and silt. It was deposited in mid-Pleistocene times in shallow coastal river channels and flood plains.

The clay present in the formation has been preconsolidated by a process of desiccation. Numerous wetting and drying cycles have produced a network of randomly oriented and closely spaced joints, which are sometimes slickensided, that is, have a shiny appearance when exposed. The joint pattern strongly influences the engineering behavior of the soil. The sand layers vary in compactness from loose to very dense, and in thickness from a fraction of an inch to many feet due to an irregular depositional environment. Sands are generally subrounded to subangular and vary from coarse to very fine, are poorly graded, and often contain significant amounts of silt-sized particles in the sand matrix. The coastal plain in this region has a complex tectonic geology, several major features of which are: Gulf Coastal geosyncline, salt domes, major sea level fluctuations during the glacial stages, subsidence and faulting activities. Most of these faulting activities have ceased for millions of years, but some are still active.

The Beaumont formation was deposited on land near sea level in flat river deltas and in inter-delta regions. Soil deposition occurred in fresh water streams and in flood plains (as backwater marsh and natural levees). The courses of major streams and deltaic tributaries changed frequently during the period of deposition, generating within the Beaumont clay a complex stratification of sand, silt and clay deposits. Frequently, stream courses were diverted significant distances from a given point in a backwater marsh, and the water overlying the soil would evaporate since it was cut off from a drainage path. Such water, which would be highly alkaline, would precipitate large nodules of calcium carbonate (calcareous nodules) throughout the surface of evaporation. With the coming of the Second Wisconsin Ice Age, the nearby sea withdrew, leaving the formation several hundred feet above sea level and permitting the soil to desiccate. The process of desiccation compressed the clays in the formation such that they became significantly overconsolidated to a large depth. In addition to preconsolidating the soil, the process of desiccation, together with the later rewetting, produced a network of fissures and slickensides that are now closed but which represent potential planes of weakness in the soil.

5.2 Geologic Faulting

The tectonic history of the Texas Gulf Coast includes a relatively stable depositional cycle since the Cretaceous Period (about 65 million years). During this period the area has been subjected to deposition of clays, silts, and sands resulting in over 30 thousand feet of sedimentary rocks. Underlying this clastic sequence are salt formations, which have migrated upwards to produce the typical salt dome features associated with the Texas Gulf Coast. In conjunction with salt movement, dewatering and compaction of some of the deeper sediments in the basin have resulted in the development of growth faults.

A review of surface faults was made from geologic literature and available in-house records. Based on our review, Eureka Heights fault is located at about 1.0 mile southwest of the project site. As the fault is not crossing over the project alignment, we believe that faulting may not impact the project site, however, it should be noted that unmapped faults that could impact the project may exist within the project area. A detailed fault study was not within the scope of this study.

5.3 Soil Stratigraphy

Our interpretation of soil and groundwater conditions at the project site is based on information obtained at the boring locations only. This information has been used as the basis for our conclusions and recommendations. Significant variations at areas not explored by the project boring may require reevaluation of our findings and conclusions.

The subsurface soils at the site generally comprise of soft to hard cohesive silty clays, sandy lean clays, lean clays and fat clays to the termination depth of the borings. Soft cohesive soils were encountered in the upper 5 feet in borings B-2, B-8 and B-11.

Cohesionless sandy silts and silty sands were encountered at the top 8 feet and below 33 feet in boring B-9; and at the top 4 feet in boring B-11. Fill material comprising of sandy lean clay and lean clay with rocks was encountered at the top 4 feet in boring B-12. Boring B-12 was moved 200 feet to the north due to existing underground utilities at a depth of 4 feet below existing grade. Ferrous and calcareous nodules were encountered at various depths in most of the borings.

Details of the subsurface stratigraphy encountered in the borings are shown on the boring logs presented in Appendix A.

5.4 Groundwater Conditions

Groundwater was encountered in borings B-6, B-7, B-8, B-9, B-11 and B-12 at a depth ranging between 16 and 26 feet during the drilling operations. No Groundwater was encountered in other borings during drilling. Two piezometers were installed at boring locations B-9 (PZ-1) and B-11 (PZ-2). The 24-hour water level readings were observed at a depth of 5 and 5.5 feet, and the 30-day water level readings were observed at a depth of 5.5 and 6.9 feet below existing grade, respectively. Piezometer installation records and groundwater level data are provided in Appendix B.

It should be noted that groundwater levels determined during drilling may not accurately reflect the true groundwater conditions, and therefore should only be considered as approximate. Groundwater levels measured in open standpipe piezometers, on the other hand are more accurate; however, these readings will fluctuate seasonally and in response to rainfall. Other factors that might impact piezometric groundwater levels include leakage from existing water lines, storm sewers and/or sanitary sewers.

6 UTILITY DESIGN CRITERIA AND RECOMMENDATIONS

6.1 General

The project involves installation of waterlines, storm sewer and sanitary sewer along Fulton Road between Parker Road and Tidwell Road in Houston, Texas. We understand that invert depth of the utilities will vary from 5 to 25 feet below existing grade. Our Analyses and recommendations for the installation of utilities using both open cut and auguring techniques are presented below.

6.2 Geotechnical Parameters

Geotechnical design parameters are presented in the following table. Design parameters given in the table are based on field and laboratory test data obtained at boring locations drilled for the proposed utilities at the approximate invert depth.

It must be noted that because of the nature of the soil stratigraphy at this site, parameters at locations away from the borings may vary substantially from values reported in the table.

Boring No.	Street Name	Approximate Invert Depth (ft)	Soil Description at Invert Depth	Total Unit Weight (pcf)	Undrained Shear Strength (psf) / or Friction Angle (deg)	Allowable Bearing Pressure (psf)	E'n, Long Term (psi)
B-1	Fulton Road	5-10	Stiff Sandy Lean Clay/ Lean Clay	134	1000	1600	300

Boring No.	Street Name	Approximate Invert Depth (ft)	Soil Description at Invert Depth	Total Unit Weight (pcf)	Undrained Shear Strength (psf) / or Friction Angle (deg)	Allowable Bearing Pressure (psf)	E'n, Long Term (psi)
B-2	Fulton Road	5-10	Stiff to Very Stiff Sandy Lean Clay	122	1200	2000	600
B-3	Fulton Road	5-10	Firm to Very Stiff Lean Clay/ Sandy Lean Clay	122	1000	1600	300
B-4	Fulton Road	5-10	Stiff to Very Stiff Sandy Lean Clay/ lean Clay	121	1500	2500	600
		10-15	Very Stiff Lean Clay w/ sand	120	2500	4000	1000
		15-20	Very stiff to Hard Lean Clay/ Sandy Lean Clay	134	3000	4500	1000
B-5	Fulton Road	5-10	Stiff to Very stiff Sandy Lean Clay	125	1500	2500	600
		10-15	Very stiff Sandy Lean Clay/ Lean Clay	133	3000	4500	1000
		15-20	Very Stiff Lean Clay w/ sand	123	2200	3600	1000
B-6	Fulton Road	5-10	Firm to Stiff Sandy Lean Clay	135	600	1000	300
		10-15	Very Stiff Sandy Lean Clay / Lean Clay	118	2400	4000	1000
		15-20	Very stiff Sandy Lean Clay / Fat Clay	119	3000	4500	1000
		20-25	Firm to stiff Fat Clay / Lean Clay	123	700	1100	300
B-7	Fulton Road	5-10	Firm to Very Stiff Sandy Lean Clay	136	700	1100	300
		10-15	Very Stiff Sandy Lean Clay/ Lean Clay	121	3000	4500	1000
		15-20	Very Stiff Lean Clay w/sand	122	3000	4500	1000
		20-25	Very Stiff Lean Clay/ Fat Clay	122	2000	3300	600
B-8	Fulton Road	5-10	Stiff Sandy Lean Clay	122	1500	2500	600
		10-15	Very Stiff to Hard Sandy Lean Clay	138	3000	4500	1000
		15-20	Very Stiff Sandy Lean Clay/ Fat Clay	135	2500	4000	1000
		20-25	Firm to Stiff Fat Clay/ Sandy Clay	135	900	1500	300

Boring No.	Street Name	Approximate Invert Depth (ft)	Soil Description at Invert Depth	Total Unit Weight (pcf)	Undrained Shear Strength (psf) / or Friction Angle (deg)	Allowable Bearing Pressure (psf)	E'n, Long Term (psi)
B-9	Fulton Road	5-8	Loose Sandy Silt	115	28	1900	300
		8-10	Stiff Sandy Lean Clay	134	1100	1800	600
		10-15	Very Stiff Sandy Lean Clay/ Lean Clay	126	2600	4300	1000
		15-20	Very Stiff Lean Clay w/ sand	126	2600	4300	1000
		20-25	Very Stiff to hard Lean Clay w/ sand	124	3000	4500	1000
		25-30	Very Stiff Lean Clay w/ sand	124	3000	4500	1000
B-10	Fulton Road	5-10	Stiff Silty Clay/Sandy Lean Clay	120	1100	1800	600
		10-15	Stiff Sandy Lean Clay/ Lean Clay	123	100	1800	600
		15-20	Very Stiff Lean Clay	123	3000	4500	1000
		20-25	Very Stiff to hard Lean Clay w/ sand	128	3000	4500	1000
		25-30	Stiff Lean Clay w/ sand	120	1300	2100	600
B-11	Fulton Road	5-10	Firm Silty Clay/Sandy Lean Clay	133	700	1100	300
		10-15	Stiff to Very Stiff Sandy Lean Clay	138	1300	2100	600
		15-20	Very Stiff Sandy Lean Clay/ Lean Clay	138	3000	4500	1000
		20-25	Very Stiff Lean Clay w/ sand/ Sandy Clay	138	2200	3600	1000
		25-30	Very Stiff Sandy Lean Clay/ Lean Clay	138	2200	3600	1000
B-12	Fulton Road	5-10	Firm to Stiff Lean Clay w/ sand	129	1000	1600	300
		10-15	Stiff to Very Stiff Sandy Lean Clay	135	1300	2100	600
		15-20	Very Stiff Sandy Lean Clay/ Fat Clay	128	3000	4500	1000
		20-25	Very Stiff Fat Clay/ Sandy Lean Clay	128	3000	4500	1000
		25-30	Very Stiff Sandy Lean Clay/ Fat Clay	128	3000	4500	1000

The values shown in the above table represent our interpretation of the soil properties based on the available laboratory and field test data.

Use of the soil properties shown above may or may not be appropriate for a particular analysis, since choice of design parameters often depends on whether total or effective stress analysis is used, rate of loading, duration of loading, geometry of loaded area, and other factors. The total unit weight values shown above represent our interpretation of soil unit weight at natural moisture content. The undrained shear strength and allowable bearing pressure values represent our interpretation of the shear strength in clay soils based primarily on the results of unconfined compression tests and hand penetrometer tests. The allowable bearing pressures include a factor of safety of three.

Pipe Design. The loads imposed on underground pipes depend principally upon the method of installation, the weight of overburden soils, roadway traffic load, and loads due to existing surface structures. For design of rigid pipes installed using open-cut excavation methods, loads due to overburden and traffic can be determined from Plate 3. The traffic load applied to the pipe can be calculated using 85% of wheel load with an impact factor of 1.5 for one foot of soil cover, 50% of wheel load with an impact factor of 1.35 for 2 feet of cover, and 30% of wheel load with an impact factor of 1.15 for 3 feet of cover.

This results in a total design traffic load on the pipe or box culvert of about 1.28, 0.68 and 0.35 times the wheel load for 1, 2 and 3 feet of cover, respectively. For pipes or box culverts with four or more feet of cover, the traffic loads may be taken as a surcharge equivalent to 250 psf.

The design of flexible pipes requires the modulus of soil reaction of the native soil (E_n') in the trench wall as input. The E_n' values are based on empirical relationships to the soil consistency as defined by unconfined compression tests for cohesive soils. E_n' values for the native soils are presented in the above table. The E_n' values for short-term conditions in cohesive soils may be assumed to be 1.5 times the long-term values. These values are based on the soil data obtained at the boring locations only and may be used for the noted invert depth zone.

Pipe Bedding. Waterlines installed using open-cut trenches should be placed using City of Houston Drawing No. 02317-04. This drawing specifies that the bottom should be dry before placement of pipe. The sanitary sewer may be installed according to City of Houston Standard Drawing Nos. 02317-01, 02317-02 or 02317-03. The storm sewer may be installed using City of Houston standard bedding details as outlined on Standard Drawing Nos. 02317-02 or 02317-03.

If needed, we recommend groundwater control in accordance with Section 01578 of City of Houston Standard Specifications be implemented to achieve stable trench conditions and satisfactory foundation base.

The excavations should be performed with equipment capable of providing a relatively clean bearing area. Stable soils are essential to provide a strong base during construction. In addition, stable soils enhance trench bottom stability, support for bedding compaction, and minimize possible pipe settlement. Whenever soft foundation soils are encountered during trench excavation, we recommend over excavating 3 feet below the base of the foundation and replacing with on-site soils compacted to at least 95% of maximum dry density in loose lifts not exceeding 8 inches.

Trench Backfill. Trench backfill for water lines, sanitary sewer, and storm sewer should be in accordance with Section 02317, Excavation and Backfill for Utilities, of the City of Houston Standard Specifications, July 2009. Backfill around the storm sewers, including manholes and other underground structures, should be in accordance with the provisions that are explained in the City of Houston Standard Details on Drawing Nos. 02317-02 and 02317-03. The backfill for the sanitary sewer lines should be in accordance with Drawing Nos. 02317-01, 02317-02 or 02317-03. The water line backfill should be in accordance with Drawing No. 02317-04.

Pipe embedment (bedding, haunching, and initial backfill) for water lines may consist of bank run sand, concrete sand, gem sand, pea gravel, crushed limestone, cement stabilized sand, or Class I, II and III embedment materials as specified in City of Houston Standard Specification Sections 02320 and 02321.

For pipes that will be located under streets or within one foot of streets and curbs, pipe embedment should extend to a minimum of 12 inches above the top of pipe and should be compacted to 95% of maximum dry density determined by ASTM D698 as outlined in City of Houston specification 02317. However, the backfill up to 12 inches above the top of the pipe should be compacted carefully so as to prevent structural damage to the pipe.

Trench zone backfill is that portion of trench backfill that extends vertically from the top of pipe embedment up to pavement subgrade or up to final grade when not beneath pavement. Trench zone backfill for water lines may consist of bank run sand, select fill, or random backfill material as specified in City of Houston Standard Specification Section 02320. Trenches that are located partially within the limit of one foot from streets or curbs should be uniformly backfilled according to the paved area criteria. Backfill material may consist of in-situ soils or imported select fill. Imported select fill should consist of sandy clay with a liquid limit less than 40 and a plasticity index between 8 and 20. Excavated material fulfilling these criteria may be used as backfill.

Fill material should be placed in loose lifts not exceeding eight inches, and should be compacted to 95 percent of the standard Proctor maximum dry density as determined by ASTM D 698.

6.3 Pressures on Primary and Permanent Liners

It is customary to place a primary liner immediately after excavation so that the ground is always supported. A permanent liner is then placed some time after the installation of the primary liner. The annular space between the liners is then filled with grout. The tunnel liners should be designed to support not only the ground loads but also the construction loads. Pressures on the liner with an example calculation of liner load due to earth and traffic load are presented on Plate 3. Deformation of the liner in the horizontal and vertical diameters can be expected due to soil-liner interaction. Experience with liner distortion in the Houston area suggest values in the range of 0.75 percent difference in length of the vertical and horizontal diameters; with shortening of the vertical diameter in most cases. To the extent that the tunnel liner reduces the soil deformation due to the rigidity of the liner, bending moments will be developed in the liner. The lining will be adequate with respect to bending if it can be deformed, without overstress, by an amount equal to the expected change in diameter.

Buckling of the liner can be a problem if non-uniform support of the liner occurs. This sometimes happens if a local overcut situation occurs during tunneling which is not properly backfilled. Buckling can also occur if the liner is used as reaction for the tunneling equipment, and the tunneling equipment unevenly applies thrust loads.

6.4 Thrust Force Design Recommendations

Piping System Thrust Restraint. Unbalanced thrust forces will be developed in water lines due to changes in direction, cross-sectional areas, or when the pipe is terminated. These forces may cause joints to disengage if not adequately restrained.

There will be a slight loss of head due to turbulence in bends in the pipes. This loss will cause a pressure change across the bend, but it is usually small enough to be neglected.

The thrust force may require more reaction than is available just from the pipe bearing against the backfill. In order to prevent intolerable movement and overstressing of the pipe, suitable buttressing

should be provided. In general, thrust blocks, concrete encasement, restrained joints and tie rods are common methods of providing reaction for the thrust restraint design. The thrust restraint design provisions described in this section are based on the American Water Works Association Manual M9 (1979)

Concrete Pressure Pipe.

Various types of thrust restraint systems are used depending on type of pipes and installation conditions. The force diagram shown on Plate 4 illustrates the thrust force generated by flow in a bend in the pipe. The equations for computing this thrust force are also given on this figure.

An example computation of a thrust force generated by flow at a bend in a pipe for a surge pressure of 150 psi and a bend angle of 90 degrees is also presented on Plate 4.

Frictional Resistance. The unbalanced force produced by grade and alignment changes can also be resisted by friction on the pipe. The length of pipe will be formed by tying or welding joints together for the distance required to develop adequate capacity or by encasing the pipe in concrete. The resisting frictional force, F_R is computed as

$$F_R = f\,(2W_e+W_w+W_p)$$

Where:

f = Coefficient of friction between pipe and soil

W_e = Weight of soil over pipe in lb/ft

W_w = Weight of contained water in lb/ft

W_p = Weight of pipe in lb/ft

The friction value depends on the material in contact with the pipe and the soil used in the backfill around the pipe. For pipe surrounded by compacted sand or crushed stone, the friction between the pipe and soil may be based on a friction angle of 30 degrees. The allowable coefficient of friction, f, of 0.28, 0.23 and 0.18 can be used for concrete, steel and PVC pipes, respectively. This value includes a factor of safety of 2.0. The weight of soil above the pipe will depend on the soil unit weight and the pipe depth. For compacted soils used for backfill, a total unit weight of 125 pcf can be used. Tied joints are used to transmit thrust across joints.

These ties may be welded or harnessed joints. Joints may be welded in the field in order to transmit the thrust involved. Information concerning types of harnessed joints available and size and pressure limitations can be obtained from the pipe manufacturers.

6.5 Utilities Installed by Trenchless Technique

We understand that trenchless construction methods may be used to install utilities at some locations along the alignment. The results of our soil borings indicate that both cohesive and cohesionless soils will be encountered at the pipe invert depth. It should be noted that due to variability in soil deposits any tunneling operations along the projected alignments could result in varying degrees of mixed face tunneling conditions where several types of soil material may be encountered at the tunneling face. Although the clays are typically stable, face stability problems can occur when water-bearing or soft soils are encountered. Even with dewatering systems operating, unstable flowing situation may occur.

Geotechnical Properties. Recommended ranges of engineering design soil parameters for the cohesive and cohesionless soils that may be encountered in the pipe zone are summarized below.

For cohesive soils:

Total Unit Weight	116 to 132 pcf
Submerged Unit Weight	53.6 to 69.6 pcf
Coefficient of Earth Pressure, K_o	1.0
Undrained Shear Strength	200 to 1600 psf
Average Undrained Shear Strength	1400 psf
Poisson's Ratio	0.45
Young's Modulus	3000 to 14000 psi

For cohesionless soils:

Total Unit Weight	115 to 125 pcf
Submerged Unit Weight	52.6 to 62.6 pcf
Coefficient of Earth Pressure, K_o	0.5
Effective Angle of Internal Friction	30° to 32°
Poisson's Ratio	0.35
Young's Modulus	2500 to 8000 psi

Pipe Design. For pipes to be installed by tunneling techniques, whereby sections of pipe are jacked forward against the surrounding soil, pipes should be designed to resist significant bending moments, along with the jacking forces exerted on the pipe during installation. These loads generally exceed the overburden pressures that are typically determined based on the prism earth load to the ground surface, plus hydrostatic pressure and surcharge loads as shown on Plate 3. Therefore, pipes designed to resist construction loads during tunneling operations should have adequate strength for most long-term overburden and traffic loads.

During design, allowance should be made for any external loads, other than soil loads, which may be exerted on the pipe. These include loads from foundations for structures located near the water line and any possible future excavation to be performed near the water line. Much of the stability of the waterlines is due to the presence of relatively uniform stress conditions in the soil around the pipe. Relief of the earth loads on one side of the utilities due to subsequent adjacent excavation could cause an overstress of the pipe.

Influence of Tunneling on Adjacent or Overlying Structures. The construction of every tunnel in soils is associated with a change in the state of stress in the ground and with the corresponding strains and displacement. In particular, some degree of settlement of the overlying ground surface is always induced. If such settlement, referred to as subsidence, is excessive, it may cause damage to structures, roads and services located above the tunnel.

It should be noted that the existing foundation of the nearby structures and buried portion of existing pipelines within the zone of influence of the tunnel might be subject to possible distress due to tunnel-induced settlement.

While the recommendations we are providing intend to reduce the settlement and distress to these structures and pipelines within the zone of influence, they still should be monitored before and for a period after tunneling operations are completed. Generally, settlements due to tunneling are not anticipated after the tunneling operations are completed.

In order to minimize settlement due to tunneling operations the contractor should use well-established techniques and provide temporary support, by advancing the primary liner continuously, as tunneling progresses. No voids should be allowed between any temporary support and the surrounding soils, and with that purpose the injection of cement grout should be considered if it is deemed necessary to fill the voids.

7 UTILITY CONSTRUCTION CONSIDERATIONS

7.1 General

This section is intended to address issues that might arise during construction. Our recommendations are intended for use as guidelines in dealing with particular soil conditions. The topics addressed in this section include trench excavation stability, groundwater control, open-cut construction and augering technique construction considerations.

The recommendations contained herein are not intended to dictate construction methods or sequences. Instead they are provided solely to assist designers in identifying potential construction problems related to excavation, based upon findings derived from sampling.

Depending upon the final design chosen for the project, the recommendations may also be useful to personnel who observe construction activity. Prospective contractors for the project must evaluate potential construction problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

7.2 Excavation Considerations

Excavations should satisfy two requirements. First, the soils above final grade must be removed without disturbing the soil below excavation grade, which will support constructed facilities. Second, the sides of the excavation must be stable to prevent damage to adjacent streets and facilities as a result of either vertical or lateral movements of the soil.

In addition, a satisfactory excavation procedure must include an adequate construction dewatering system to lower and maintain the water level at least a few feet below the lowest excavation grade.

Excavation Stability. Excavations shall be shored, laid back to a stable slope or some other equivalent means may be used to provide safety for workers and adjacent structures. Earth pressures for braced excavations are presented on Plates 5A & B. Assessment of the need for excavation sloping, use of trench boxes or other measures required to provide a stable excavation, and the use of appropriate construction practices and/or equipment is the contractor's responsibility.

The following comments are intended to represent common solutions to stability problems encountered in similar soil conditions in the Houston area, and may not be construed as excavation system design recommendations. The excavation operations shall be performed in accordance with 29 CFR Part 1926 subpart P, as amended, including rules published in the Federal Register, Vol. 54, No. 209, dated October 31, 1989, as a minimum. In addition, the provisions of legislation enacted by the Texas Legislature and City of Houston should be satisfied.

Boring No.	Street Name	OSHA Soil Type			
		Depth of Trench (ft)			
		0 – 5	5 – 10	10-15	15-20
B-1	Fulton Road	B	B	B	-
B-2	Fulton Road	C	B	B	-
B-3	Fulton Road	B	B	B	-
B-4	Fulton Road	B	B	B	B
B-5	Fulton Road	B	B	B	B
B-6	Fulton Road	C	B	B	C
B-7	Fulton Road	C	B	B	B
B-8	Fulton Road	C	B	B	B
B-9	Fulton Road	C	C	B	B
B-10	Fulton Road	B	B	B	B
B-11	Fulton Road	C	B	B	B
B-12	Fulton Road	C	B	B	B

In general, it is our opinion that the pressure distribution (for braced walls) should be used for design of sheeting or trench boxes.

To reduce the potential for ground movement adjacent to the top of the excavation, the bracing should be preloaded in stages as the excavation is deepened. The detailed earth pressure diagram for clay is presented on Plate 5A and for sand/ silt is presented on Plate 5B.

The planned construction will be performed along alignments near existing utility installations (either crossing or paralleling the new alignments). The contractors should be aware of potential excavation stability problems while working in the vicinity of old trenches and the excavation system should be designed to accommodate this weak material (trench backfill).

The vertical walls of excavations should be located a safe distance from existing utilities in order to prevent movement in the soil mass behind the excavation that may adversely affect the utilities. We recommend that the horizontal distance should be 4 feet for excavation depths of up to 10 feet.

7.3 Auger Construction Considerations

In augering, a launch pit is excavated and a horizontal boring rig is used to excavate an unsupported bore distance of up to 300 to 400 feet to a receive pit. Once the bore is excavated, dragging a tool through the bore cleans it, and then the pipe is dragged through the bore. This technique is commonly used in the Houston area for installation of small diameter pipes at depths above the groundwater table. Augering operations should generally be in accordance with City of Houston Standard Specification, 02447.

Bore Stability. In auger construction, where the bore must stand open unsupported for a period of several hours, the structure of the soil is very important. Augering operations have encountered difficulties such as slowed production rates, ground surface settlement above the bore, and bore collapse in some soil conditions in the Houston area.

We do not recommend augering in unstable soils or in soils below the water table without providing casing to prevent running ground condition. Firm to very stiff clay soils are generally suitable for augering, however, the secondary structure of the soil is an important consideration.

Where a blocky, slickensided, or fissured condition is noted on the boring logs, the clay soil may slough excessively from the bore walls. This will lead to an excessive number of cleaning passes to allow passage of the pipe, and it will result in formations of large voids around the pipe.

Collapse of these voids after pipe placement commonly results in noticeable settlement of the ground surface above the bore.

Loss of Ground. A properly designed and controlled augering operation can eliminate or reduce immediate soil movement and subsidence to a tolerable level. Nevertheless, some ground loss should be expected during any tunnel construction operation. With good construction techniques, ground loss can be held to acceptable levels. Generally, tunnels constructed beneath pavement and buried utilities can be expected to create a loosened subgrade or bedding condition which may lead to subsequent deformations.

Large ground loss can result from uncontrolled flowing ground. The potential for such ground loss exists wherever water-bearing sands or silts are encountered along the alignment. Careful dewatering of such layers will reduce the potential for development of flowing conditions, but local experience shows that complete dewatering is difficult to achieve as discussed in a later section.

Ground Control and Improvement. We recommend that tunnels be constructed using techniques that provide positive support to the soil during augering operations. Several measures are available to overcome adverse ground conditions including groundwater lowering and grouting. We expect that groundwater at some locations may be encountered in tunnels that are excavated below 10 feet. Groundwater control and dewatering recommendations are provided in Section 7.7 of this report.

7.4 Auger Pit Construction Considerations

It is our understanding that auger pits constructed for augering operations will vary in size depending on whether the pit is a drive or receive pit, the size of machine, and the length of auger pit. Pit construction should be in accordance with City of Houston Standard Specification 02447. Pit should be backfilled in accordance with City of Houston Standard Specification 02317. Bedding and backfill for water lines through auger pits should be in accordance with City of Houston Drawing No. 02447-01.

Pit Excavation Stability. Pit excavations shall be shored or some other equivalent means may be used to provide safety for workers and adjacent structures. Assessment of the need for excavation shoring or other measures required to provide a stable excavation, and the use of appropriate construction practices and/or equipment is the contractor's responsibility.

The lateral earth pressures recommended for short-term design are generally lower than the long-term pressures as the state of stress in the soil changes from "at rest" to "active" conditions immediately after excavation. In calculating the "design" lateral earth pressures, a combination of lateral soil pressures; hydrostatic water pressures; and surcharge loads need to be considered.

We recommend that pressure distribution as shown on Plate 5 be used, and that the hydrostatic water pressure be computed by assuming the groundwater table to coincide with the ground surface. Calculation of these pressure components is explained on Plate 5A for clay and 5B for sans.

Pit Bottom Stability. Bottom instability results from inadequate shear strength in clay soils to resist stress relief at the base of the excavation, or from piping of water bearing granular soil. This mode of failure results in loss of ground at the ground surface outside the pit and heave of the excavation base inside the pit. Pits for augering operations are typically excavated approximately 4 feet below pipe invert depth.

Whenever soft foundation soils are encountered during trench excavation, we recommend over excavating 3 feet below the base of the foundation and replacing with on-site soils compacted to at least 95% of maximum dry density in loose lifts not exceeding 8 inches.

Loss of Ground. Installation of pits may experience some loss of ground around the outside of the excavation due to sloughing of material into the excavation.

If proper construction procedures are followed, little or no loss of ground should occur. If loss of ground is excessive, it may cause damage to structures, pavement and services located near the excavation.

If loss of ground does occur, soft disturbed soils may develop beneath existing pavement and utilities located close to the excavation location. Corrective measures to address loss of ground problems often include improved dewatering and/or grouting around the pit from the ground surface or within the pit. Repairs associated with loss of ground often include replacement of paving near the top of the pit, and making up for ground loss through placement of cement stabilized sand fill.

7.5 Select Fill and General Earthwork Recommendations

Select fill required to raise the grade or backfill should consist of sandy clay with a liquid limit less than 40 and a plasticity index between 8 and 20. Fill material that is used should be placed in loose lifts not exceeding eight inches and should be compacted to 95 percent of standard Proctor maximum dry density as determined by ASTM D698.

7.6 Spoil Disposal

Spoil from construction will be generated from augering and trench excavations. Economically, possible uses of the cohesive spoil material may be limited to land reclamation, site grading, and final cover in sanitary landfill operations. Soils that will be excavated from this project area will consist primarily of cohesive soils, although a mixture of cohesive and cohesionless soils is expected at some locations within the project area. These soils may be suitable for use in engineered fill if it meet the criteria.

7.7 Groundwater Control

Groundwater seepage may be expected during excavation depending upon the groundwater conditions at the time of construction. Assessment of the need for groundwater control and installation of appropriate dewatering equipment is the contractor's responsibility. The following comments are intended to represent common solutions to groundwater control problems encountered in similar soil conditions in the Houston area, and may not be construed as dewatering system design recommendations. A conventional pump and sump arrangement may be adequate if water bearing cohesive soils are encountered during trench excavations. Well points or eductors may be utilized to lower the groundwater level to at least three feet below the excavation level where water bearing cohesionless soils are encountered. Well points are generally not effective below about 15 feet beneath the top of the well point, and deeper dewatering requires deep wells with submersible pumps and eductors. In any case, the groundwater control system used must provide a relatively dry, stable base for construction. However, it should be noted that groundwater conditions will change due to rainfall and seasonal changes.

Based on the subsurface soils encountered, we anticipate groundwater to be controlled using pump and sump arrangement at most of the boring locations. In any case, the groundwater control system used must provide a relatively dry, stable base for construction. However, it should be noted that groundwater conditions will change due to rainfall and seasonal changes.

Control of groundwater should be accomplished in a manner that will preserve the strength of the foundation soils; will not cause instability of the excavation; and will not result in damage to existing structures. Where necessary to this purpose, the water will be lowered in advance of excavation by pump and sump arrangement, wells, well points, or similar methods.

Open pumping should not be permitted if it results in boils, loss of fines, softening of the subgrade, or excavation instability. Discharge should be arranged to facilitate sampling by the owner's representative or engineer.

8 BOX CULVERT DESIGN RECOMMENDATIONS

8.1 General

It is our understanding that the project will include the construction of a box culvert near the intersection of Fulton Road and Tidwell Road. We also understand that the invert depth of the box culvert will not exceed 20 feet below existing grade and it will be constructed using the open cut technique. We have utilized information from boring B-12 to develop recommendations for the proposed box culvert.

8.2 Reinforced Box Culvert Design Recommendations

Vertical Soil Loads. Loading on the top of the box culvert may be calculated using a total soil unit weight of 125 pounds per cubic foot (pcf). For buoyancy calculations, the unit weight of soil should be taken as 65 pcf, which assumes a water table at the ground surface.

The total load on the culverts should not exceed the maximum allowable pressure provided in the following table. It must be noted also that because of the nature of soil deposits, parameters at locations away from the borings may vary substantially from values reported in the table.

Table 10-1 – Allowable Bearing Capacity for Box Culvert

Boring No.	Location	Approximate Invert Depth (ft)	Soil Description	Dry Unit Weight (pcf)	Total Unit Weight (pcf)	Allowable Bearing Capacity (psf)
B-12	Fulton Road & Tidwell Road	5-10	Firm to Stiff Lean Clay	111	129	1600
		10-15	Stiff to Very Stiff Lean Clay/ Sandy Lean Clay	117	135	2100
		15-20	Very Stiff Sandy Lean Clay/ Fat Clay	104	128	4500

Design Lateral Pressure. The soil pressure exerted on a culvert wall is mainly a function of the type of backfill and its method of placement. Over-compaction of backfill behind walls and utilization of highly plastic expansive clay backfill are practices that generally produce the highest wall pressures.

In these cases, horizontal earth pressures exceeding the vertical earth pressure can be expected. Design at-rest lateral pressures for culvert walls may be calculated for each backfill type using the equivalent fluid densities for drained level backfill as stated in the following Table.

Table 10-2 - Equivalent Fluid Density

Fill Type	Equivalent Fluid Density (pcf)
Select Cohesive Soil (PI < 20)	75
Cement Stabilized Sand	50
On Site Cohesive Soil (PI > 20)	90

Over-compaction of the backfill should be avoided to prevent the increase of lateral earth pressures on the culvert. Over-compaction of the backfill should be avoided to prevent the increase of lateral earth pressures on the culvert. The recommended design pressures do not include a groundwater pressure component. Since the groundwater table in the Peizomter was encountered at about 5 to 6 feet depth, we recommend that the culvert be designed to resist groundwater pressures to at least this level.

Bedding and Backfill. Cohesive soils were encountered in the vicinity of the foundation zone of the box culvert. Bedding and backfill for cohesive foundation soils should be performed in accordance with City of Houston Standard Drawing No. 02317-05. When water bearing sands are encountered within the foundation zone, bedding and backfill should be performed according to Drawing No. 02317-07. Soils that will be removed from the excavation will consist of lean clays, fat clays and sandy clays. These soils may be used for backfill if it is satisfying select fill criteria discussed in section 7.5 of this report. Backfill should be compacted to 90% standard Proctor density, except under roadway, the material from two feet below subgrade be compacted to 95% of the Standard Proctor density.

9 PAVEMENT DESIGN RECOMMENDATIONS

9.1 General

We understand that the project includes replacement of existing pavement with new concrete pavement. Pavement design recommendations presented in this report were developed in accordance with the "AASHTO Guide for Design of Pavement Structures", 1993 Edition.

9.2 Existing Pavement Thickness

The existing pavement within the project area was cored at all boring locations prior to drilling. The existing pavement structure and thickness are presented in the following table:

Boring No.	Total Thickness (inch)	Asphalt Thickness (inch)	Concrete Thickness (inch)	Base Thickness and Description
B-1	24.5	1.5	8.0	15" Cement Stabilized Sand
B-2	16.5	2.5	-	14' Stabilized Sand Shells
B-3	13.5	2.5	-	11" Stabilized Sand Shells
B-4	8.5	2.5	-	6" Stabilized Sand w/ gravel
B-5	11	3	-	8" Stabilized Sand Shells
B-6	11	3	-	8" Crushed Limestone w/ shells
B-7	9	3	-	6" Stabilized sand Shells

Boring No.	Total Thickness (inch)	Asphalt Thickness (inch)	Concrete Thickness (inch)	Base Thickness and Description
B-8	11.5	3.5	-	8" Stabilized Sand w/ rocks
B-9	10.5	1.5	-	9" Stabilized Sand Shells
B-10	12	3	-	9" Stabilized Sand Shells
B-11	12	2	-	10" Cement Stabilized Sand Shells
B-12	10	3	-	7" Stabilized Sand Shells

The existing pavement were cored at all twelve boring locations prior to drilling and the core data revealed that the existing pavement comprise of 1.5 to 3.5 inches of asphaltic concrete over 6 to 15 inches of stabilized base material except at boring location B-1 where the asphaltic concrete was over an 8-inch concrete pavement.

9.3 Moisture Density Relationship

Based on the results of the standard Proctor test, presented on Plate C-1 of Appendix C, the maximum dry density of the composite sample was determined to be 120.7 pounds per cubic foot (pcf) at optimum moisture content of 12 percent.

9.4 CBR Value

One California Bearing Ratio (CBR) test was performed on the composite sample obtained from the all the boring locations at the top 2 feet. A design CBR of 1.8 was estimated at 95% of the maximum dry density. The results of the CBR test are presented on Plates C-2 and C-3 of Appendix C.

9.5 Rigid Pavement Design Recommendations

The recommendations presented in this report for the pavement design were developed in accordance with the "AASHTO Guide for Design of Pavement Structures", 1993 Edition. The design procedure for determining concrete slab thickness for rigid pavements is based on an extension of the algorithms that were originally developed from the AASHTO Road Test. The categories required for the design of pavement includes: (a) design variables, (b) performance criteria, (c) pavement structural characteristics, (d) material properties for structural design, and (e) reinforcement variables. Parameters relative to these categories are discussed below.

9.6 Traffic Load and Design Period

We have estimated a traffic loading of 2.5 million 18-Kip equivalent single axle load (ESAL) assuming 3% trucks and 2% growth rate. This estimate is based on the 24 hour traffic volume counts for Fulton Road from Tidwell Road to Parker Road (8,052 as on September 2009) provided to us by HNTB Corporation.

Reliability Level and Overall Standard Deviation. Based on City of Houston guidelines, reliability (R) of 95 percent was selected for the pavement design performance. A mean value of the overall standard deviation (S_o) was selected to be 0.35 for Portland cement concrete pavement.

Serviceability. The serviceability of a pavement is defined as its ability to serve the type of traffic that uses the facility. The condition of the pavement after the performance period is characterized by a Terminal Serviceability Index (P_t), which is a function of the pavement structure. We recommend that a Terminal Serviceability Index of 2.5 be used for all pavements.

Since the time at which a given pavement structure reaches its terminal serviceability depends on traffic volume and the original or initial serviceability (P_o), some consideration also must be given to the selection of P_o. As obtained at the AASHTO Road Test, a P_o value of 4.5 was selected.

Drainage. The treatment for the expected level of drainage for a rigid pavement is through the use of a drainage coefficient, C_d. A C_d value of 1.2 was selected for good quality of drainage.

Load Transfer. The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer load across discontinuities, such as joints. Based on the values developed by AASHTO, a mean value of the load transfer coefficient (J) of 3.2 was selected for the design of jointed reinforced concrete pavement with tied curbs.

Loss of Support. This factor, LS, was included in the design of rigid pavement to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movement. An LS value of 1.0 was selected according to the AASHTO suggestion for the condition of stabilized soils beneath the pavement.

Effective Modulus of Subgrade Reaction. Based on the California Bearing Ratio test result, we have estimated a subgrade resilient modulus of 2,700 psi. The modulus of subgrade reaction, K, was computed to be 139 pci.

However, AASHTO recommends that the composite K-value be adjusted to account for the potential loss of support arising from subbase erosion.Based on the loss of support factor (LS) described previously (LS=1.0), an effective modulus of subgrade reaction (k) was found to be 64 pci.

Concrete Elastic Modulus and Modulus of Rupture. Based on the City of Houston Standard Specification 02751, a mean value of 600 psi for S'c is considered appropriate for the design.

A value of 3.37×10^6 psi was used for the modulus of elasticity of the concrete (E_c) using the correlation recommended by the American Concrete Institute.

$$E_c = 57{,}000(f'c)^{0.5}$$

Where,
E_c = elastic modulus of concrete in psi and,
f'c = compressive strength of concrete in psi; a value of 3500 psi is used here.

9.7 Rigid Pavement Design Considerations

The estimated and/or assumed values for the parameters relative to these categories are summarized in the following table.

Parameter	Value
Subgrade Resilient Modulus, M_R	2,700 psi
Subbase Thickness, D_{sb}	6 inches
Compressive Strength of Concrete f'c	3,500 psi
Subbase Elastic Modulus, E_{sb}	30,000 psi
Loss of Support Factor, LS	1.0
Concrete Elastic Modulus, E_c	3.37×10^6 psi
Mean Concrete Modulus of Rupture, S'_c	600 psi
Load Transfer Coefficient, J	3.2
Drainage Coefficient, C_d	1.2
Design Serviceability Loss, D psi	2.0
Reliability, R	95%
Overall Standard Deviation, S_o	0.35

Rigid Pavement Thickness and Load Capacity. Based on the above parameters, a minimum of 9 .0-inch thick concrete pavement is recommended for Fulton Road for a traffic load of 2.5 million, 18-kip equivalent single axle load (ESAL) repetitions for a 20-year design life period. In addition, we recommend that six inches of the subgrade soils be stabilized with 6% lime at locations where the subgrade soils are cohesive (lean clays, sandy clays and silty clays); or with two (2%) lime and eight (8%) flyash where subgrade soils are cohesionless (sandy silts) at the locations of B-9 and B-11.

Reinforcing Steel Requirement: Longitudinal and transverse reinforcing steel is required to resist warping stresses in the pavement section and to hold pavement cracks that develop tightly closed. In addition, reinforcement is required at pavement joints in order to prevent deflections across the joint.

Recommendations for reinforcement at pavement joints are described in the AASHTO Pavement Design Manual.

The amount of longitudinal and transverse reinforcement required depends on the distance between pavement joints. We recommend using steel reinforcement for concrete pavement including the bar size and spacing in accordance to City of Houston standard drawing 02751-01.

However, the overall cross-sectional area of steel (A_s) required per foot of slab width can be calculated as follows:

$$A_s = \frac{FLW}{2 f_s}$$

Where:

A_s = Required cross-sectional area of reinforcing steel per foot of width
F = Coefficient of resistance between slab and subgrade
L = Distance between free transverse joints or between free longitudinal edges, feet
W = Weight of pavement slab, psf
f_s = Allowable working stress in the steel, psi

Slab Length. This refers to the joint spacing or distance, L, between free transverse or longitudinal joints. The maximum spacing should be in accordance with City of Houston Specifications.

Steel Working Stress. The allowable working stress (f_s) in steel reinforcement is a value equivalent to 75 percent of the steel yield strength. City of Houston specifies that the reinforcement be Grade 60 deformed bars. For Grade 60 steel, the allowable working stress is 45,000 psi.

Friction Factor. This factor, F, represents the frictional resistance between the bottom of the slab and the top of the underlying subbase or subgrade layer. Based on the values recommended by AASHTO, a friction factor of 1.8 can be used for the condition of stabilized soils beneath the pavement.

Steel Requirements. The design consultant should determine reinforcing schedule after the joint spacing (L in the equation above) is determined. The estimated minimum amount of required cross-sectional area of reinforcing steel is 0.032 in^2 per foot for longitudinal reinforcement and 0.18 in^2 per foot for transverse reinforcement. These estimates are based on a pavement thickness of 9 inches, maximum longitudinal joint spacing of 14 feet and maximum transverse joint spacing of 80 feet.

9.8 Structural Fill

Structural fill required to replace in-situ material beyond the pavement limits or to raise the design grade should consist of sandy clay with a liquid limit less than 40 and a plasticity index between 8 and 20. Fill material that is used should be placed in loose lifts not exceeding eight inches and should be compacted to 95 percent of the maximum dry density as determined by ASTM D 698.

9.9 Preparation of Subgrade

The surficial soils mostly consist of sandy clays, silty clays, lean clays and sandy silts at some locations. We recommend stabilizing the top six inches of the subgrade soil beneath the proposed concrete pavement with lime and/or lime flyash mixture. Stabilization of the subgrade will increase the modulus of subgrade reaction and provide subgrade stability for construction during inclement weather.

Subgrade stabilization will enhance long-term pavement performance by reducing the tendency of the soil to displace from beneath pavement by pumping. We recommend the following procedures for subgrade preparation.

1. Clear the proposed development area. Grubbing operations should be performed to remove root systems of any trees cleared within the limits of the proposed construction.

2. Strip the surface soil to suitable depths. In areas where soft, compressible or loose soils are encountered, additional stripping may be required. Stripping should extend a minimum of two feet beyond the edge of the proposed pavement.

3. Surfaces exposed after stripping should be proof-rolled in accordance with TxDOT Standard Specification Item 216 or equivalent City of Houston specification. If rutting develops, tire pressures should be reduced. The purpose of the proof-rolling operation is to identify any underlying zones or pockets of soft soils and to remove such weak materials.

4. Before stabilizing the subgrade, scarify the upper six inches of exposed surface as required, mix with lime and flyash and compact it to 95 percent of standard proctor maximum dry density (ASTM D698). We recommend that six inches of the subgrade be stabilized with 6% lime at locations where the subgrade soils are cohesive (lean clays, sandy clays and silty clays); or with two (2) percent lime and eight (8) percent flyash where subgrade soils are cohesionless (sandy silt) at the locations of B-9 and B-11. The above lime and lime flyash percentages are for estimation purposes.

Construction of lime and lime/flyash- stabilized subgrade should conform to City of Houston Section 02336 and Section 02337, respectively. The actual amount of lime should be determined for subgrade soils by conducting laboratory tests on the exposed subgrade material during construction.

10 SIGNAL POLES FOUNDATION RECOMMENDATIONS

10.1 Signal Poles Foundation Recommendations

Foundation Type and Depth. The signal poles may be supported by drilled and under-reamed footings constructed at a depth ranging between 15 to 25 feet below existing grade. To resist overturning forces the pier under-reams should be at least one foot larger than the pier shaft diameter to serve as an anchor. The drilled footings may be designed with a bell/shaft ratio of up to 3:1. If sloughing occurs during construction, the bell to shaft ratio may be reduced to 2:1. Allowable axial loads for shaft diameters of 12 through 36 inches are presented on the following table. These values include a factor of safety of at least two and are based on skin friction and end bearing capacities, neglecting the upper five feet of embedment and one stem diameter from the bottom.

Boring Number	Location	Shaft Diameter (in)	Shaft Depth (ft)	Allowable Axial Load (tons)
B-12	Fulton Road at Tidwell	12	15	6.0
		18	15	17.0
		24	15	19.0
		30	15	59.0
		36	15	80.0
		12	20	14.0
		18	20	26.0
		24	20	30.0
		30	20	66.0
		36	20	92.0
		12	25	18.0
		18	25	34.0
		24	25	39.0
		30	25	71.0
		36	25	98.0

Based on our investigation, we recommend that the under-reamed piers should be founded at the above recommended depths and designed for allowable loads equal or less than the values in the above table.

Lateral Loads. Resistance to lateral loads may be assumed to be provided by lateral load capacity of the drilled shafts. The lateral load capacity can be assessed using active and passive lateral earth pressures or TxDOT Design Charts for high mast signal foundations. An allowable passive earth pressure of 1,200 psf may be used for the sides of shafts poured against undisturbed cohesive soils; however, passive resistance should be neglected in the upper 5 feet of the soil profile due to the potential for shrinkage of the soil away from the shaft in dry periods. The allowable passive pressure includes a factor of safety of two. An active earth pressure based on an equivalent fluid density of 60 pcf should be used for lateral design of the light pole foundation. This value represents the full active pressure of the soil in response to short term loading.

11 MONITORING

11.1 Excavation Safety

As required under OSHA regulations, the contractor should provide a "competent person" to inspect trench excavations daily before the start of work, as needed during the shift, and after every rainstorm or other hazard increasing occurrence. When the competent person finds evidence of a hazardous condition, exposed workers should be removed from the hazardous area until the necessary precautions have been taken to ensure their safety.

A competent person means one who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous or dangerous to workers, and who has authorization to take prompt corrective measures to eliminate them.

11.2 Preconstruction Survey

We recommend that a preconstruction survey be performed prior to any tunneling operations. As part of the survey, a complete visual record should be made of all structures along the tunnel alignment. This survey should be comprised of a combined photographic and video taped documentation of the condition of the surrounding structures. Settlement sensitive structures and structures with pre-existing damage should be of particular concern during the visual record process. In addition to the visual record, a review of the operating conditions of facilities located within a horizontal distance equal to approximately twice the invert depth from the centerline of the tunnel is recommended. Particular attention should be paid to the conditions of existing utilities near the tunnel bore.

Existing leaking utilities need to be identified and repaired prior to tunneling to prevent tunneling difficulties due to infiltration of water or sewage into the bore. The location of settlement sensitive utilities should be established and a monitoring program implemented to determine whether tunneling operations are proceeding without loss of ground prior to the tunnel being driven near the utility.

11.3 Construction Monitoring - Tunneling

We recommend that surface elevations along the tunnel alignment be monitored prior to, at intervals during, and after construction. Ground surface settlements can be measured by taking precise leveling measurements, by standard surveying methods, on settlement monuments installed in the ground along the centerline of the tunnel. The monuments should be suitably protected against vandalism and accidental damage. Survey benchmarks should be established in close proximity to the alignment but outside the influence of any settlement trough.

11.4 Construction Materials Testing

We recommend that backfill be monitored by an accredited testing laboratory to verify that construction is performed in conformance with project specifications. HVJ Associates routinely provides these services and would be pleased to do so for this project.

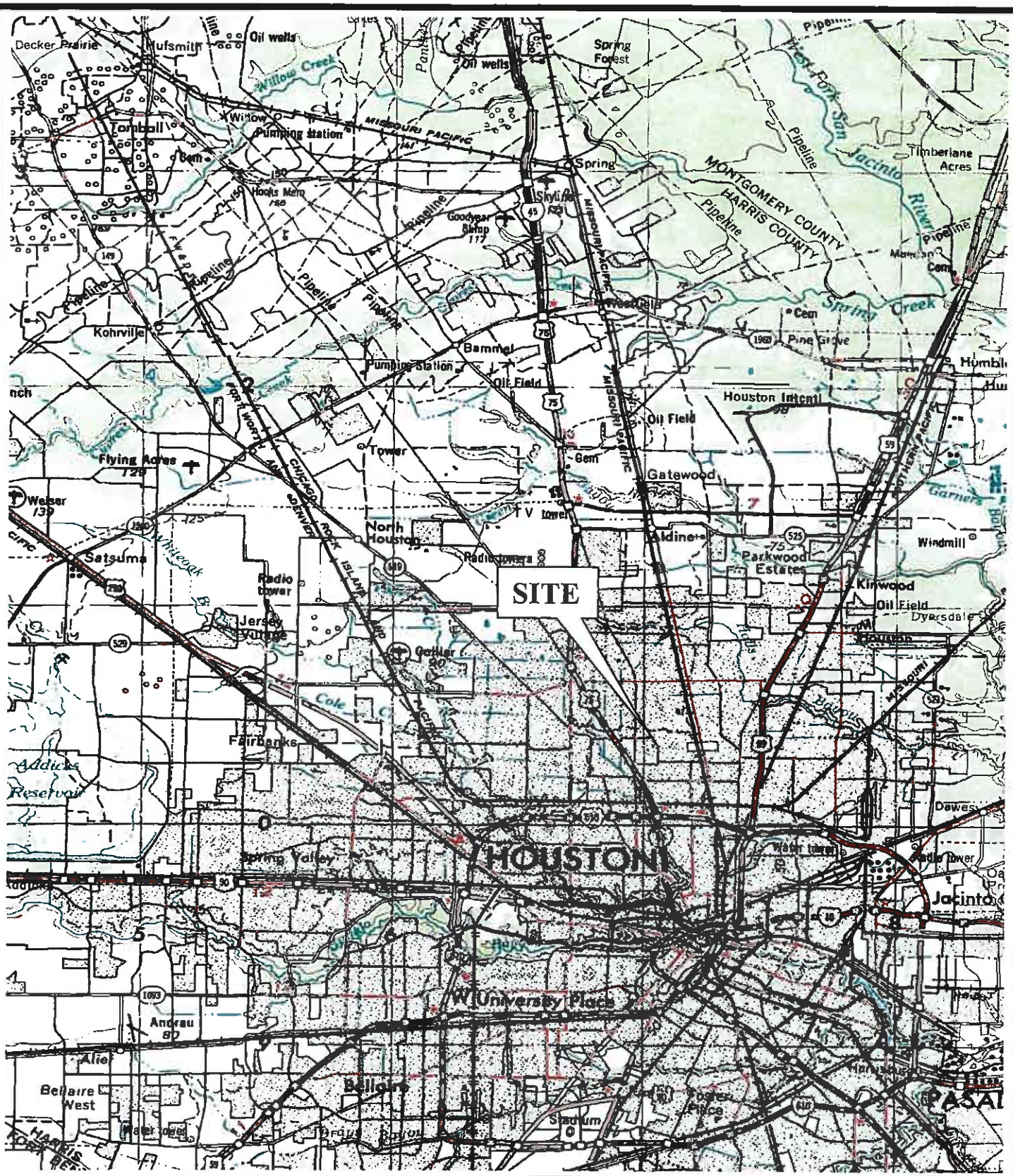
12 DESIGN REVIEW

HVJ Associates should be retained to review the final design plans and specifications for this project. During all excavation, grading and construction phases of this project, HVJ should provide the materials testing verification and observation services so our geotechnical recommendations may be interpreted and implemented correctly.

13 LIMITATIONS

This investigation was performed for the exclusive use for the exclusive use of HNTB Corporation and City of Houston for the proposed Fulton Road Paving from Tidwell Road to Parker Road in Houston, Texas. HVJ Associates, Inc. has endeavored to comply with generally accepted geotechnical engineering practice common in the local area. HVJ Associates, Inc. makes no warranty, expressed or implied. The analyses and recommendations contained in this report are based on data obtained from subsurface exploration, laboratory testing, the project information provided to us and our experience with similar soils and site conditions. The methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect the strata variations that usually exist between sampling locations. Should any subsurface conditions other than those described in our boring logs be encountered, HVJ Associates, Inc. should be immediately notified so that further investigation and supplemental recommendations can be provided.

PLATES



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DATE: 4/19/2010	APPROVED BY: SV	PREPARED BY: NL

SITE VICINITY PLAN
FULTON ROAD PAVING: TIDWELL ROAD TO PARKER ROAD

PROJECT NO.: HG0614605	DRAWING NO.: PLATE 1

HNTB

PRELIMINARY ENGINEERING REPORT
Fulton Street: Tidwell to Parker
WBS No. N-000542-0003-3

Exhibit 2
Project Location Map
Houston City Council District H
Key Map: 453B, 413X
GIMS Tile Grid: 5361B, 5361D

LEGEND:

- APPROXIMATE BORING LOCATIONS
- APPROXIMATE PIEZOMETER LOCATIONS

Note:
Boring B-12 was moved 200ft north from its proposed boring B-12a.

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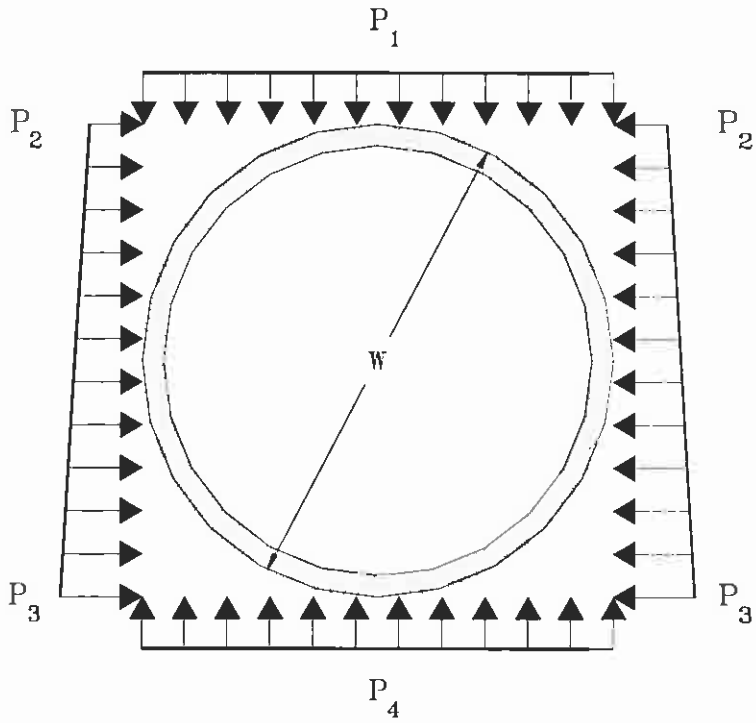
PLAN OF BORINGS
FULTON ROAD: PARKER ROAD TO TIDWELL ROAD

PROJECT NO.:	DRAWING NO.:
HG0614605	PLATE 2



For

$D_w \leq H$

$P_1 = \gamma D_w + (H - D_w)(\gamma - \gamma_w) + P_s + (H - D_w)\gamma_w$

$P_2 \equiv [\gamma D_w + (H - D_w)(\gamma - \gamma_w) + P_s]K_o + (H - D_w)\gamma_w$

$P_3 = [\gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s]K_o + (H + W - D_w)\gamma_w$

$P_4 = \gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s + (H + W - D_w)\gamma_w$

For

$H < D_w < H + W$

$P_1 \equiv H\gamma + P_s$

$P_2 = (\gamma H + P_s)K_o$

$P_3 = [\gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s]K_o + (H + W - D_w)\gamma_w$

$P_4 = \gamma D_w + (H + W - D_w)(\gamma - \gamma_w) + P_s + (H + W - D_w)\gamma_w$

For

$D_w \geq (H + W)$

$P_1 \equiv H\gamma + P_s$

$P_2 = (\gamma H + P_s)K_o$

$P_3 = [(H + W)\gamma + P_s]K_o$

$P_4 = (H + W)\gamma + P_s$

Where

P_1, P_2, P_3 = Pressure imposed on pipe, psf

D_w = Depth of groundwater, feet

H = Depth of top of pipe from ground surface, feet

W = Diameter of pipe, feet

γ = Total Unit weight of soil, pcf

γ_w = Unit weight of water, pcf

P_s = Surcharge load, psf

K_o = Coefficient of earth pressure, (1.0 for clays and 0.5 for sands)

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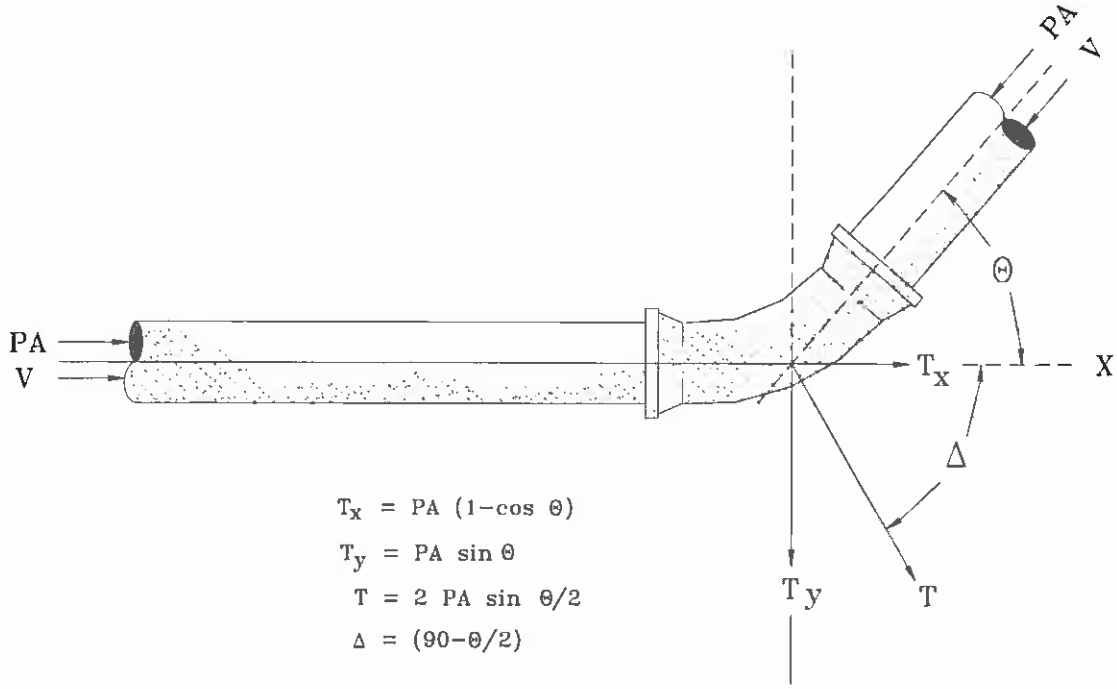
RIGID PIPE AND TUNNEL LINER LOADS

PROJECT NO.: HG0614605	DRAWING NO.: PLATE 3



$T_x = PA\ (1-\cos\Theta)$

$T_y = PA \sin\Theta$

$T = 2\ PA \sin\Theta/2$

$\Delta = (90-\Theta/2)$

Where:

- T is the resultant force on the bend.
- T_x is the component of thrust force in x-direction.
- T_y is the component of thrust force in y-direction.
- P is the maximum sustained pressure.
- A is the pipe cross-sectional area.
- Θ is the bend deflection angle.
- Δ is the angle between T and X-axis.
- V is the fluid velocity.
- D is the inside diameter of conduit.

Sample Calculation:

Given: $P = 150$ psi, $D = 1.0' = 12''$ For: $\Theta = 90^\circ$

$A = (\pi D^2)/4 = 113.1\ in^2$

Find: $T = 2\ PA \sin\Theta/2 = 2 \times 150 \times 113.1 \times \sin(90^\circ/2)$
$= 23{,}299$ lb $= 24.0$ kips

$T_x = PA\ (1-\cos\Theta) = 150 \times 113.1 \times (1-\cos 90^\circ)$
$= 16{,}969$ lb $= 17.0$ kips

$T_y = PA \sin\Theta = 150 \times 113.1 \times \sin(90^\circ)$
$= 16{,}969$ lb $= 17.0$ kips

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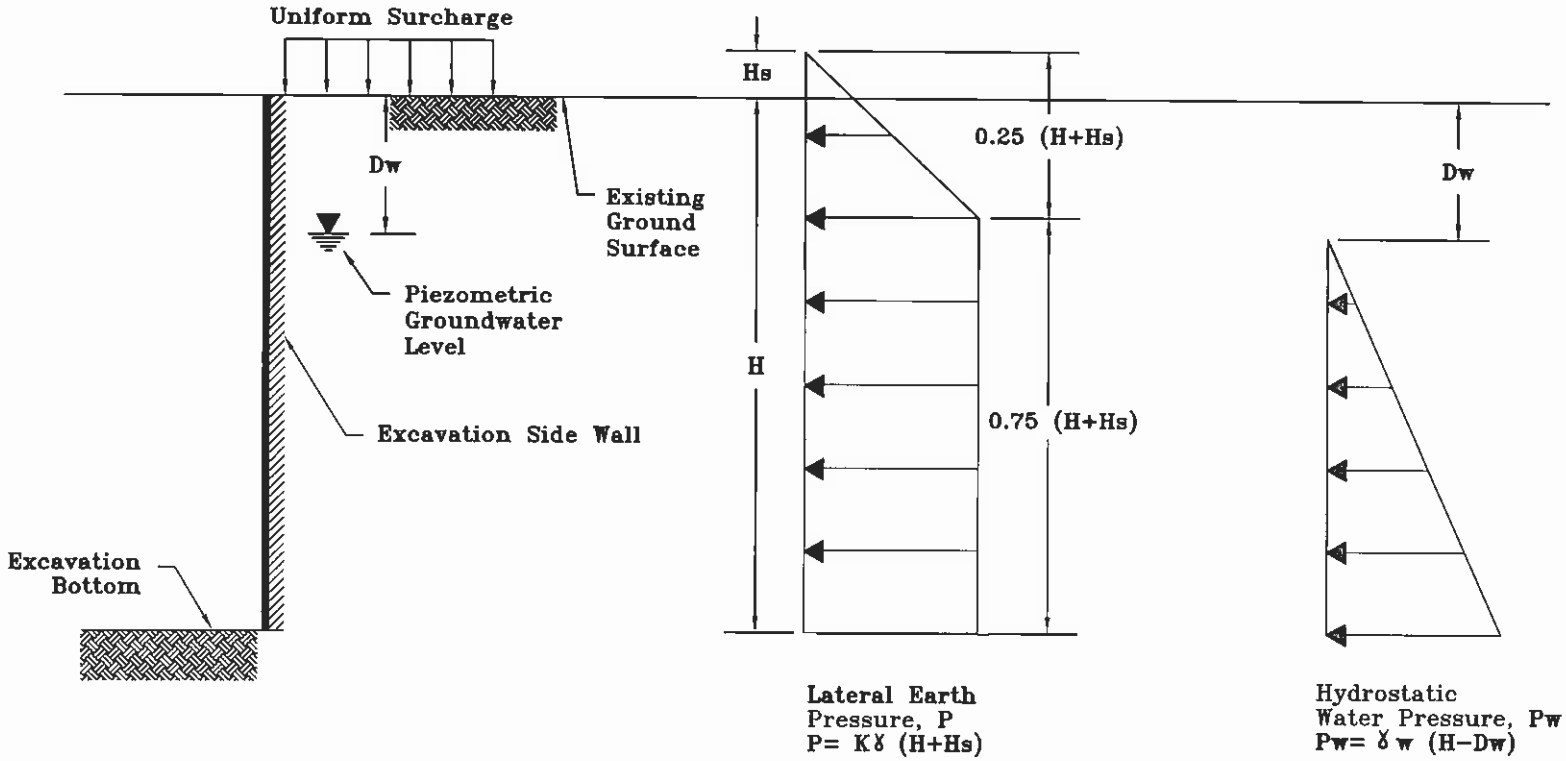
THRUST FORCE ACTING ON A BEND

PROJECT NO.: HG0614605	DRAWING NO.: PLATE 4



H, (ft) = Depth to Excavation Bottom

S, (psf) = Surcharge loading adjacent to Excavation wall

Dw, (ft) = Depth to groundwater below Existing grade

= Zero for temporary excavation

Hs, (ft) = Equivalent Depth of surcharge loading

$$Hs = \frac{S}{120}$$

K = Lateral Earth Pressure coefficient

= Ka "active" for short-term conditions (use 0.50)

= Ko "at rest" for long-term conditions (use 1.0)

γ, (pcf) = Total unit weight above water table or submerged unit weight below groundwater level

γw, (pcf) = Unit weight of water = 62.4 pcf

Note: The pressure diagram shown is not appropriate for design of cantilever walls.

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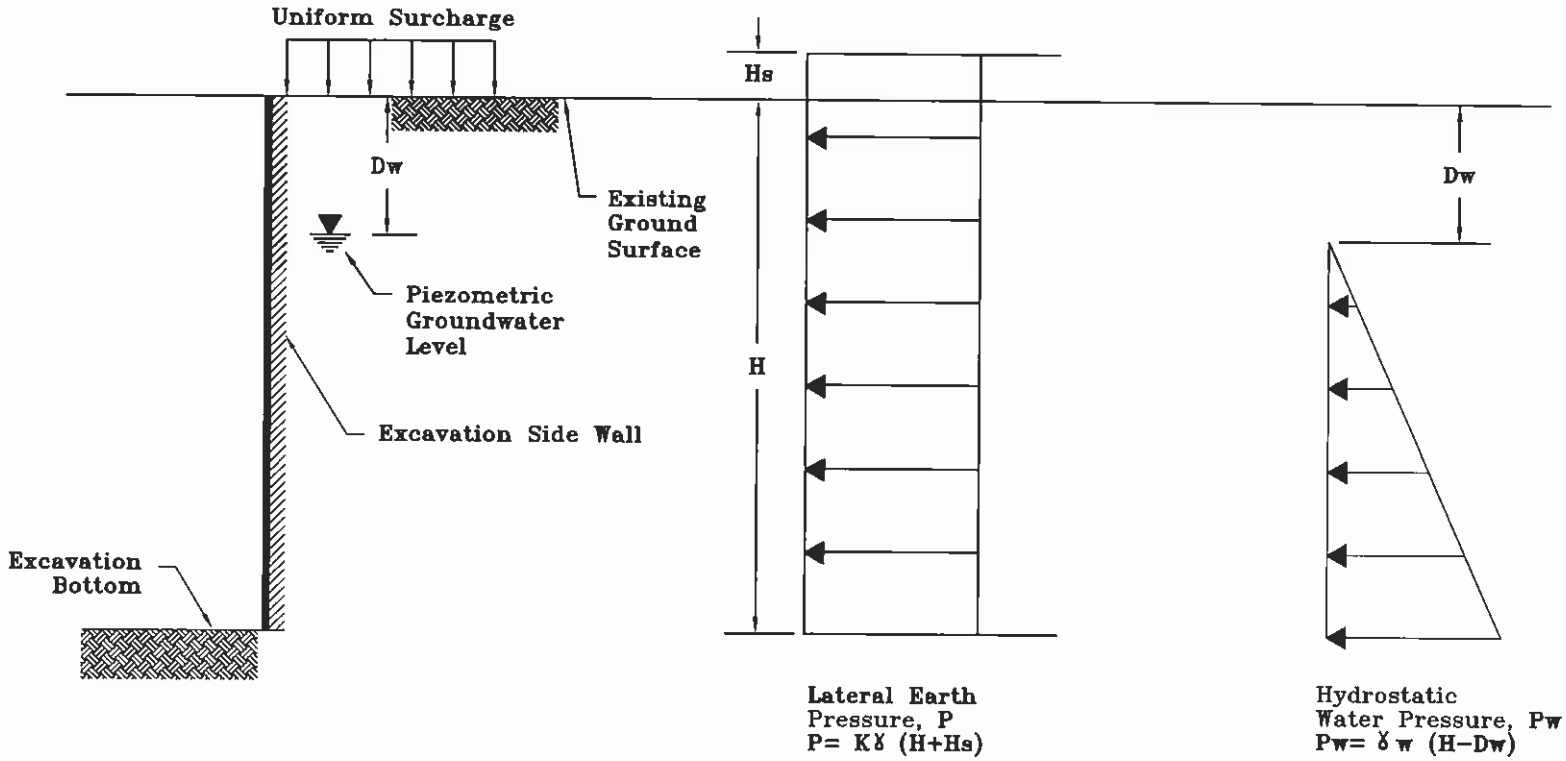
BRACED EXCAVATION
LATERAL EARTH PRESSURE DIAGRAM (CLAY)

PROJECT NO.:	DRAWING NO.:
HG0614605	PLATE 5A



H, (ft) = Depth to Excavation Bottom

S, (psf) = Surcharge loading adjacent to Excavation wall

Dw, (ft) = Depth to groundwater below Existing grade

= Zero for temporary excavation

Hs, (ft) = Equivalent Depth of surcharge loading

$$Hs = \frac{S}{120}$$

K = Lateral Earth Pressure coefficient

= Ka "active" for short-term conditions (use 0.35)

= Ko "at rest" for long-term conditions (use 0.50)

γ, (pcf) = Total unit weight above water table or submerged unit weight below groundwater level

γw, (pcf) = Unit weight of water = 62.4 pcf

Note: The pressure diagram shown is not appropriate for design of cantilever walls.

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BRACED EXCAVATION
LATERAL EARTH PRESSURE DIAGRAM(SAND/SILT)

PROJECT NO.:	DRAWING NO.:
HG0614605	PLATE 5B

APPENDIX A

BORING LOGS & KEY TO TERMS AND SYMBOLS

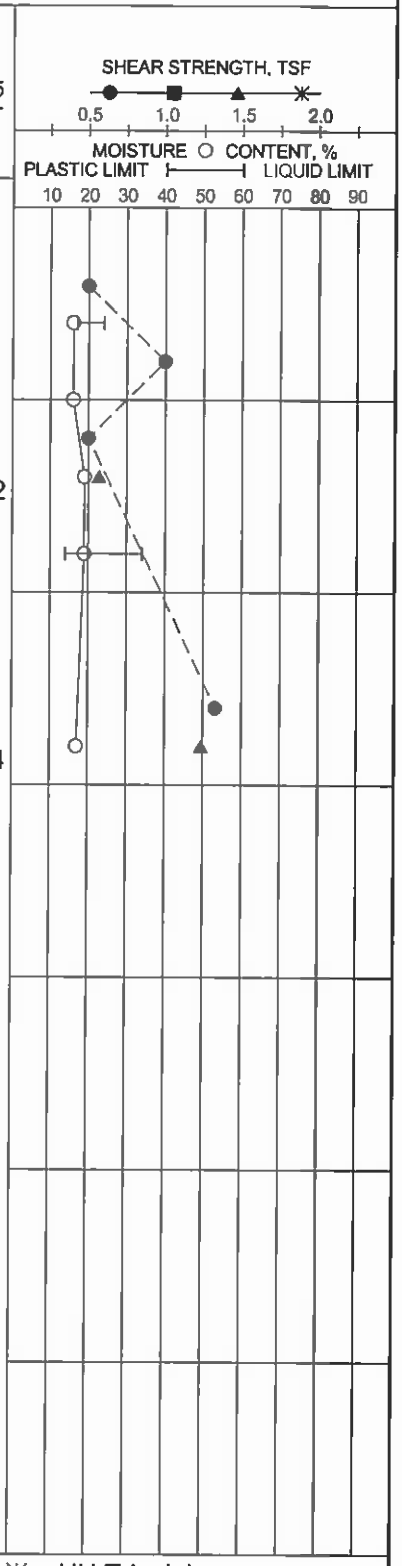
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-1
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,878,802.1
Easting: 3,114,831.9

Project No.: HG0614605-6
Elevation: 71.40 feet
Station: 51+27.18
Offset: -15.99

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 1.5" Asphaltic Concrete, 8" Concrete, 15" Cement Stabilized Sand		
		Stiff light gray SANDY SILTY CLAY (CL-ML)	56	
		- w/ calcareous and ferrous nodules 4'-6'		
		Stiff light gray SANDY LEAN CLAY (CL)		112
	3-3-5	Stiff to very stiff reddish brown LEAN CLAY w/ sand (CL)	76	
				104



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-1

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

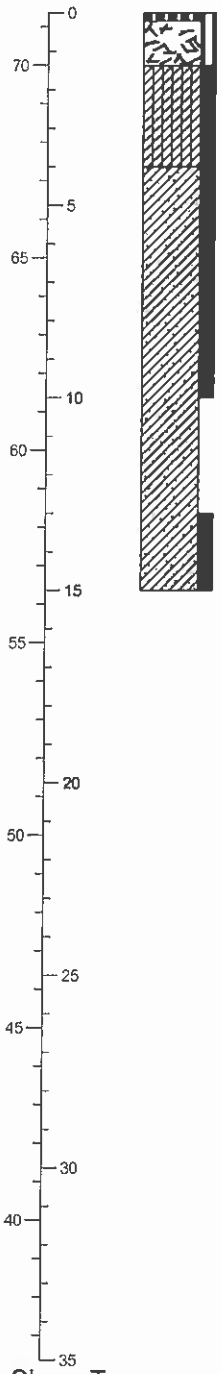
LOG OF BORING

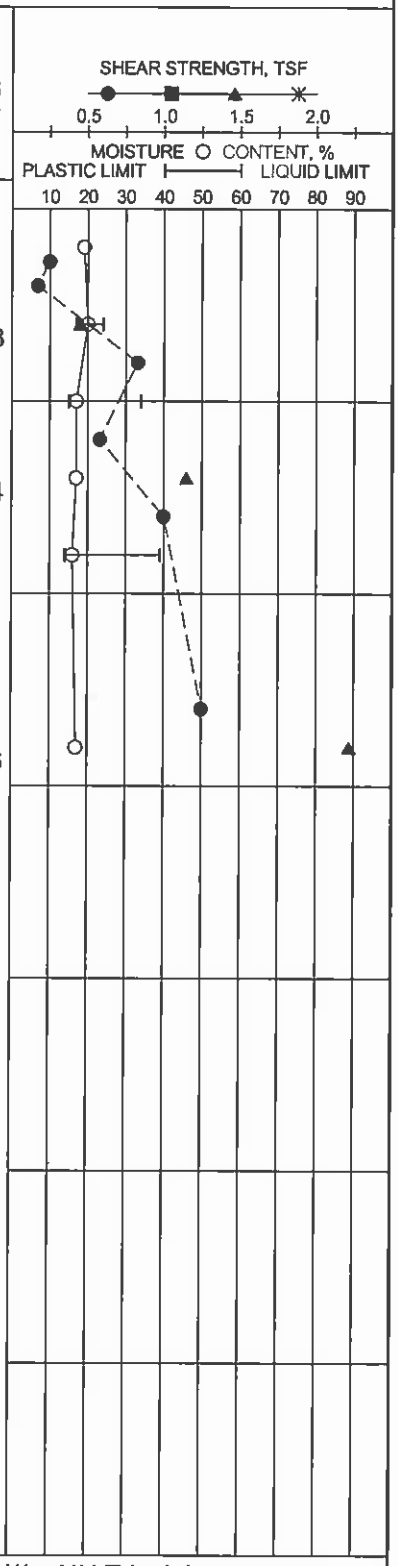
Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-2
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,878,802.9
Easting: 3,115,025.6

Project No.: HG0614605-6
Elevation: 71.36 feet
Station: 46+65.55
Offset: -7.827

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 2.5" Asphaltic Concrete, 14" Stabilized Sand Shells		
		Soft to firm dark brown SILTY CLAY (CL-ML)		
		Stiff to hard brown and gray SANDY LEAN CLAY (CL) -w/ ferrous nodules 4'-10'	68	108
				104
				116





Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-2

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

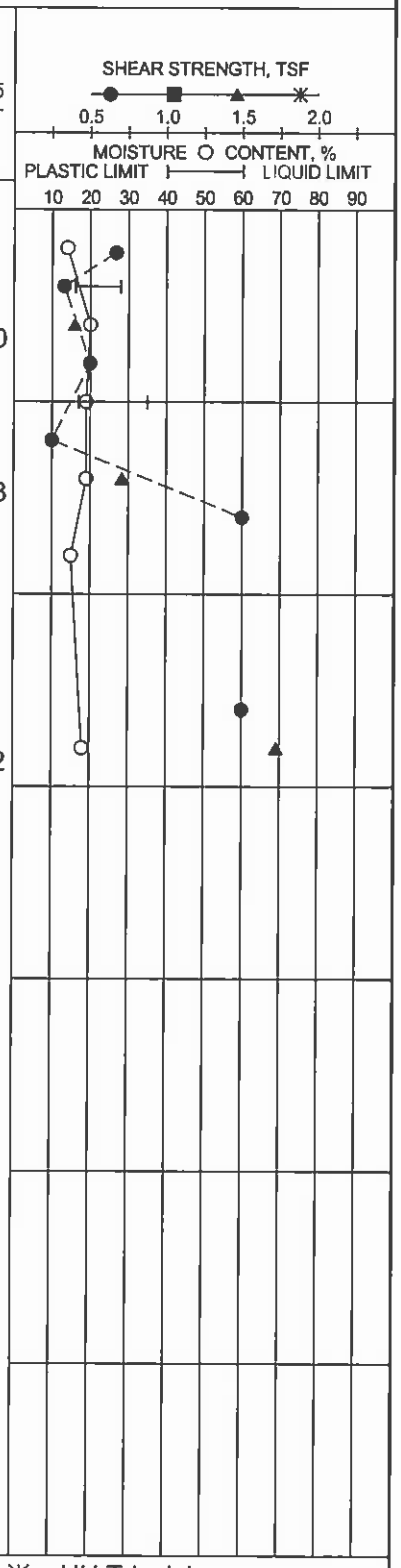
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-3
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,878,142.9
Easting: 3,115,142.2

Project No.: HG0614605-6
Elevation: 70.92 feet
Station: 43+98.86
Offset: 2.139

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 2.5" Asphaltic Concrete, 11" Stabilized Sand Shells		
		Firm to stiff dark brown and gray SANDY LEAN CLAY (CL)		110
5		Firm to stiff brown and gray LEAN CLAY (CL) -w/ ferrous nodules 4'-8'	91	
		Very stiff brown and gray SANDY LEAN CLAY (CL)		103
10				
		Very stiff reddish gray LEAN CLAY (CL)		102
15				
20				
25				
30				
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-3

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

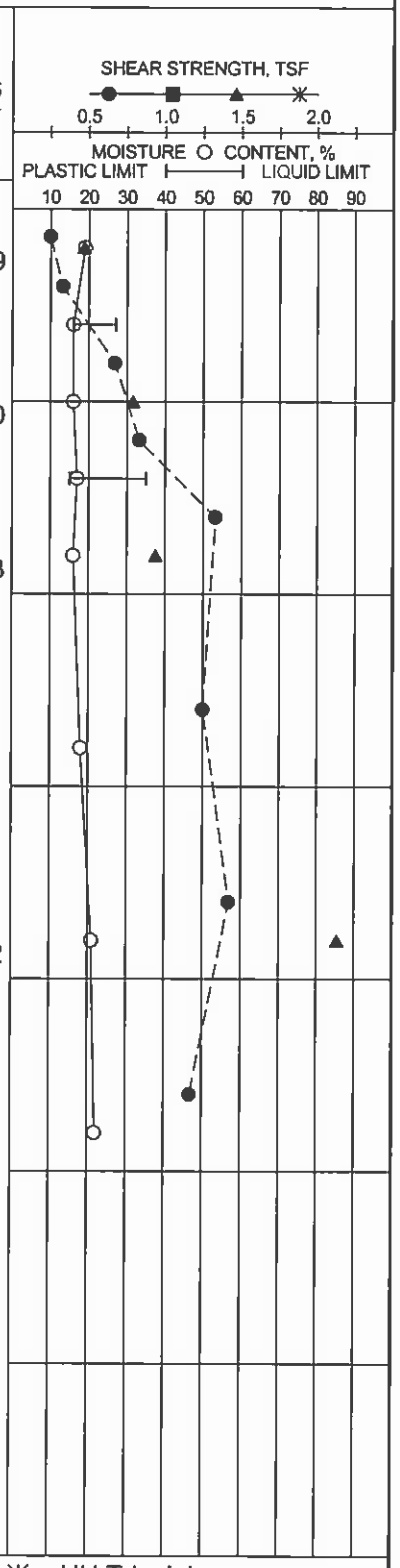
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-4
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,877,653.1
Easting: 3,115,346.2

Project No.: HG0614605-6
Elevation: 70.39 feet
Station: 38+68.35
Offset: -8.770

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
70 / 0		Pavement: 2.5" Asphaltic Concrete, 6" Stabilized Sand with Gravel Firm to stiff brown and gray SANDY LEAN CLAY (CL)		109
		-w/ ferrous nodules 4'-8'	54	110
65 / 5		Stiff to very stiff gray and reddish brown LEAN CLAY w/ sand (CL)		103
60 / 10			79	
55 / 15		Very stiff to hard gray and reddish brown SANDY LEAN CLAY (CL) -w/ calcareous and ferrous nodules 18'-25'		112
50 / 20				
45 / 25				
40 / 30				
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-4

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

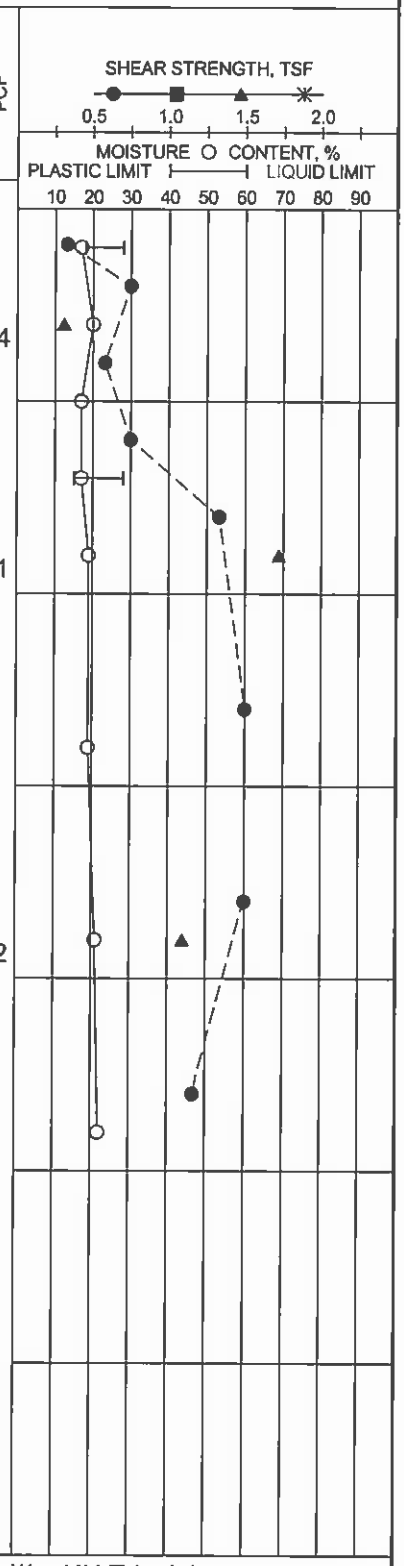
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-5
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,877,208.5
Easting: 3,115,536.7

Project No.: HG0614605-6
Elevation: 70.04 feet
Station: 33+84.74
Offset: -13.763

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
70 — 0		Pavement: 3" Asphaltic Concrete, 8" Stabilized Sand Shells		
		Firm to very stiff brown and gray SANDY LEAN CLAY (CL) -w/ calcareous and ferrous nodules 2'-13'		104
65 — 5				
			70	111
60 — 10				
		Very stiff gray and reddish brown LEAN CLAY w/ sand (CL) -w/ ferrous nodules 13'-20'	85	
55 — 15				
				102
50 — 20				
45 — 25				
40 — 30				
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-5

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

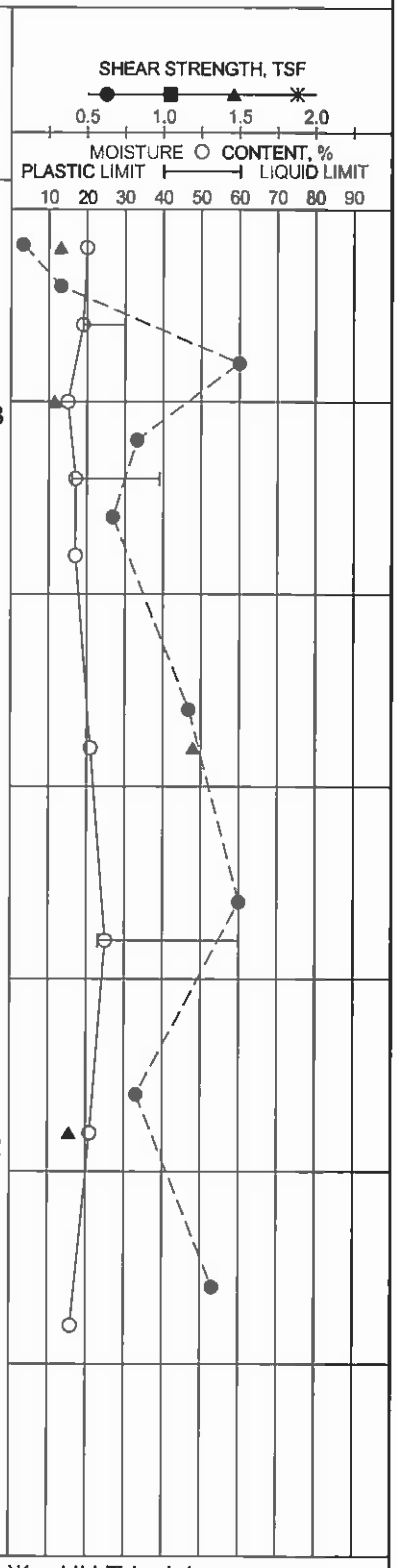
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-6
Groundwater during drilling: 16 feet
Groundwater after drilling: ---

Date: 3/19/2010
Northing: 13,876,749.4
Easting: 3,115,742.9

Project No.: HG0614605-6
Elevation: 69.82 feet
Station: 28+81.39
Offset: -10.199

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 3" Asphaltic Concrete, 8" Crushed Limestone w/ Shells		
		Very soft to firm brown LEAN CLAY w/ sand (CL)		99
		-w/ calcareous nodules 1'-2'		
		Firm to very stiff gray and reddish brown SANDY LEAN CLAY (CL)		
5		-w/ calcareous and ferrous nodules 4'-8'		108
			68	
10				
		Very stiff reddish gray LEAN CLAY (CL)		
		-w/ ferrous nodules 13'-15'		99
15				
		Very stiff reddish brown FAT CLAY (CH)		
		-w/ calcareous nodules 18'-20'		
20				
		Firm to very stiff reddish gray LEAN CLAY w/ sand (CL)		
		-w/ calcareous and ferrous nodules 23'-30'		102
25				
30			85	
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-6

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

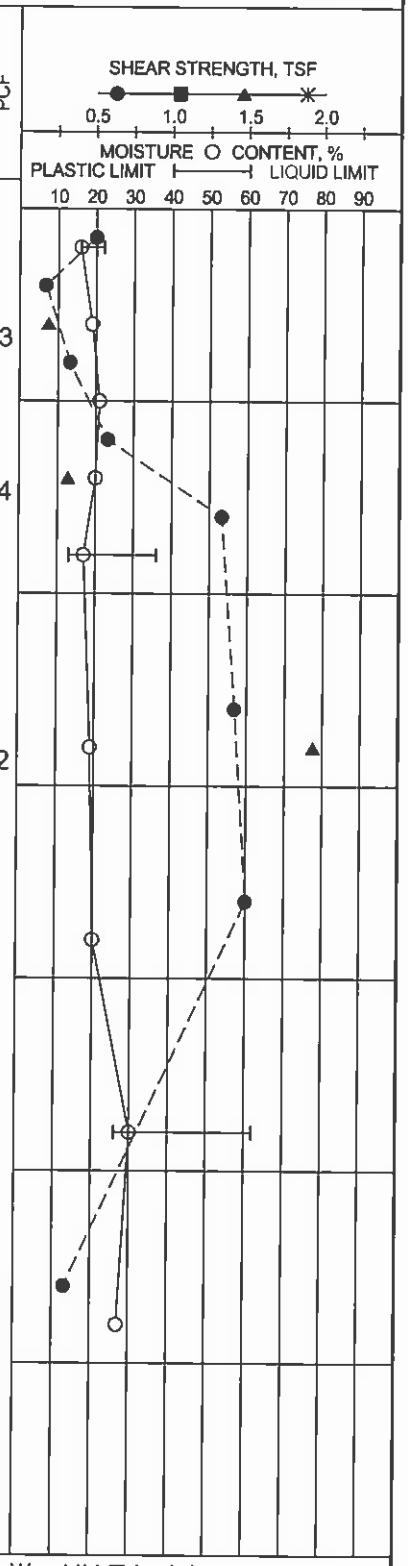
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-7
Groundwater during drilling: 22 feet
Groundwater after drilling: ---

Date: 3/22/2010
Northing: 13,876,274.0
Easting: 3,115,966.5

Project No.: HG0614605-6
Elevation: 69.09 feet
Station: 23+56.22
Offset: 2.618

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 3" Asphaltic Concrete, 6" Stabilized Sand Shells		
		Firm brown SILTY CLAY (CL-ML)		
		Soft to very stiff brown and gray SANDY LEAN CLAY (CL)		113
65 / 5		-w/ ferrous nodules 4'-10'		114
60 / 10				
		Very stiff gray and reddish brown LEAN CLAY w/ sand (CL)		102
55 / 15				
50 / 20				
45 / 25	3-7-11	Firm to very stiff brown and gray FAT CLAY w/ sand (CH)	92	
40 / 30				
35 / 35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-7

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

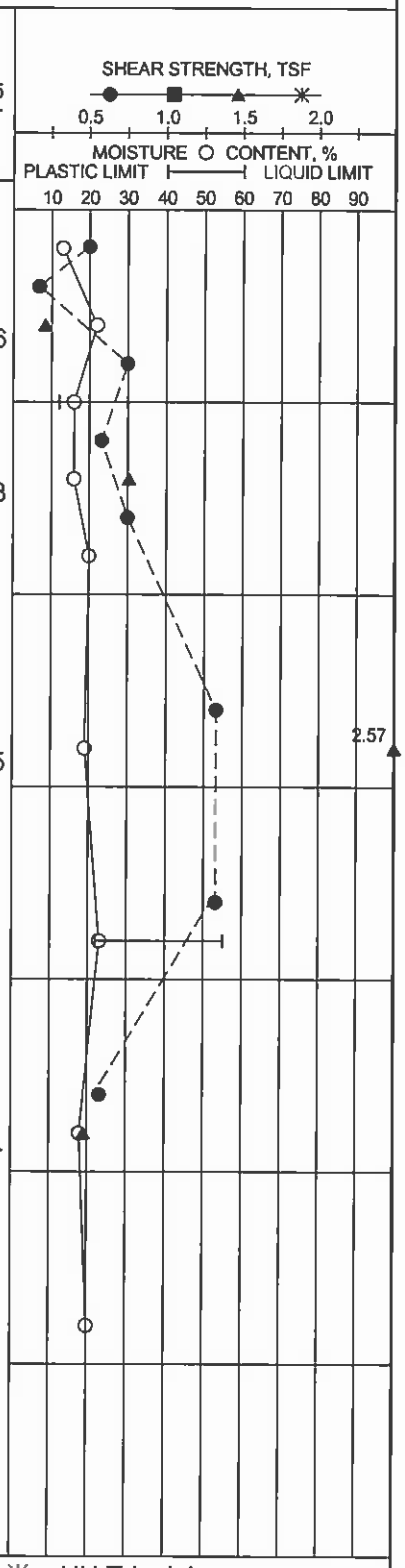
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-8
Groundwater during drilling: 22 feet
Groundwater after drilling: ---

Date: 3/22/2010
Northing: 13,875,937.3
Easting: 3,116,103.5

Project No.: HG0614605-6
Elevation: 68.77 feet
Station: 19+92.93
Offset: -7.785

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 3.5" Asphaltic Concrete, 8" Stabilized Sand with Rocks		
		Soft to hard gray and reddish brown SANDY LEAN CLAY (CL)	62	
65 / 5		-w/ ferrous nodules 4'-15'		106
60 / 10				108
55 / 15				115
		Very stiff reddish brown FAT CLAY (CH) -w/ calcareous nodules 18'-20'	91	
50 / 20				
45 / 25		Firm to very stiff reddish brown SANDY LEAN CLAY (CL)		114
40 / 30	7-9-11			
35 / 35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-8

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

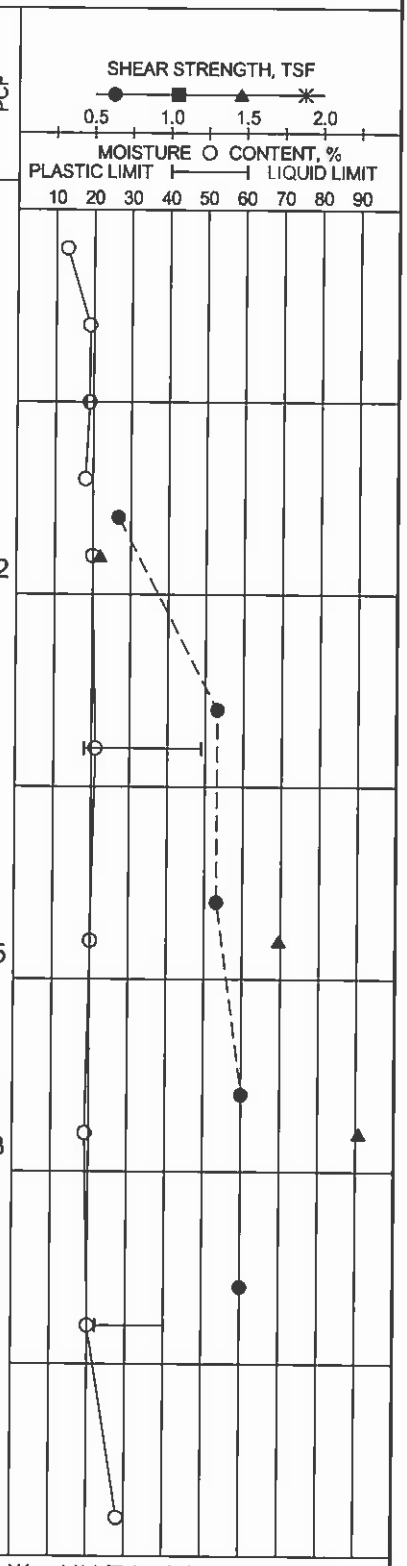
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-9
Groundwater during drilling: 26 feet
Groundwater after drilling: 5 feet

Date: 3/22/2010
Northing: 13,875,529.6
Easting: 3,116,278.4

Project No.: HG0614605-6
Elevation: 68.42 feet
Station: 15+49.26
Offset: -12.167

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 1.5" Asphaltic Concrete, 9" Stabilized Sand Shells		
		Loose gray SANDY SILT (ML)	63	
65 / 5	2-2-3		60	
	5-4-4		64	
60 / 10		Stiff brown and gray SANDY LEAN CLAY (CL)		112
55 / 15		Very stiff to hard gray and reddish brown LEAN CLAY w/ sand (CL)		
50 / 20				105
45 / 25		-w/ calcareous nodules 23'-30'		103
40 / 30			90	
35	9-23-50/4	Hard gray and reddish brown FAT CLAY w/ sand (CH)		
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-9

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

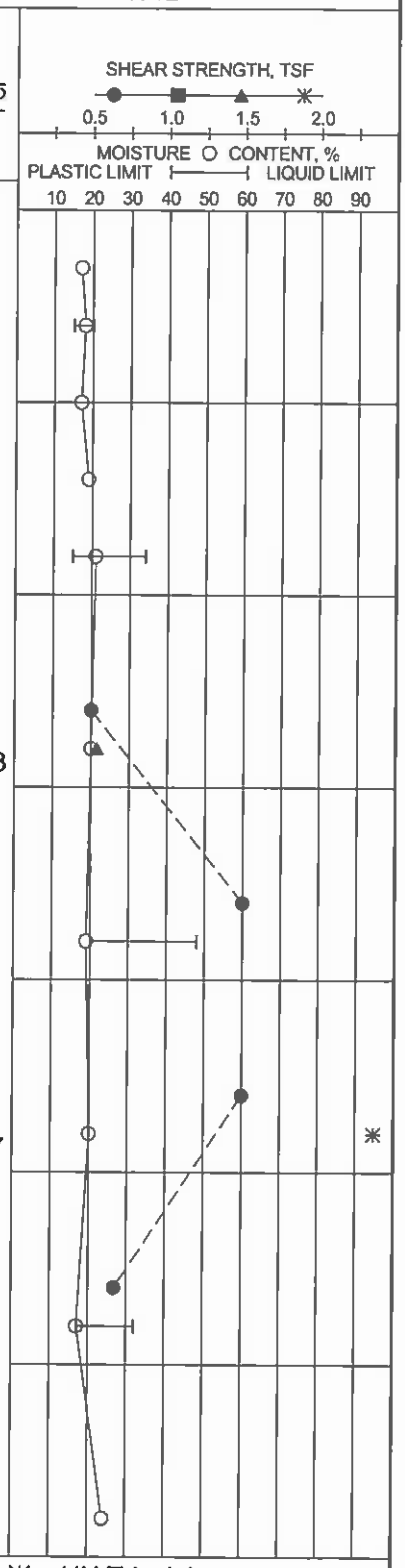
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-10
Groundwater during drilling: ---
Groundwater after drilling: ---

Date: 3/22/2010
Northing: 13,875,203.9
Easting: 3,116,427.0

Project No.: HG0614605-6
Elevation: 67.95 feet
Station: 11+91.17
Offset: 2.012

ELEV. DEPTH, FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 3" Asphaltic Concrete, 9" Stabilized Sand Shells		
	2-2-4	Firm brown and tan SANDY SILTY CLAY (CL-ML)	67	
65 / 5	4-4-5	Stiff tan SILTY CLAY w/ sand (CL-ML)		
	4-5-9		74	
60 / 10	5-5-10	Stiff brown and gray SANDY LEAN CLAY (CL)		
55 / 15		Stiff to hard gray and reddish brown LEAN CLAY w/ sand (CL)		103
50 / 20				
45 / 25				107
40 / 30			74	
35 / 35	42-31-35			



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-10

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

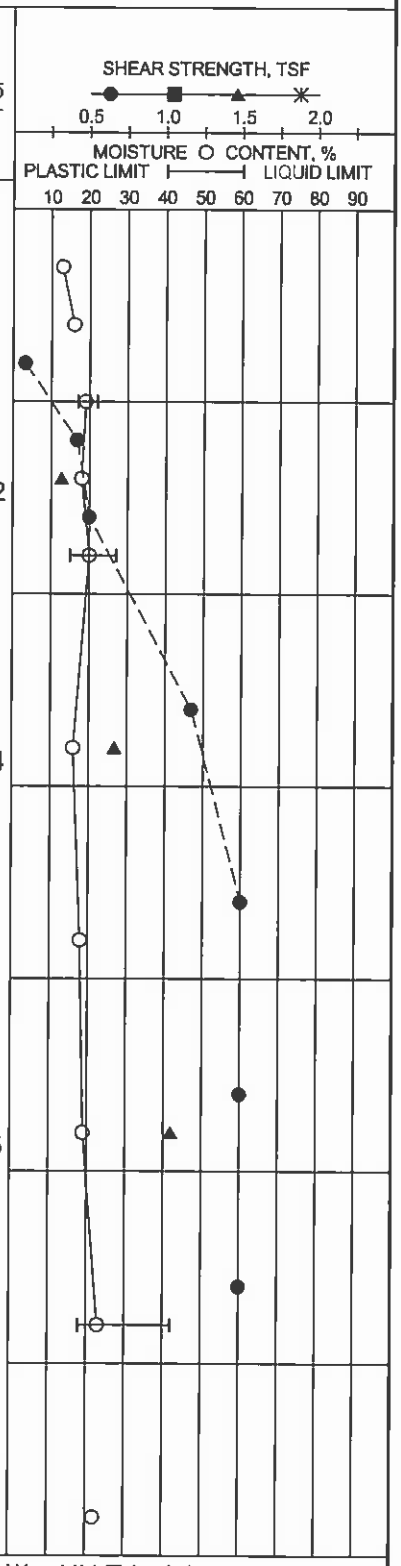
LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Boring No.: B-11
Groundwater during drilling: 22 feet
Groundwater after drilling: 5.5 feet

Date: 3/23/2010
Northing: 13,874,817.0
Easting: 3,116,611.4

Project No.: HG0614605-6
Elevation: 67.40 feet
Station: 7+62.67
Offset: 16.32

ELEV. DEPTH. FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
0		Pavement: 2" Asphaltic Concrete, 10" Cement Stabilized Sand Shells		
65	11-9-4	Medium dense brown SANDY SILT (ML)	62	
			66	
5		Very soft light brown SILTY CLAY (CL-ML) - w/ ferrous nodules		
		Firm to very stiff brown SANDY LEAN CLAY (CL)		112
60				
10				
55				114
15				
50		Very stiff reddish brown LEAN CLAY w/ sand (CL)		
20			80	
45		Very stiff reddish brown SANDY LEAN CLAY (CL)		115
25				
40		Very stiff gray and reddish brown LEAN CLAY w/ sand (CL) -w/ claystone 28'-30'		
30				
35	25-50/2	Very dense reddish brown SILTY SAND (SM) -w/ sandstone 33'-35'	35	
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-11

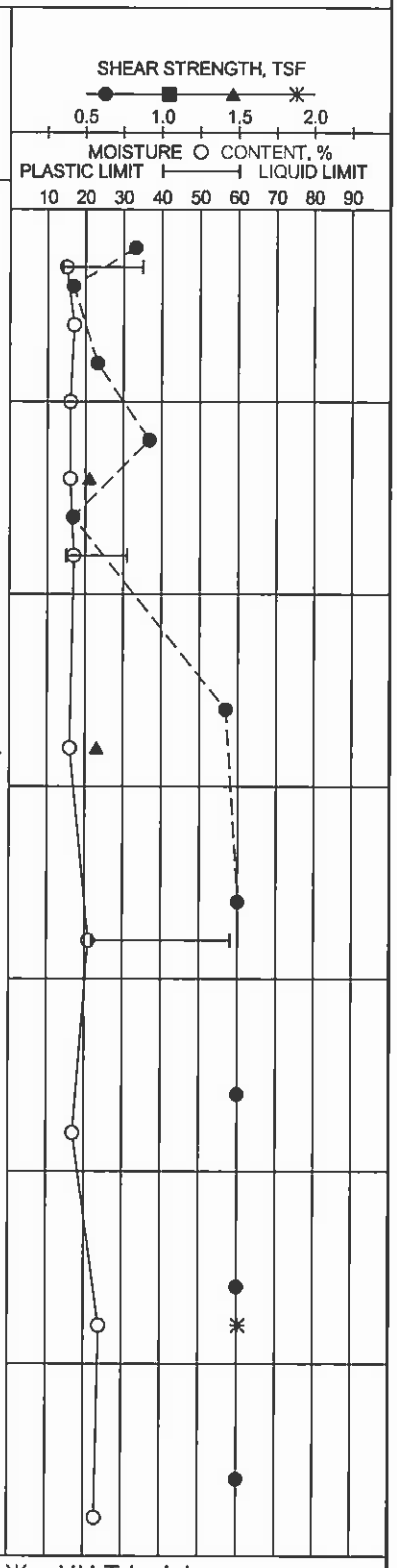
HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

LOG OF BORING

Project: Fulton Road Paving: Tidwell to Parker
Project No.: HG0614605-6
Boring No.: B-12
Date: 3/23/2010
Elevation: 65.59 feet
Groundwater during drilling: 24 feet
Northing: 13,874,470.0
Station: 3+99.45
Groundwater after drilling: ---
Easting: 3,116,726.3
Offset: -19.71

ELEV. DEPTH. FEET	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	SOIL/ROCK CLASSIFICATION	% PASSING NO. 200 SIEVE	DRY DENSITY PCF
65 / 0		Pavement: 3" Asphaltic Conrete, 7" Stabilized Sand Shells		
		FILL: Brown and gray sandy lean clay -w/ rocks		
		FILL: Brown and gray lean clay with sand -w/ rocks	72	
		Firm to stiff brown and gray LEAN CLAY w/ sand (CL) -w/ ferrous nodules 6'-10'	74	
60 / 5				111
			79	
55 / 10				
		Stiff to very stiff brown and gray SANDY LEAN CLAY (CL)		117
50 / 15				
		Very stiff gray and reddish brown FAT CLAY (CH) -w/ calcareous nodules 18'-20'	90	
45 / 20				
		Very stiff brown and gray SANDY LEAN CLAY (CL) -w/ calcareous nodules 23'-25'	58	
40 / 25				
		Very stiff gray and reddish brown FAT CLAY (CH) -w/ calcareous nodules 28'-30'		104
35 / 30				
		-w/ claystone 33'-35'		
35				



Shear Types: ● = Hand Penet. ■ = Torvane ▲ = Unconf. Comp. ✳ = UU Triaxial

See Plate 2 for boring location.

PLATE A-12

HVJ ASSOCIATES

LOG OF SOIL BORING HG-06-14605.GPJ HVJ.GDT 7/23/10

SOIL SYMBOLS

Soil Types

Clay | Silt | Sand | Gravel

Modifiers

Clayey | Silty | Sandy | Cemented

Construction Materials

Asphaltic Concrete | Stabilized Base | Fill or Debris | Portland Cement Concrete

SAMPLER TYPES

- Thin Walled Shelby Tube
- No Recovery
- Split Barrel
- Core
- Liner Tube
- Jar Sample

WATER LEVEL SYMBOLS

- Groundwater level after drilling in open borehole or piezometer
- Groundwater level determined during drilling operations

SOIL GRAIN SIZE

Classification	Particle Size	Particle Size or Sieve No. (U.S. Standard)
Clay	< 0.002 mm	< 0.002 mm
Silt	0.002 - 0.075 mm	0.002 mm - #200 sieve
Sand	0.075 - 4.75 mm	#200 sieve - #4 sieve
Gravel	4.75 - 75 mm	#4 sieve - 3 in.
Cobble	75 - 200 mm	3 in. - 8 in.
Boulder	> 200 mm	> 8 in.

DENSITY OF COHESIONLESS SOILS

Descriptive Term	Penetration Resistance "N" * Blows/Foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

CONSISTENCY OF COHESIVE SOILS

Consistency	Undrained Shear Strength (tsf)	Penetration Resistance "N" * Blows/Foot
Very Soft	0 - 0.125	0 - 2
Soft	0.125 - 0.25	2 - 4
Firm	0.25 - 0.5	4 - 8
Stiff	0.5 - 1.0	8 - 16
Very Stiff	1.0 - 2.0	16 - 32
Hard	> 2.0	> 32

PENETRATION RESISTANCE

3/6	Blows required to penetrate each of three consecutive 6-inch increments per ASTM D-1586 *
50/4"	If more than 50 blows are required, driving is discontinued and penetration at 50 blows is noted
0/18"	Sampler penetrated full depth under weight of drill rods and hammer

* The N value is taken as the blows required to penetrate the final 12 inches

TERMS DESCRIBING SOIL STRUCTURE

Slickensided	Fracture planes appear polished or glossy, sometimes striated
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Inclusion	Small pockets of different soils, such as small lenses of sand scattered through a mass of clay
Parting	Inclusion less than 1/4 inch thick extending through the sample
Seam	Inclusion 1/4 inch to 3 inches thick extending through the sample
Layer	Inclusion greater than 3 inches thick extending through the sample
Laminated	Soil sample composed of alternating partings of different soil type
Stratified	Soil sample composed of alternating seams or layers of different soil type
Intermixed	Soil sample composed of pockets of different soil type and laminated or stratified structure is not evident
Calcareous	Having appreciable quantities of calcium carbonate
Ferrous	Having appreciable quantities of iron
Nodule	A small mass of irregular shape

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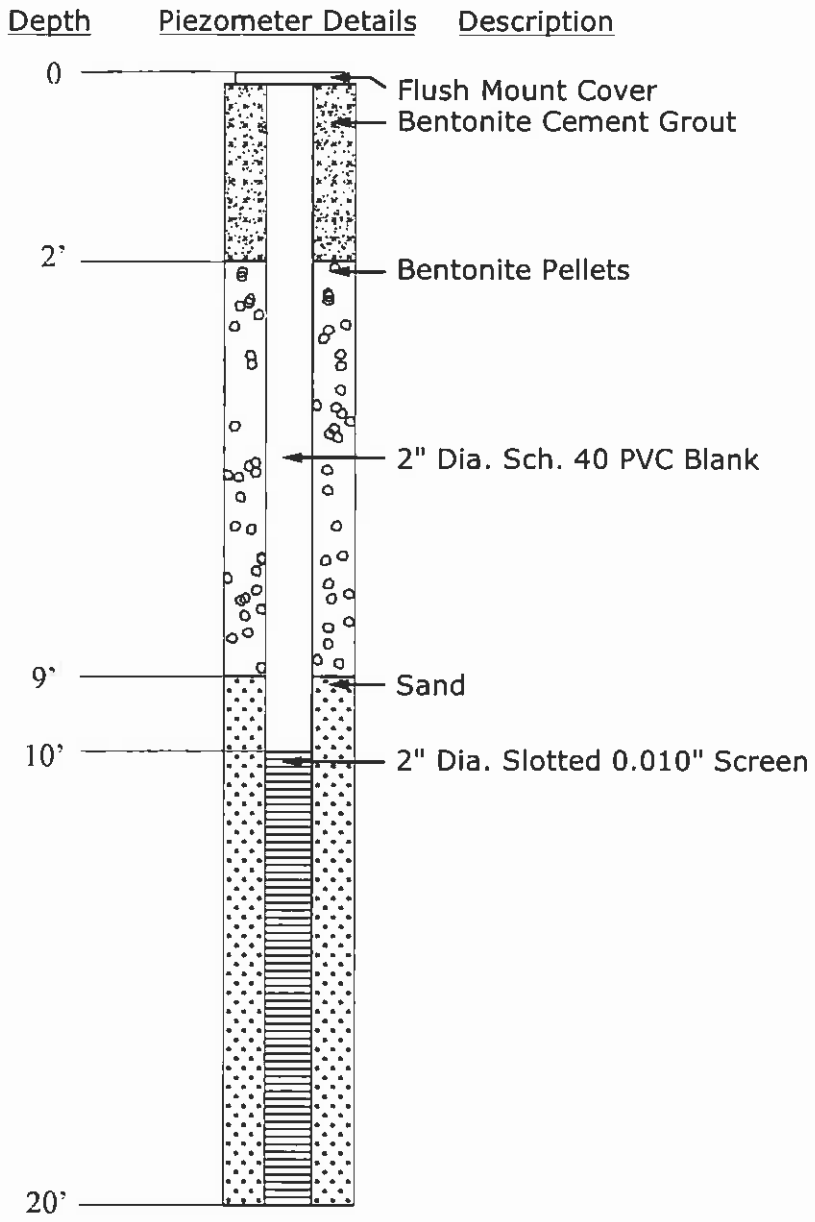
KEY TO TERMS AND SYMBOLS USED ON BORING LOGS

PROJECT NO.: HG0614605

DRAWING NO.: PLATE A-13

APPENDIX B

PIEZOMETER INSTALLATION RECORDS



Water Level Readings

Date	Depth (ft.)	Elev. (ft.)
03/22/10	5.0	63.42
04/21/10	5.5	62.92

NOTES:

- Piezometer was installed on 03/21/10.
- See Plate 2 for boring location; see Plate A-9 for boring log.

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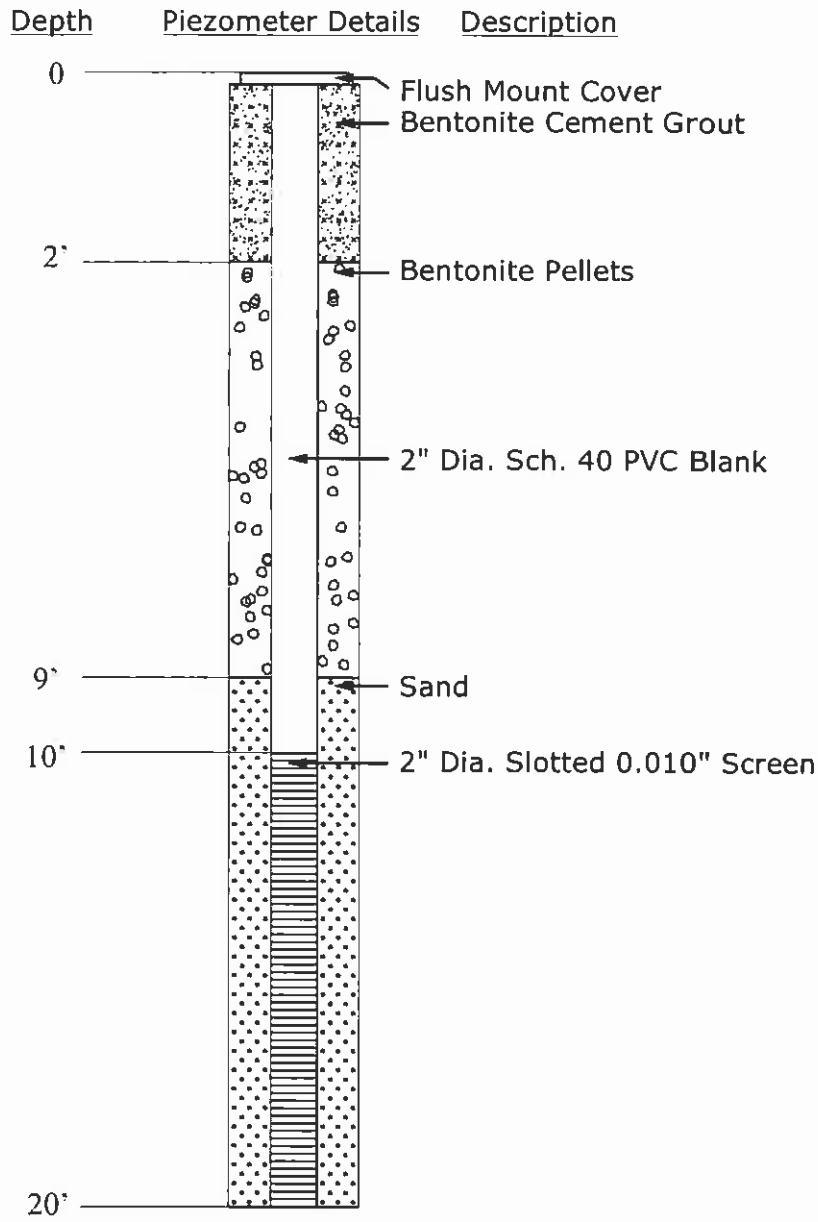
PIEZOMETER INSTALLATION REPORT
PIEZOMETER NO. PZ-1 (Boring B-9)

PROJECT NO.:	DRAWING NO.:
HG0614605	PLATE B-1



Water Level Readings

Date	Depth (ft.)	Elev. (ft.)
03/24/10	5.5	61.9
04/22/10	6.9	60.5

NOTES:

- Piezometer was installed on 03/23/10.
- See Plate 2 for boring location; see Plate A-11 for boring log.

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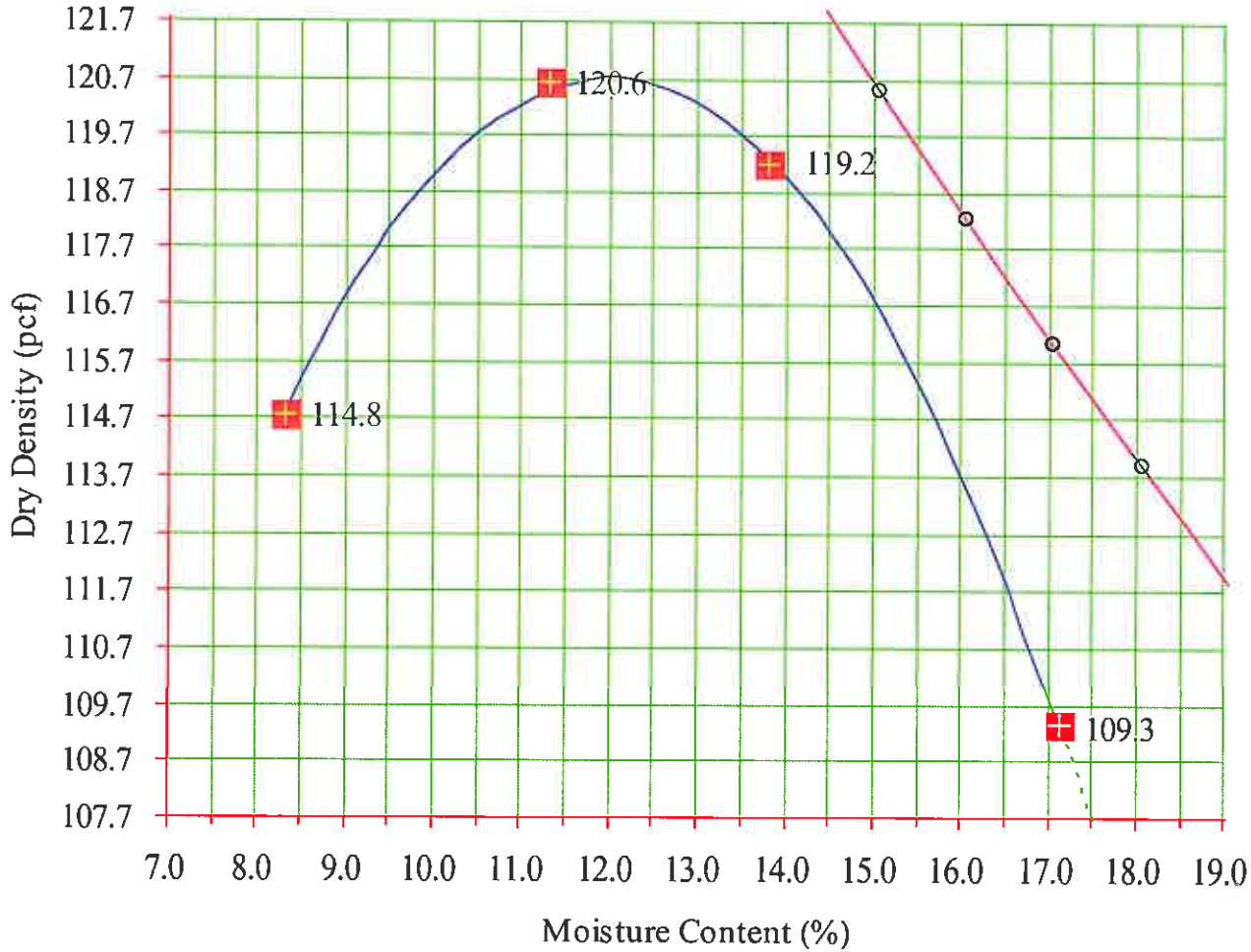
PIEZOMETER INSTALLATION REPORT
PIEZOMETER NO. PZ-2 (Boring B-11)

PROJECT NO.:	DRAWING NO.:
HG0614605	PLATE B-2

APPENDIX C

MOISTURE-DENSITY RELATIONSHIP & CBR TEST RESULTS

Moisture - Density Relationship



TEST DATA

TYPE OF MATERIAL	Brown and gray Sandy Clay		SAMPLE LOCATION	Upper 2' all Borings	
MAXIMUM DRY DENSITY	120.7	pcf	SG: 2.72	LIQUID LIMIT	29
OPT. MOISTURE CONTENT	12.0	%		PLASTICITY INDEX	13
METHOD OF TEST	Standard ASTM D-698 Method A				

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DATE: 05/07/2010	APPROVED BY: SV	PREPARED BY: DM

FULTON ROAD PAVING FROM TIDWELL TO PARKER
PROCTOR TEST RESULTS

PROJECT NO.: HG0614605	DRAWING NO.: PLATE C-1

CBR (CALIFORNIA BEARING RATIO) OF LABORATORY COMPACTED SOILS ASTM D1883

Project: Fulton Road Paving From Tidwell to Parker

Sample Location: Composite Sample for the upper two feet for all borings

Liquid Limit: 29 **Plastic Limit:** 16 **Plasticity Index:** 13

Method of Compaction: ☒ ASTM D698
☐ ASTM D1557

Sample Condition: ☒ soaked ☐ unsoaked

No. of Blows:	10	25	65
Dry Density Before Soaking (pcf):	111.8	122.01	129.7
Dry Density After Soaking (pcf):	111.7	122.9	127.9
Moisture Content:			
Before Compaction (%):	12.3	12.1	12.2
Top 1-inch Layer After Soaking (%):	19.9	16.5	15.3
Swell (%):	4.45	1.64	1.40
Bearing Ratio (%): (☒ soaked ☐ unsoaked)	1.13	3.80	5.25

Surcharge: 10 lbs.

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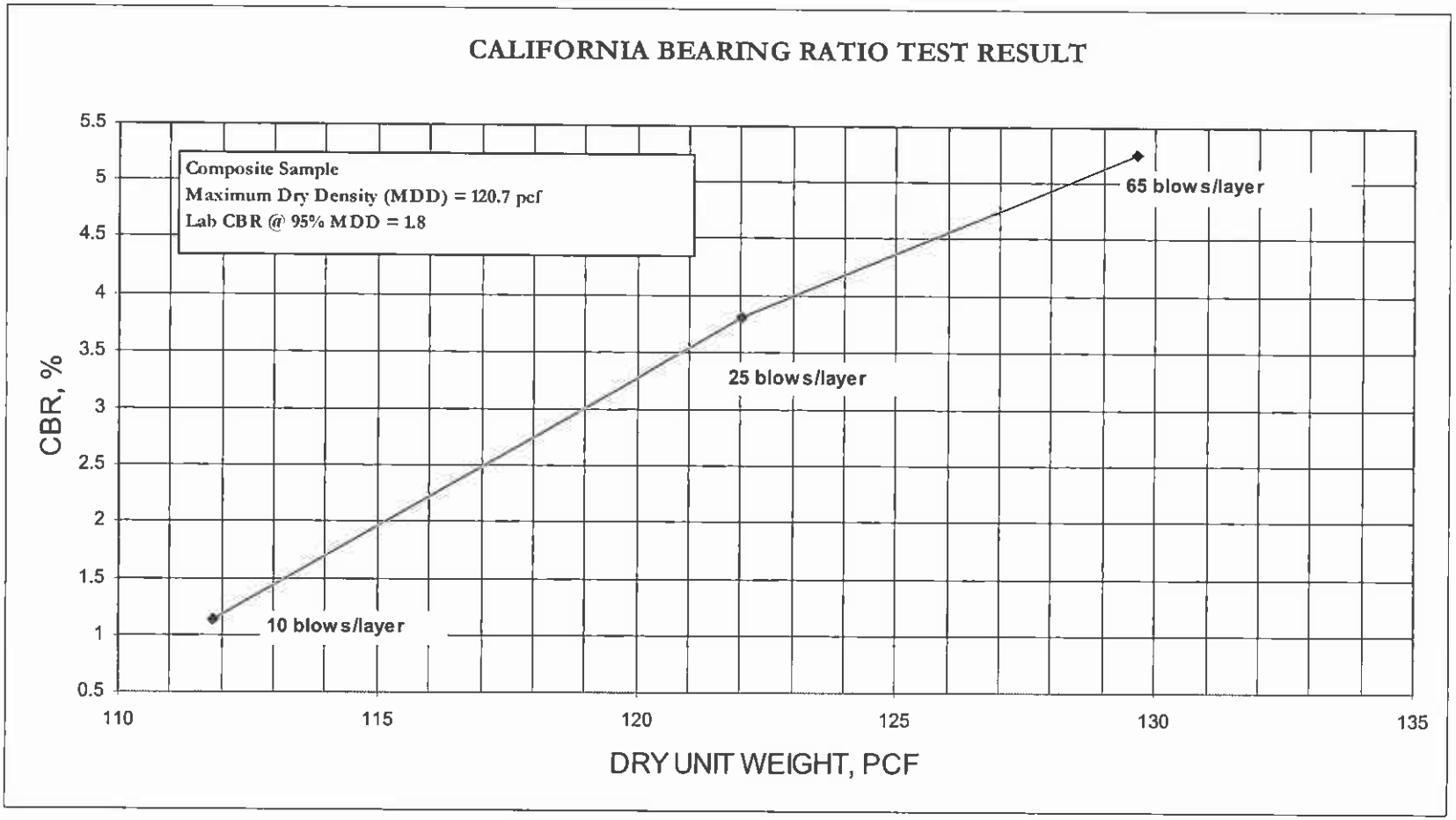
DATE: 07/08/10	APPROVED BY: SV	PREPARED BY: DM

FULTON ROAD PAVING FROM TIDWELL TO PARKER
CBR TEST RESULTS

PROJECT NO.: HG0614605	DRAWING NO.: PLATE C-2



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DATE: 05/07/2010	APPROVED BY: SV	PREPARED BY: DM

FULTON ROAD PAVING FROM TIDWELL TO PARKER
CBR TEST RESULTS

PROJECT NO.: HG0614605	DRAWING NO.: PLATE C-3

APPENDIX D

SUMMARY OF LABORATORY TEST RESULTS

Project: Fulton Road Paving: Tidwell to Parker
Location: Houston, Texas
Number: HG0614605

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Moisture Content (%)	Unit Weight (pcf)	Shear Strength (UC) (tsf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-1	2									0.5
B-1	3	24	17	7	56	16.2				
B-1	4									1
B-1	5					16				
B-1	6									0.5
B-1	7					18.6	133.1	0.57		
B-1	9	34	14	20	76	19.1				
B-1	13									1.33
B-1	14					17.5	121.9	1.24		
B-2	1					18.5				
B-2	1.4									0.25
B-2	2									0.17
B-2	3	24	17	7		19.9	129.6	0.45		
B-2	4									0.83
B-2	5	34	15	19	68	17.3				
B-2	6									0.58
B-2	7					16.7	122	1.15		
B-2	8									1
B-2	9	39	14	25		16.4				
B-2	13									1.25
B-2	14					17.4	135.7	2.22		
B-3	1					14.1				
B-3	1.1									0.67
B-3	2	28	16	12						0.33
B-3	3					20.1	131.9	0.4		
B-3	4									0.5
B-3	5	35	17	18	91	18.5				
B-3	6									0.25
B-3	7					18.6	121.9	0.71		
B-3	8									1.5
B-3	9					14.7				
B-3	13									1.5
B-3	14					18	120.9	1.73		
B-4	0.7									0.25
B-4	1					19.2	130.5	0.47		
B-4	2									0.33
B-4	3	27	16	11	54	15.8				
B-4	4									0.67
B-4	5					16.5	127.6	0.79		
B-4	6									0.83
B-4	7	35	15	20		17.4				
B-4	8									1.33
B-4	9					16.5	120.3	0.94		
B-4	13									1.25
B-4	14				79	17.5				
B-4	18									1.42
B-4	19					21.2	136.2	2.14		

Project: Fulton Road Paving: Tidwell to Parker
Location: Houston, Texas
Number: HG0614605

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Moisture Content (%)	Unit Weight (pcf)	Shear Strength (UC) (tsf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-4	23									1.17
B-4	24					21.7				
B-5	0.9									0.33
B-5	1	28	18	10		17.2				
B-5	2									0.75
B-5	3					19.9	124.7	0.31		
B-5	4									0.58
B-5	5					17.2				
B-5	6									0.75
B-5	7	28	15	13	70	17.3				
B-5	8									1.33
B-5	9					18.6	131.4	1.72		
B-5	13									1.5
B-5	14				85	18.9				
B-5	18									1.5
B-5	19					20.5	123.2	1.1		
B-5	23									1.17
B-5	24					22.1				
B-6	0.9									0.08
B-6	1					20.2	118.5	0.33		
B-6	2									0.33
B-6	3	30	19	11		18.9				
B-6	4									1.5
B-6	5					15	124	0.29		
B-6	6									0.83
B-6	7	39	16	23	68	17.3				
B-6	8									0.67
B-6	9					17.3				
B-6	13									1.17
B-6	14					21.3	120	1.2		
B-6	18									1.5
B-6	19	60	23	37		24.7				
B-6	23									0.83
B-6	24					21.2	123.2	0.39		
B-6	28									1.33
B-6	29				85	16.4				
B-7	0.8									0.5
B-7	1	22	16	6		16.1				
B-7	2									0.17
B-7	3					19.4	135.5	0.19		
B-7	4									0.33
B-7	5					21.5				
B-7	6									0.58
B-7	7					20.2	137.3	0.32		
B-7	8									1.33
B-7	9	36	13	23		17.4				
B-7	13									1.42

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Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Moisture Content (%)	Unit Weight (pcf)	Shear Strength (UC) (tsf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-7	14					18.9	121.6	1.94		
B-7	18									1.5
B-7	19					20.4				
B-7	24	62	26	36	92	29.9				
B-7	28									0.33
B-7	29					27				
B-8	1									0.5
B-8	1				62	12.8				
B-8	2									0.17
B-8	3					22.1	129.8	0.21		
B-8	4									0.75
B-8	5	30	12	18		16				
B-8	6									0.58
B-8	7					16.4	125.3	0.76		
B-8	8									0.75
B-8	9					20.4				
B-8	13									1.33
B-8	14					18.6	136.5	2.57		
B-8	18									1.33
B-8	19	55	22	33	91	22.9				
B-8	23									0.58
B-8	24					18.1	135.2	0.48		
B-8	29					19.9				
B-9	1				63	12.5				
B-9	3				60	18.9				
B-9	5	20	18	2	64	18.8				
B-9	7					18.2				
B-9	8									0.67
B-9	9					20.3	134.5	0.55		
B-9	13									1.33
B-9	14	49	18	31		21.1				
B-9	18									1.33
B-9	19					20.1	125.8	1.74		
B-9	23									1.5
B-9	24					18.8	122.7	2.27		
B-9	28									1.5
B-9	29	40	22	18	90	19.9				
B-9	34					27.7				
B-10	1				67					
B-10	1.5					17.2				
B-10	3	20	15	5		17.6				
B-10	5				74	17.2				
B-10	7					19.4				
B-10	9	34	15	19		21.3				
B-10	13									0.5
B-10	14					20.3	124.4	0.54		
B-10	18									1.5

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Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Moisture Content (%)	Unit Weight (pcf)	Shear Strength (UC) (tsf)	Shear Strength (UU) (tsf)	Shear Strength (Pocket Pen) (tsf)
B-10	19	48	20	28		18.8				
B-10	23									1.5
B-10	24					19.6	127.6		2.37	
B-10	28									0.67
B-10	29	32	17	15	74	17				
B-10	34					24.3				
B-11	1.5				62	12.8				
B-11	3				66	15.6				
B-11	4									0.08
B-11	5	22	17	5		18.9				
B-11	6									0.42
B-11	7					17.7	131.2	0.32		
B-11	8									0.5
B-11	9	27	15	12		19.6				
B-11	13									1.17
B-11	14					16.4	133.2	0.67		
B-11	18									1.5
B-11	19				80	18				
B-11	23									1.5
B-11	24					18.8	136	1.05		
B-11	28									1.5
B-11	29	42	18	24		23				
B-11	34				35	22.4				
B-12	1									0.83
B-12	1.5	35	14	21		14.9				
B-12	2									0.42
B-12	3				72	17.1				
B-12	4									0.58
B-12	5				74	16.2				
B-12	6									0.92
B-12	7					16	129.2	0.53		
B-12	8									0.42
B-12	9	31	15	16	79	17.4				
B-12	13									1.42
B-12	14					16	135.3	0.58		
B-12	18									1.5
B-12	19	58	22	36	90	21.4				
B-12	23									1.5
B-12	24				58	16.9				
B-12	28									1.5
B-12	29					23.7	128.2		1.51	
B-12	33									1.5
B-12	34					22.9				
Total		**32**	**32**	**32**	**29**	**100**	**37**	**35**	**2**	**83**