FINAL REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED WATER LINE ALIGNMENT
ALONG LINK ROAD
FROM AIRLINE TO FULTON
ACCELERATED SWTP CONTRACT 6C-1
GFS NO. S-0900-64-3, FILE NO. WA10637
HOUSTON, TEXAS

FOR

THOMPSON PROFESSIONAL GROUP, INC.
6110 CLARKSON LANE
HOUSTON, TEXAS 77055

PREPARED BY

ASSOCIATED TESTING LABORATORIES, INC.
HOUSTON, TEXAS
MARCH 2001

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ESTABLISHED 1959

Date: March 16, 2001

ATL Job No: G00-801

Thompson Professional Group, Inc.
6110 Clarkson Lane
Houston, Texas 77055

Attention: Mr. David Kubala, P. E.

Reference: Final Report
Geotechnical Investigation
Proposed Water Line Alignment
Along Link Road
From Airline to Fulton
Accelerated SWTP Contract 6C-1
GFS No. S-0900-64-3, File No. WA10637
Houston, Texas

Dear Sir:

Submitted herewith is our final report for the Geotechnical Investigation at the above referenced location. Our findings, analysis and recommendations are submitted herein.

It has been a pleasure working with you on this project. Should you have any questions concerning this project work, please call us at (713) 748-3717.

Sincerely,

ASSOCIATED TESTING LABORATORIES, INC.

Jay Vaghela, P.E.
Project Engineer

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Project Manager

STATE OF TEXAS
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GEOTECHNICAL INVESTIGATION

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EXECUTIVE SUMMARY

Associated Testing Laboratories, Inc. (ATL) has conducted a Geotechnical Investigation for the proposed eighty four (84)-inch water line along Link Road from Airline to Fulton in Houston, Texas. The purpose of this study was to determine subsurface conditions in the project area.

A total of nine (9) soil borings were drilled in the project area to a depth ranging from 5- to 55-feet each. One (1) boring was drilled to a depth of 55-feet. One (1) boring was drilled to a depth of 50-feet. One (1) boring was drilled to a depth of 45-feet. One (1) boring was drilled to a depth of 35-feet. One (1) boring was drilled to a depth of 30-feet. One (1) boring was drilled to a depth of 25-feet and three (3) borings were drilled to a depth of 5-feet each. Groundwater was measured during drilling and at 24 hours of drilling and was observed at depths ranging from 10- to 24-feet. At the piezometer location, ground water was encountered below the depth of 14-feet at boring GB-6 (PZ-1). ATL's subsurface investigation disclosed the following details regarding the subsurface soil types along the proposed project alignment:

A – COHESIVE SOILS:

Cohesive soils are present in the subsurface throughout the project alignment. Soft to very stiff fill sandy clay (CL) soils were encountered at borings GB-5, GB-6, GB-9 to a depth ranging from two- to 15-feet. Fill silty sand (SM) soils were encountered at boring GB-9 between the depths of six- to 10-feet. Gravel and asphalt fill was encountered to a depth of about two-feet at boring GB-7. Firm to very stiff sandy clay (CL) soils were encountered from the ground surface to varying depths ranging up to 33-feet. Very stiff sandy clay (CL) soils were again encountered below the depth of 53-feet and extending to the maximum depth of boring GB-5 to 55-feet. Firm to very stiff clay (CH) soils were encountered at borings GB-3, GB-5, GB-6, GB-7 and GB-9 below the depth of two-feet and extending to depths up to 53-feet.

B – GRANULAR SOILS:

Non-cohesive granular soils were encountered at boring locations GB-5 and GB-6 along the project alignment. Medium dense to dense silty sand (SM) were encountered between the depths of 28- to 38-feet at boring GB-6. Medium dense clayey silt (CL-ML) soils were encountered at boring GB-5 between the depths of 23- to 33-feet.

CONCLUSIONS AND RECOMMENDATIONS

Based on the field investigation, laboratory testing, records and document review, the conclusions and recommendations are summarized as below:

- A preliminary fault study based on review of available fault maps does not indicate any known fault located in immediate vicinity to the proposed project alignment. A detailed fault investigation study was not in the scope of our work.

- Dewatering in cohesive and semicohesive soils to a depth of 15-feet, can usually be accomplished by sump and pump arrangements because the seepage is relatively slow. However, since the excavations will be deeper than 15-feet, well point dewatering or other suitable dewatering measures will be required.

- Trench excavation will be used for this project along sections where water line is placed by constructing open cut trenches and for tunneling access shafts where tunneling is used. The earth cuts will require a suitably designed trench protection system if the trenches are deeper than five-feet. Deeper trenches can be made using open slopes, stepped back to stable slope, vertical cuts supported with sheet piles or other suitably designed retaining system. Selection of the trench protection method is the contractor's responsibility.

- Trench shields, if used for the water line trenches or tunneling access shafts may be designed for a lateral earth pressure equivalent to a fluid pressure of 96 PCF for cohesive soils below the water table and 65 PCF for cohesive soils above the water table. In non-cohesive soils, the trench shields should be designed for a lateral earth pressure equivalent to a fluid pressure of 85 PCF below the water table and 45 PCF above the water table. At the Interstate 45 crossing location, the trench shield may be designed for a lateral earth pressure equivalent to a fluid pressure of 130 PCF for cohesive soils both above and below the ground water. At this location for sandy soils, the lateral earth pressure may be taken equivalent to a fluid pressure of 96 PCF below the water table and 65 PCF above the water table. In general, a surcharge magnitude of q psf will result in lateral earth pressure of 0.5q in cohesive soils and 0.33q in sandy soils. At the Interstate 45 location, these values may be taken as 1 and 0.5, respectively. For the trench supporting system, the lateral pressures exerted by surrounding soils are presented in Figures 6A and 6B.

- The water line will be installed using open trench or tunneling techniques. The tunneling is likely to be performed in sandy clay, clay, silty sand, clayey sands and clayey silt soils. Recommendations for tunneling are given in section 5.4 of the report.

- In areas where the water lines are installed using open-cut excavations, the bedding criteria for the water lines and the backfilling of trenches should be in accordance with the standard SWTP specifications and requirements.

I. FACTUAL DATA

1.0 INTRODUCTION

1.1 General

This investigation for the Accelerated Surface Water Transmission Program (ASWTP), Contract 6C-1, GFS No. S-0900-64-3 and Water File No. WA10637 was authorized by Thompson Professional Group, Inc., with the acceptance of the **Associated Testing Laboratories, Inc.** proposal no. GP00-1201 dated December 11, 2000. Project details were provided to ATL by Mr. David Kubala, P.E., and Mr. Ryan Simper, P.E., of Thompson Professional Group, Inc. This report includes results of the field investigation, laboratory testing, geotechnical engineering analysis, and recommendations for the proposed design and construction of water lines and paving.

1.2 Location and Description of the project

The project alignment is located along Link Road from Airline to Fulton in Houston, Texas. A general site vicinity map of the project alignment is shown on Figure 1. It is planned to construct approximately 4,000 linear feet of new 84-inch diameter water mains. We understand that the new water line will be placed at a depth ranging from about eighteen- to 30-feet generally, with tunnel section at about 42 -feet at tunnel location at Little White Oak Bayou and possible tunnel section at about 32-feet at the Interstate 45 intersection. We understand that both alternatives of using open trench method or tunneling is presently being considered at the Interstate 45 intersection.

The Link Road alignment is generally along asphalt street. Little White Oak Bayou crosses the project alignment near Enid. The project alignment goes underneath the Interstate 45. The project alignments are generally adjacent to residential properties. Some commercial businesses are located along the project alignment. Photographs of the project site were taken at the time of our site visit. These photographs are presented in Appendix 1.

1.3 Scope of Work

A geotechnical investigation has been conducted to determine subsurface soil conditions in the proposed project area and to develop geotechnical engineering recommendations for the construction of the new water line along the project alignment.

Associated Testing Laboratories, Inc. (ATL) has completed a subsurface exploration program consisting of the following scope:

- Drilling and sampling a total of nine (9) soil borings to depths ranging from five (5) to fifty five (55) feet below the existing ground surface level. The boring depths and locations were approved by the client. One (1) boring was drilled to a depth of fifty five (55) feet. One (1) boring was drilled to a depth of fifty (50) feet. One (1) boring was drilled to a depth of forty five (45) feet. One (1) boring was drilled to a depth of thirty five (35) feet. One boring was drilled to a depth of thirty (30) feet. One (1) boring was drilled to a depth of twenty five (25) feet. Three (3) borings were drilled to a depth of 5-feet each along the project alignment. One (1) boring (boring GB- 6) was later converted into a piezometer (PZ-1). In addition, bucket samples were obtained from two locations along the project alignment for performing California Bearing Ratio (CBR) and Proctor test.

- Laboratory testing on selected soil samples recovered from the soil borings and the bucket samples taken from the surface at two locations.

- Developing boring log profiles to assess subsurface soil and groundwater conditions.

- Preliminary fault study (ASCE Phase I) of the proposed project area based on the review of available fault maps. It should be noted that a detailed investigative fault study is beyond the scope of this report.

Based on results from the field investigation, laboratory testing and gathered geological information, ATL performed an engineering analysis to develop geotechnical recommendations for the installation of the new water lines. It should be noted that a phase I environmental site assessment study was not in our scope of work.

2.0 SUBSURFACE INVESTIGATION PROGRAM

The field investigation consisted of drilling and sampling of a total of nine (9) soil borings in the project area. Coring of the existing pavement was done prior to drilling and sampling. The information from our coring operations is presented in section 4.3.

Boring locations as drilled for this geotechnical exploration are shown in Figure 2. One (1) of the soil boring was drilled to a depth of fifty five (55) feet. One (1) of the boring was drilled to a depth of fifty (50) feet. One (1) boring was drilled to a depth of forty five (45) feet. One (1) boring was drilled to a depth of thirty five (35) feet. One (1) boring was drilled to a depth of thirty (30) feet. One (1) boring was drilled to a depth of twenty five (25) feet. Three (3) borings were drilled to a depth of five (5) feet. **It should be noted that underground obstruction was encountered at previously planned location of boring GB-6 near the bayou high bank.** Several attempts were made to drill around this location but failed. Subsequently this boring was shifted to the street. One (1) of these boring (boring GB- 6) was later converted into a piezometer (PZ-1). The structure of the piezometer well is shown in Figure 3. The total footage drilled and sampled was 255 feet. The boring depths and locations were approved by the client. Dry auger drilling methods were adopted to drill the soil borings till the encountering of water. In cohesive soils, undisturbed soil samples were collected using a conventional 3-inch O.D. Shelby tube. Cohesionless soils were sampled using split spoon sampler. All soil samples were examined, classified and logged by a geotechnical technician. A representative portion of each sample was packed in containers to prevent moisture loss. All soil samples were properly labeled and subsequently transported to the ATL laboratory. All soil samples were classified according to Unified Soil Classification System (ASTM D-2847). A key to soil classifications and symbols used in this report is presented in Appendix 2.

No unusual staining or hydrocarbon odors were encountered during the visual inspection of the soil samples.

3.0 LABORATORY TESTING PROGRAM

Laboratory testing was performed on selected representative soil samples that were collected during the field investigation. The laboratory testing program included Atterberg Limits (ASTM D-4318), Density, Moisture Content (ASTM D-2216), Unconfined Compressive Strength (ASTM D-2166), Particle Size Analysis (ASTM D-422), California Bearing Ratio (CBR) and Proctor tests. The lab testing results are presented on the boring logs in Appendix 3 and summarized in tables of Appendix 4. Overall numbers and types of tests performed for this study are presented below:

TYPE OF TEST	NUMBER OF TEST
Dry Density	24
Moisture Content	69
Atterberg Limits	24
Unconfined Compression	24
Sieve Analysis thru #200	10
California Bearing Ratio	2
Maximum Dry Density and Optimum Moisture	2

II. INTERPRETIVE REPORT

4.0 SUBSURFACE AND SITE CONDITIONS

4.1 Geology of Coastal Plain

The proposed project area is located within the Gulf Coast Structural Province, a huge sedimentary basin containing several thousand feet of sediments. In general, these sediments consist of loose sands, silts and clays which slope gently toward the Gulf of Mexico.

The proposed project site is underlain by the Beaumont Formation of the Pleistocene age. This formation consists of over consolidated clays, silts and sands with some shell, calcium carbonate and iron oxides. These formations are quite strong and extend to an approximate depth of 100 feet. The surface materials are often weakened by the weathering process.

The materials of Beaumont Formation were deposited during the last of the interglacial periods. During interglacial periods when water from the melting glaciers flowed back into the ocean, the sea rose, the depended valley backfilled and several Pleistocene formations were deposited. Beaumont Formation may have been deposited during a mid-Wisconsin interglacial interval or during the Sangamon Stage, an interval between the Wisconsin and Illinoian Glaciations. The Sangamon Stage is currently estimated as taking place about 70,000 years ago. The Beaumont formation is the youngest formation of Pleistocene age that crops out in the proposed project area. Its origins are mainly fluvial and deltaic, but probably some small areas originated as coastal marsh and lagoonal deposits.

4.2 Natural Hazards

Among the geologic and geomorphological features in this region are sedimentary deposits broken by structure such as normal faults, salt domes, etc. The sedimentary deposits slope gently toward the Gulf of Mexico. They are broken by normal faults, most of which dip toward the Gulf and extend downward many thousands of feet. The earth movements that caused these faults took place within the last 50,000 years. In general, the regional faults in the Houston area trend parallel to the Gulf Coast. Only the local faults over the salt domes show a radial pattern associated with the up thrust of the salt mass.

There are numerous faults and fault systems in the greater Houston area. The movement of many of these faults has been affected in recent history by area subsidence. The subsidence is caused by removal of oil and groundwater. As much as nine (9) feet of subsidence has taken place in the area east of Houston in the last 70 years. Based on the review of the subsidence contour shown in Figure 4, four- to five-feet of subsidence has occurred in the last 70 years in the area of the project alignment. Conversion to surface water usage and the limiting of oil production has greatly reduced the subsidence rate in Houston.

A preliminary fault study (ASCE Phase I) of the Geological Atlas of Texas (Houston Sheet), a principal active fault map, and Land-surface Subsidence Maps does not indicate the potential presence of any known fault in the close vicinity of the proposed project area. However, it should be noted that a detailed investigative fault study is beyond the scope of this Geotechnical Exploration. Figure 4 shows the principal active faults and subsidence in the Houston area.

4.3 Site Stratigraphy and Geotechnical Characterization

A brief description of various soil types and the depths from the ground surface in the project area based on the information obtained from our borings are presented below:

4.3.1 Cohesive Soils

Cohesive soils are present in the subsurface throughout the project alignment from the ground surface to the maximum depth of the borings at 55-feet. Soft to very stiff fill sandy clay (CL) soils was encountered at borings GB-5, GB-6, GB-9 to a depth ranging from two- to 15-feet. Fill silty sand (SM) soils were encountered at boring GB-9 between the depths of six- to 10-feet. Gravel and asphalt fill was encountered to a depth of about two-feet at boring GB-7. Firm to very stiff sandy clay (CL) soils were encountered at all boring locations from below the pavement to varying depths ranging up to 33-feet. Very stiff sandy clay (CL) soils were again encountered below the depth of 53-feet and extending to the maximum depth of boring GB-5 too 55-feet. Firm to very stiff clay (CH) soils were encountered at boring GB-3, GB-5, GB-6, GB-7 and GB-9 below the depth of two-feet and extending to depths up to 53-feet. The sandy clay soils were found to have a liquid limit ranging from 24 to 48, a plastic limit ranging from 15 to 18 and a plasticity index ranging from 9 to 30. These soils are non - to moderately expansive. The non-expansive soils in their present condition are suitable for use as select fill. The moderately expansive sandy clay soils are not suitable for use as select fill material in their present condition. These soils, once lime-stabilized (5% by dry weight) are suitable for use as select fill material. However, these soils in their present condition are suitable for use as random fill material in the trench zone (outside pavement areas). The clay soils were found to have a liquid limit ranging from 50 to 71, a plastic limit ranging from 19 to 21 and plasticity indices ranging from 31 to 49. These soils are expansive and not suitable for use as select fill material. The expansive soils once lime-stabilized (7% by dry weight) should be suitable for use as select fill material. These soils in their present condition are suitable for use as random fill material in the trench zone (outside pavement areas).

We reviewed the boring logs furnished by Thompson Professional Group, Inc., for the location at Interstate 45 and Link Road (borings 979, 980 and 985). Based on these 1959 boring logs, the soils are very stiff to hard sandy clay and silty clay to the maximum depth of the borings at 62-feet. At borings 980 and 985 layers of silty sand, clayey sand and clayey silt were encountered at varying depths between 23- to 40-feet below the then existing grade. More information can be obtained from the attached boring logs and location of borings in Appendix 5.

4.3.2 Granular Soils

Granular soils were encountered at boring locations GB-5 and GB-6 along the project alignment. Medium dense clayey silt (CL-ML) soils were encountered between the depths of 23- to 33-feet at boring GB-5. Medium dense to dense silty sands (SM) were encountered between the depths of 28 – to 38-feet at boring GB-6. The silty and sandy soils are moisture sensitive, compressible and difficult to compact in a wet condition (they may pump). These soils are not suitable for use as select fill material.

The letters in parenthesis indicate soils classification in accordance with Unified Soils Classification System. A more detailed stratigraphy is presented in boring logs, GB-1 through GB-9 in Appendix 3. Definition of terms and a key to symbols used in the boring logs are presented in Appendix 2. Boring log profile maps were developed based on the boring locations and the subsurface soils encountered in each boring. The boring log profiles are presented in Figure 5. Coring of the existing pavement at the boring locations was performed prior to drilling. The existing pavement thickness is shown in the table below:

Boring No.	Location	Asphalt Paving	Concrete Paving	Base
GB-1	Station 10+ 00	5"	--	4" lime stabilized soil
GB-2	Station 15+ 00	3"	--	2" lime stabilized soil
GB-3	Station 20+ 00	2"	--	6" lime stabilized soil
GB-4	Station 25+ 00	4"	--	5" lime stabilized soil
GB-5	Station 30+ 78	--	--	--
GB-6	Station 33+ 80	10.5"	--	4" shell
GB-7	Station 39+ 80	--	--	--
GB-8	Station 44 + 96	2"	--	9" lime stabilized soil
GB-9	Station 49 + 58	2.5"	--	4" lime stabilized soil

Potentially Petroleum Contaminated Area (PPCA) were not observed during our field exploration. We did not encounter petroleum or any other foul/unusual odors in the soil samples. However, it should be noted that it is possible that there may be PPCA which were not identified through our borings or that may be present at locations in between our borings.

4.4 Groundwater

Groundwater conditions were observed in open soil borings during the field investigation and at 24 hours of drilling. Groundwater was observed in the piezometer at three days after drilling. Groundwater was encountered during drilling operations at the boring locations ranging from 10 to 24 feet below the existing ground surface. Groundwater was not encountered at some boring locations at the time of drilling. At 24 hours of drilling, groundwater was encountered at depths ranging from 13.3- to 24.3-feet. One (1) of the soil boring (GB-6) was converted into piezometer (PZ-1) and water level measured at 3 days after drilling. Groundwater was measured at depth of about 14-feet at the piezometer location.

Predominantly clay/silty clay soil contains water due to lenses and seams of more permeable soils such as silty sand or sandy silt. The rate of flow of groundwater produced by these layers will depend upon the weather conditions such as amount of precipitation and ambient temperature etc. at the time of construction. It should also be noted that the groundwater level is generally influenced by such factors as topography and surface drainage features.

It should be noted that a detailed hydrogeological investigation of the proposed project area is beyond the scope of this investigation. Groundwater (Gw) depths measured during drilling, at 24 hours of drilling and after three days (at piezometer locations) are shown in the table below.

Boring Number	**Location**	**Gw during drilling**	**Gw at 24 hrs.**	**Gw on 01-22-2001 in piezometers**
GB-1	Station 10+ 00	17'	24'3'	--
GB-2	Station 15+ 00	--	--	--
GB-3	Station 20+ 00	22'	18'4"	--
GB-4	Station 25+ 00	--	--	--
GB-5	Station 30+ 78	24'	--	--
GB-6 / PZ-1	Station 33+ 80	--	--	14'
GB-7	Station 39+ 80	11'	18'2"	--
GB-8	Station 44 + 96	--	--	--
GB-9	Station 49 + 58	10'	13'3"	--

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

5.1 Trench Excavation

At the federal level, Occupational Safety and Health Act (OSHA) requires protective systems for all trenches exceeding 5 feet in depth. OSHA has developed a soil classification system to be used as a guideline in determining sloping and protective system requirements for trench excavations. This system has set forth a hierarchy of Stable Rock, Type A, Type B, and Type C, in decreasing amounts of stability.

Stable Rock: Natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed.

Type A: Cohesive soils with an unconfined compressive strength of 1.5 ton per square foot (tsf) or greater.

However, no soil is Type A if:

- The soil is fissured; or
- The soil is subject to vibrations from heavy traffic, pile driving, or similar effects; or
- The soil has been previously disturbed; or
- The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four (4) horizontal to one (1) vertical or greater; or
- The material is subject to other factors that would require it to be classified as a less stable material.

Type B:

- Cohesive soil with an unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf; or
- Granular Cohesionless soils, including angular gravel, silt, silty loam, and sandy loam, and in some cases, silty clay loam and sandy clay loam; or
- Previously disturbed except those which would otherwise be classified as Type C; or
- Soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration; or
- Dry rock that is not stable; or
- Material that is part of a sloped, layered system where the layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.

Type C:

- Cohesive soil with an unconfined compressive strength of 0.5 tsf or less; or
- Granular, including gravel, sand, and loamy sand; or
- Submerged soil or soil from which water is freely seeping; or
- Submerged rock that is not stable;. or
- Material is a sloped, layered system where the layers dip into the excavation on a slope of four (4) horizontal to one (1) vertical or steeper. Under the assumption that appropriate groundwater control measures are carried out, and the groundwater table, if present, is lowered and maintained at least 3 feet below the excavation depths, the stable cohesive soils (CL & CH), with unconfined compressive strength greater than 0.5 tsf, are classified as OSHA soil Type "B". The granular soils, which are less stable, are classified as OSHA soil Type "C".

Based upon the soil conditions revealed by the borings, ATL recommends the use of OSHA soil classification Type "B" for the determination of allowable maximum slope or selection and design of the protective system to a depth of about 17-feet in the area of borings GB-1 through GB-5. Below this depth, the soils should be considered as Type "C". In the area of borings GB-6 through GB-9, the soils should be considered as Type "C". The recommended short term stable slope for OSHA Type "B" soil is 1.5 Horizontal to 1 Vertical. For OSHA Type "C" soil, the recommended stable slope is 2 Horizontal to 1 Vertical. Groundwater should be adequately controlled at all times.

The proposed water line alignment in the project area will be constructed using either open trench or tunneling method. We understand that the water line will be placed at depths ranging from eighteen- to 30-feet generally, with tunnel sections ranging up to 42 -feet at tunnel location under Little White Oak Bayou and up to 32-feet at the tunnel location under the Interstate 45. Hence, the open-cut excavations for water line and the tunneling access shafts will be deeper than five-feet from the street level and will require protective measures. The trench and access shafts excavations can be made using open slopes, stepped back to stable slope, vertical cuts supported with sheet piles or other suitably designed retaining system. The excavation should be performed in accordance with the current OSHA 29 CFR Part 1926 of OSHA (Trench Safety System).

Water line open-cut trenches or access shafts trenches or pits should be provided with a proper trench support system. The trenches should be provided with a temporary shoring system, if it is deeper than five-feet. Timber shoring as outlined in 29 CFR Part 1926 of OSHA may be used in the construction of the trench supporting system. Trench boxes are commonly used for trench safety without shoring or bracing in open-cut excavations with vertical walls. In all cases, excavations should conform to OSHA guidelines.

For the trench supporting system, the lateral pressures exerted by surrounding soils are presented in Figure 6A. For any trench supporting system close at the I-45 bridge location, the lateral pressures may be taken as given in Figure 6B. In case that a trench shield is used, the trench shield may be designed for a lateral earth pressure equivalent to a fluid pressure of 96 PCF for cohesive soils below the water table and 65 PCF for the cohesive soils above the water table. In non-cohesive soils below water table, the trench shield should be designed for a lateral earth pressure equivalent to a fluid pressure of 85 PCF. For non-cohesive soils above the water table, the trench shield should be designed for a lateral earth pressure equivalent to a fluid pressure of 43 PCF. At the Interstate 45 bridge crossing location, the lateral pressures exerted by surrounding soils may be taken as 130 pcf in cohesive soils both above and below the water table. In sandy soils, these pressures may be taken as 96 PCF below the water table and 65 PCF above the water table. In general, a surcharge magnitude of q psf will result in lateral earth pressure of 0.5q in clayey soils and 0.33 q in sandy soils. At the Interstate 45 bridge crossing location, the surcharge magnitude of q psf may be taken as resulting in lateral earth pressure of q in clayey soils and 0.5q in sandy soils. For the calculations of the above given earth pressures, unit weight of 130 pcf was assumed along with a coefficient of active earth pressure Ka of 0.5 in cohesive soils and 0.33 in sandy soils. At the Interstate 45 bridge crossing location, the value of coefficient of earth pressure K was assumed as 1 in cohesive soils and 0.5 in sandy soils. To illustrate better, an example calculation for the cohesive soils below the water table is shown below:

Unit weight of cohesive soils = 130 pcf, Unit weight of water = 62.4 pcf

Submerged (bouyant) weight of cohesive soils = 130 – 62.4 = 67.6 pcf

Now, total earth pressure in pcf = pressure from soil + hydrostatic pressure

= Ka * Submerged weight of soil + unit wt. of water

= 0.5 (67.6) + 62.4 = 96.2 pcf say 96 pcf

Timber shoring as outlined in 29 CFR Part 1926 of OSHA recommendation may be used in the construction of trench supporting system.

Due to the presence of the roadway adjacent to the likely excavation areas at the project site, the effect of vehicular traffic may be considered while designing the lateral supporting systems. Boussinesq's equation should be used for calculating the loads on the retaining systems due to the vehicular traffic. We recommend that a HS20 vehicle loading be considered adjacent to the pit for design purposes. An impact factor of 1.5 should be used in the design. Surcharge loading due to construction machinery should be considered as applicable.

Stockpiling of excavated material may not be allowed near the excavation. Generally, a distance of one half the excavation depth on both sides of the trench should be kept clear of any excavated material. If this is not possible due to space limitations then the retaining system design should take into account the surcharge loads.

Care is urged during excavations since conditions such as sloughing or caving of the excavation trench or excavation slope may result in movement of the surrounding soils resulting in possible settlement of the surrounding structures or features. Additionally, at Interstate 45 crossing location, one of the alternative for the water line under consideration is to install it by using open trench method. We understand that the water line will be about six- to seven-feet from the bridge pier. Since, the water line will be farther than five-feet, we do not anticipate any significant effect on the side resistance capacity of the drilled pier. However, it should be noted that the side resistance capacity of the bridge pier may be affected if conditions such as caving or sloughing of the excavation side occurs during trench excavations. Hence, we recommend that effective trench retention measures be planned and monitored. The retention should be as rigid as possible and designed for the higher earth pressures as given in this report at the Interstate 45 location.

Based upon our groundwater investigations seepage may cause problems during excavation of trenches deeper than 10- to 14-feet. Seepage of water may also occur at shallower depths if fluctuations in ground water level takes place. The flow of ground water may vary depending upon depth of construction and weather conditions. A conventional sump and pump arrangement can be used for the shallow trench excavations up to fifteen-feet in cohesive soils. For deeper trench/pit excavations and in non-cohesive soils, multi-stage pumps, well points or educators will be required. Based on the drawings provided by Thompson Professional Group, Inc., we understand that the water lines installed by open trench excavation will be placed at depths ranging from about 18- to 30 -feet and the water lines placed by tunneling will be at depths ranging from 32 - to 42 -feet below existing grade. Hence it appears that the water lines installed by open-trench excavations will be placed in firm to very stiff sandy clay and very stiff clays, generally below the water table. The water lines installed using tunneling will be in firm sandy clays, very stiff clays, medium dense to dense silty sands and clayey silts below the water table. Hence, it appears that dewatering will be required for open trench excavations. Dewatering will also be required for the trench excavations for access shafts at tunneled section. Since the excavation depth is deep, dewatering should be undertaken using well points or educators. In stable cohesive soils, the trench bottom stability can be evaluated in the following manner.

If sheeting terminates at the base of cut:

$$\text{Factor of safety } (F_s) = \frac{(N_c)\,C}{(\Upsilon)\,H+q}$$

N_c = Bearing Capacity factor which depends on dimensions of the excavation:(width), (length) and (depth) (Use H=Z and N_c = 5.7)

C = Undrained shear strength of clay in failure zone beneath and surrounding base of cut (may be taken as half the unconfined compressive strength)

Υ = Unit weight of surrounding soils (use 130 pcf)

q = Surface surcharge.

If the factor of safety is less than 1.5, sheeting should be extended below the base of the cut to insure stability.

$$\text{Extended Sheeting Depth (D)} = \frac{1.5\,(\Upsilon H + q) - N_c C}{(C/b) - 0.5\Upsilon}$$

for $D \geq$ 5-feet

Shown below is our example calculation for the bracing pressures. Say for access shaft trench excavations near boring GB-5, soil consist of stiff to very stiff sandy clays to the depth of about 23-feet underlained by clayey silt soils to a depth of about 33-feet underlained by cohesive clays till the maximum depth of the boring at 55-feet. Groundwater was encountered during drilling below 24-feet. Say the access shaft is 48-feet deep. Using Figure 6, Pa equals 0.4 YH in cohesive soils and 0.25 YH in sandy soils, take H equal to 48-feet and Y in soils of 130 pcf and submerged Y in soils of 67.6 pcf. Since groundwater is at 24-feet and excavation depth is 48-feet, the average Y in soils may be taken as (130*24 + 24*67.6) / 48 = 98.8 pcf. Hence using Figure 6, the pressure Pa at top of trench is zero, at depth of 0.25 H (12-ft), Pa is 0.4*98.8*48 = 1896.96 psf. At depth of 36-feet, Pa is 1896.96 psf, at depth of 48-ft Pa = 0. Hydrostatic pressure of 62.4 times water depth should be added below 24-feet. A surcharge pressure of 0.5*500 should be added from the top.

5.2 Excavation Dewatering

Groundwater was found at depths ranging from 10- to 24-feet. At 24 hours of drilling, groundwater was measured at depths ranging from 13.3- to 18.2-feet. At piezometer location PZ-1 (Boring GB-6) groundwater was measured at depth of about 14 –feet at three days after drilling. For construction below the groundwater table, precautions should be taken to control the groundwater since the presence of groundwater destroys the cohesion of the soil (thus reducing the angle of repose) and can separate and wash away individual particles. In non-cohesive soils, a total collapse or caving in of the soils will occur.

Dewatering in cohesive and semi cohesive soils can usually be accomplished by sump and pump arrangements because the seepage is relatively slow. For shallow dewatering to a depth of about fifteen (15) feet, vacuum well points may be adapted. For dewatering below this depth and in non-cohesive soils, deep wells with submersible pumps or educators would be preferable. Based on the information on drawings provide by Thompson Professional Group, Inc., it appears that the pipelines installed using open-trench excavations will be at depths ranging from 18- to 30-feet and that installed using tunneling will be at depths ranging from 32- to 42-feet. Groundwater was encountered along the project alignment below depths of 13-feet. Since the depth of excavations are deep, dewatering using well points or educators will be necessary. The selection of the appropriate dewatering system for the project is the contractors responsibility.

Seams and pockets of sands, silt, ferrous nodules, and calcareous nodules that exist in the shallow cohesive soil layers may pose a threat if they form a drainage path for the groundwater and as a result, accelerate the rate of seepage. Also, in non-cohesive soil layer, the groundwater seepage will occur at a high rate. Hence, during the trench or tunnel access shaft excavation and construction, appropriate measures, such as proper dewatering and shoring methods, will have to be implemented under supervision of a Professional Civil/Geotechnical Engineer.

5.3 Vehicular Traffic and Railroad Loads

The trench bottom for water line placement should be over-excavated to a minimum of 6 inches. The space should be filled with bank sand and compacted to a minimum of 95 percent of the maximum Standard Proctor density (ASTM D558) with a moisture content of -3% to +5% of the optimum moisture content. Over-excavation of trench bottoms will be required for wet soils below the depth of groundwater. The trench bottom should be shaped to receive the 84-inch diameter pipe. The bedding details should be in accordance with the standard SWTP specifications.

The annular space between the pipe and the trench should be backfilled with bank run sand placed in 6 to 8 inch loose lifts and compacted to a minimum of 95 percent of the maximum dry density as determined by Standard Proctor test (ASTM D 698) at – 3 to +3 percent of the optimum moisture content. The backfill should conform to standard SWTP Specifications.

The pipelines placed at depths under the ground will be subjected to loads due to backfill (earth loads) and loads due to vehicular traffic and railroad loads (live loads). These loads can be calculated based on Marston's and Boussinesq formulas. A unit weight of 130 pcf can be used for the calculation of these loads. A value of $K\mu$ equal to 0.132 and 0.11 can be taken in cohesive soils above water table and cohesive soils below water table, respectively. In sand this value can be taken as 0.165. Here K is the active earth pressure coefficient and μ is the coefficient of sliding friction between the fill material and the sides of the trench. The height of the fill and the horizontal width of trench should be considered from the top of the conduit. The load on a rigid conduit, Wd in lb/ft can be calculated using the following equation:

$$Wd = Cd*\Upsilon*(Bd)^2$$

Here, Υ is the total unit weight, Bd is the width of trench. Cd is the load coefficient which can be obtained from the attached Figure 7.

For calculation of live loads, the width of the loaded area should be taken as the outside horizontal width of the pipe. Loading due to HS20 vehicle should be considered for vehicular traffic. We understand that the pipelines will be placed at a depth ranging from 18 - to 30-feet generally with depths up to 32- feet at tunnel location under Little White Oak Bayou and up to 42-feet at Interstate 45 crossings. Pipelines placed at 5, 6, 7 and 8-feet, respectively are likely to get an additional loading due to HS 20 vehicle of about 400, 250, 176 and 100 psf, respectively. Loading due to HS20 vehicle on pipelines placed deeper than 8-feet is negligible and can be neglected for design purposes.

An example is shown here as a sample. At boring GB-3, the depth of pipe placement is about 26-feet. The soil is cohesive sandy clay. The pipe width is 84-inch. The top of the pipe from the ground surface is about 19-feet and say the width of open trench is 10-feet.

Hence $H/Bd = 19/10 = 1.9$, $K\mu = 0.13$

From Figure 7, $Cd = 1.4$

Therefore the earth load on pipe is

$Wd = 1.4 * 130 * (10)^2 = 18200$ lb/ft.

As given in text above, the live load at a depth of 19-feet is negligible.

Therefore total load on pipe is taken as earth load + live load

= 18200 + 0 = 18200 lb/ft.

5.4 Pressures on Primary and Permanent Liners

We understand that the proposed eighty four (84)-inch water line will be installed using tunneling techniques at portions of the site on Link Road under Little White Oak Bayou crossing and possibly under Interstate 45. It is anticipated that the tunneling excavations will be performed at the depths ranging from about up to forty two (42)-feet at Little White Oak Bayou location to up to thirty two (32)-feet at the Interstate 45 crossing location.

Based on our geotechnical exploration, the tunneling will be performed in water bearing very stiff clay soils and medium dense silty sand/clayey silt soils at the tunneling location under Little White Oak Bayou and in stiff clays and firm sandy clays and/or clayey sands below the water table under the Interstate 45.

It should be noted that underground obstruction were encountered around six-feet depth at the Little White Oak Bayou high bank near the previously planned location of boring GB-6. We could not drill at these locations despite several attempts. Hence, the tunneling contractor should keep this in mind while planning for tunneling access shaft excavation and tunneling. It is our opinion that it is possible that the underground obstructions that prevented our drilling there could have been concrete debris that may have been thrown there to backfill the bayou side. We have previously encountered such conditions near bayou sides in other parts of Houston. The size and depth of the debris is difficult to guess. However as the tunnel alignment is at 42-feet, it is our opinion that it is unlikely that the debris may be present at the tunneling depth and hence is unlikely to affect the tunneling operations. However, for excavating the tunnel access shaft, concrete debris may affect in smooth operations of excavating. The contractor should realize that the size of the underground debris is not known and hence plan his contract bidding, scheduling and excavation operations accordingly.

We recommend that record search be done to see if there is any documentation of backfilling of the Little White Oak Bayou at this location. In addition, old aerial photo's (if available) may be reviewed to see if there is any indication of backfilling being done there. Aerial photographs are normally obtained and reviewed as part of Phase I environmental investigations. These photographs if available, should be reviewed as indicated above for any backfilling.

Granular soils below the groundwater level will tend to flow into the tunnel, while granular soil above the groundwater level will not stand unsupported, but will tend to ravel until a stable slope is formed at the face with a slope angle equal to the angle of repose of the material in a loose state. If some clay binder is present in the soils or if the material is sufficiently moist to exhibit some cohesiveness termed as "apparent cohesion" due to development of negative pore pressures, granular materials may be able to stand unsupported at the tunnel face for a limited period of time. However, this possible mode of face stability should not be relied on in the design. We recommend that tunnels in cohesionless soil layers be constructed using techniques which will provide positive support to the soil.

For tunneling in water bearing granular soils, the tunneling operations may consist of the closed type slurry shield machine with or without dewatering procedures. We recommend that the slurry pressure should be determined by the contractor based on the subsurface conditions and the tunnel boring machine characteristics. Slurries can be formed with bentonite mixtures or with clay-soil muck. This method minimizes subsidence by maintaining balance of earth pressures. For tunneling in cohesive soils, a conventional shield tunnel machine can be used. The tunneling contractor should determine the areas where a conventional shield tunnel machine is applicable.

The selection of the appropriate tunneling method is the contractor's responsibility. The tunneling contractor should select the tunnel boring machine / tunnel excavation technique based on the soil stratigraphy and groundwater conditions. Based on the soils encountered at the tunneling depths, a closed type slurry shield machine is recommended for tunneling under Little White Oak Bayou. Also due to the encountering of granular soils in the borings furnished by Thompson Professional Group, Inc., tunneling with a closed type slurry shield machine is recommended at the Interstate 45 location also.

Temporary tunneling shaft structures, should be designed based on the lateral earth pressures and other considerations discussed in section 5.2. Groundwater near the tunneling alignments was measured as high as 14-feet below existing grade. Dewatering will be required to provide a dry working platform.

The stress distribution at the point of the tunnel liner prior to construction is equal to:

$P_z = (\Upsilon - \Upsilon_w)\, z$

$P_h = k_o\, p_z$

where: P_z = vertical effective stress

P_h = horizontal (lateral) effective stress

Υ = unit weight of soil

Υ_w = unit weight of water

z = depth of the tunnel liner

k_o = the coefficient of earth pressure at rest and may be taken as 1.0.

The loads on the rigid tunnel liner after construction may be P_1, P_2 and P_3 as shown on Figure 8. These loads may be computed and used for the design of the liners. A example is shown below to better understand the use of the procedure shown on Figure 8.

For the tunnel section near boring GB-7, the tunnel diameter (D) is 7-feet, hence the tunnel top (H) is at a depth of about 24 feet from the existing ground surface elevation. Vehicular loading at a depth of about 24-feet is negligible. Assume additional surcharge of about 500 psf. At boring GB-7, groundwater was encountered at a depth of about 18-feet upon completion of drilling. Here groundwater depth Dw is less than H + D/2.

Hence P1 = {[(24 + 7/2) * 130 – (24 -18)*62.4] / 1000 } + 500/1000

= 3.7 ksf

P2 = (24 * 130) / 1000 + 0.5 = 3.62 ksf

P3 = {(24 + 7)*130} / 1000 + 0.5 = 4.53 ksf

5.5 Piping System Thrust Restraint

Unbalanced thrust forces result from changes in flow directions and/or velocity in a pressurized pipe system. The force acting on a pipe system is resisted by the bearing area between the pipe and the backfill soils. Adequate restraint may be achieved by using restraint joints, tie rods, or a combination of these systems. The restraint joints are employed to allow thrust and shear forces to be transmitted across the pipe joints to allow a number of pipe sections to act integrally in bearing.

Thrust blocks are not used for pipes larger than 16-inches. Hence, for this project, restraint joints are likely to be used. A detailed procedure for designing restrained joints including example calculations is outlined in the AWWA design manual M9 (Ref. 11).

In general, the frictional resistance F needed along each leg of the bend is $PA(1 - \cos\Delta)$, where P is internal pressure (lbs/sq. in), A is the cross sectional area of the pipe (sq. in) and Δ is the pipe bend angle.

The frictional resistance of the pipe against soil is equal to $f(2We + Wp + Ww)$, where f is the coefficient of friction between pipe and soil, We is the overburden load on pipe (lb/lin-ft), Wp is the dead weight of the pipe (lb/lin-ft) and Ww is the dead weight of water in pipe (lb/lin-ft). The length of the pipe L to be tied to each leg of an elbow is calculated as

$$L = PA(1 - \cos\Delta) / f(2We + Wp + Ww)$$

The following soil parameters are recommended for the design of the restrained joint(s):

Average unit weight of soil, Υ	= 110 pcf	
Cohesion of soils, C	= 1000 psf	(for clay soils)
Angle of internal friction, φ	= 30^{0}	(for sand backfill)
Coefficient of friction between pipe and soil, f	= 0.3	

A design example is shown below for illustration purposes only. Say, near Enid (boring GB-5), the pipe is 84-inches in diameter and the pipe crown is placed at a depth of about 8.5-feet. Say, the pipe bend angle is 45 degrees. Assume the joint diameter of 88.25-inch and cylinder diameter of 87.75-inch. Assume pipe working pressure of 140 psi and test pressure of 170 psi., pipe weight of 850 lb/lin-ft and allowable axial stress of 12,500 psi at working pressure and 16,000 psi at test pressure.

The weight of water in pipe = Area of pipe * Unit weight of water = 38.48 * 62.4 = 2401.4 lb/lin-ft
Earth load We on pipe = 10,692 lb/lin-ft (obtained using procedure shown in section 5.3)
Cross sectional area of joint = 6116.7 sq. in
Hence, F = PA (1- COS 45) = 170 * 6116.7 (1-0.707) = 304,673 at test pressure
= 140 * 6116.7 (1-0.707) = 250,907 at working pressure
Frictional resistance is f (2We + Wp + Ww) = 0.3(2*10,692 + 850 + 2401.4) = 7390.6 lb/lin-ft

Hence, the pipe length to be tied, L = 304,673 / 7390.6 = 41.2 linear feet

Longitudinal reinforcement needed at elbow = F / fs

= 250,907 / 12500 = 20.07 sq. in (at working pressure)

= 304,673 / 16000 = 19.04 sq. in (at test pressure)

5.6 Influence Of Tunnel On Adjacent Structures

A properly designed and controlled tunneling operation can 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. Tunnels constructed below roadway, railroad and buried utilities may lead to some future settlement due to loosening of the sub grade or bedding condition. Depression in the pavement and associated distress in paving, breaks in existing utility lines can occur as a result of settlement due to loosening of the sub grade or bedding condition. Large ground loss can result from uncontrolled flowing ground. Such condition may occur wherever water-bearing sands or silts are encountered along the tunnel alignment. Water was encountered below depth of 14-feet near the tunnel alignment at Little White Oak Bayou and below the depth of about 11-feet near the tunnel alignment at Interstate 45.

The zone of influence of the tunnel roughly extends to a distance equal to the invert depth on each side of the centerline of the tunnel alignment. The amount of settlement due to tunneling can only be estimated. It is difficult to determine the percentages of ground movement. We anticipate based on our experience in the area that if good construction practices and control are exercised, the amount of ground settlement will be limited. Elevation of the roadway, sidewalk and other important structures along the tunnel alignment should be taken prior to, during and after construction to evaluate the amount of settlement due to tunneling and the effectiveness of the tunneling technique adopted. Existing damages to the surrounding structures should be documented prior to starting of the tunneling operations.

Our review of the profile plans furnished by Thompson Professional Group, Inc., and out discussion with Mr. Ryan Simper indicates that the tunneling alignment may be planned below the bridge pier bearing depth. We understand that one of the option currently under consideration includes tunneling at a distance of about six- to seven-feet from the bridge piers i.e. the horizontal distance between the edge of the bridge pier bottom and the closest point of the tunnel will be about six- to seven-feet. Additionally, the top of the tunnel will be roughly at an elevation three-feet lower than the bridge pier bottom. Please inform us immediately if bridge pier and tunnel spacing is any closer than our understanding as stated above. The load transferred in end bearing by the drilled pier will be to a soil zone, the outer limits of which may be roughly taken as a line drawn at a slope of 2 vertical to 1 horizontal from the drilled pier outer edge. The soil outside this zone will not contribute significantly to the end bearing capacity of the drilled pier. Based on the tunnel alignment location being considered as mentioned above, it appears that the tunnel will be in a zone which will just be outside the soil zone influencing the bridge pier end bearing capacity. Hence, no significant influence of the tunneling is anticipated on the end bearing capacity of the existing bridge piers. However, it should be noted that if ground loss or conditions such as flowing sands occurs during tunneling, then this may affect the soil zone of influence of the bridge pier and result in significant settlement and subsequent distress to the pier. Hence, if tunneling below the bridge pier bottom elevation is required, extreme care should be taken to prevent conditions such as flowing sands which may result in ground loss and subsequent settlement. Also, as the tunneling is being considered at a distance of more than five-feet from the bridge pier we do not anticipate any significant loss in the side resistance capacity of the existing piers. However, as in the case for the end bearing capacity, care should be taken to prevent any ground loss during tunneling.

5.7 Lateral Earth Pressure Diagram

For the trench supporting and braced system, the lateral pressures exerted by surrounding soils are presented in Figure 6A. For any trench supporting system close to the bridge piers at the Interstate 45 location, the lateral pressures may be taken as given in Figure 6B. In case that a trench shield is used, the trench shield may be designed for a jlateral earth pressure equivalent to a fluid pressure of 96 PCF for cohesive soils below the watertable and 65 PCF for the cohesive soils above the watertable. In non-cohesive soils below water table, the trench shield should be designed for a lateral earth pressure equivalent to a fluid pressure of 85 PCF and about 43 PCF above the water table. At the Interstate 45 bridge location, the lateral pressures exerted by surrounding soils may be taken as 130 pcf in cohesive soils for both above and below the water table. In sandy soils, these pressures may be taken as 96 PCF below the water table and 65 PCF above the water table. In general, a surcharge magnitude of q psf will result in lateral earth pressure of 0.5q in clayey soils and 0.33 q in sandy soils. At the Interstate 45 bridge crossing location, the surcharge magnitude of q psf may be taken as resulting in lateral earth pressure of q in clayey soils and 0.5q in sandy soils. For the calculations of the above given earth pressures, unit weight of 130 pcf was assumed along with a coefficient of active earth pressure Ka of 0.5 in cohesive soils and 0.33 in sandy soils. At the Interstate 45 bridge crossing location, the value of coefficient of earth pressure K was assumed as 1 in cohesive soils and 0.5 in sandy soils. To demonstrate how the above earth pressure were obtained, here is the example calculations for the cohesive soils below the water table.

Unit weight of cohesive soils = 130 pcf, Unit weight of water = 62.4 pcf

Submerged (buoyant) weight of cohesive soils = 130 – 62.4 = 67.6 pcf

Now, total earth pressure in pcf = pressure from soil + hydrostatic pressure

= Ka * Submerged weight of soil + unit wt. of water

= 0.5 (67.6) + 62.4 = 96.2 pcf say 96 pcf

Due to the presence of the roadway adjacent to the likely excavation areas at the project site, the effect of vehicular traffic may be considered while designing the lateral supporting systems. Boussinesq's equation should be used for calculating the loads on the retaining systems due to the vehicular traffic. We recommend that a HS20 vehicle loading be considered adjacent to the pit for design purposes. An impact factor of 1.5 should be used in the design. Surcharge loading due to construction machinery should be considered as applicable. All loads acting within a distance of one half the excavation depth on all sides should be considered for designing retaining systems.

Stockpiling of excavated material may not be allowed near the excavation. Generally, a distance of one half the excavation depth on both sides of the trench should be kept clear of any excavated material. If this is not possible due to space limitations then the retaining system design should take into account the surcharge loads.

Care is urged during excavations since conditions such as sloughing or caving of the excavation trench or excavation slope may result in movement of the surrounding soils resulting in possible settlement of the surrounding structures or features. Additionally, at Interstate 45 crossing location, one of the alternative for the water line under consideration is to install it by using open trench method. We understand that the water line will be about six- to seven-feet from the bridge pier. Since, the water line will be farther than five-feet, we do not anticipate any significant effect on the side resistance capacity of the drilled pier. However, it should be noted that the side resistance capacity of the bridge pier may be affected if conditions such as caving or sloughing of the excavation side occurs during trench excavations. Hence, we recommend that effective trench retention measures be planned and monitored. The retention should be as rigid as possible and designed for the higher earth pressures as given in this report at the Interstate 45 location.

Shown below is our example calculation for the bracing pressures. Say for tunnel access shaft trench excavations for tunnel near boring GB-5, soil consist of stiff to very stiff sandy clays to the depth of about 23-feet underlained by clayey silt soils to a depth of about 33-feet underlained by cohesive clays till the maximum depth of the boring at 55-feet. Groundwater was encountered during drilling below 24-feet. Using Figure 6, Pa equals 0.4 YH in cohesive soils and 0.25 YH in sandy soils, take H equal to 48-feet and Y of 130 pcf above water table and submerged Y of 67.6 pcf below water table. Since the groundwater is at 24-feet, the composite Y may be obtained as (24* 130 + 24*67.6)/48 =98.8 pcf. Hence using Figure 6, the pressure Pa at top of trench is zero, at depth of 0.25 H (12-ft), Pa is 0.4*98.8*48 = 1896.96 psf. At depth of 36-feet, Pa is 1896.96 psf, at depth of 48-ft Pa is = 0. Hydrostatic pressure of 62.4 times water depth should be added below 5.5-feet. A surcharge pressure of 0.5*500 should be added from top.

5.8 Quality Control

Associated Testing Laboratories, Inc. (ATL) recommends implementation of a comprehensive quality control program under the supervision of a Professional Engineer due to the fact that a considerable amount of excavation and back filling may be required in the proposed project area. Structural integrity and stability is particularly dependent on quality foundation installation, bedding and subgrade preparations.

An independent testing laboratory should be assigned to test and inspect construction materials during the construction phase.

To ensure that excavation will remain stable, to provide sufficient headroom for working, to provide worker's safety and to protect adjacent structures, the excavations will have to be provided with sufficient side slopes or shored in accordance with OSHA "Trench Safety Systems" (29 CFR Part 1926), as published in the Federal Register, Vol. 52, No.72, Section 1926-650 through 1926-653. Excavation of the trenches and access pits should be carried out under the supervision of an experienced construction supervisor and necessary shoring and/or bracing of the trenches should be properly installed. In temporary braced or shored excavations and in access pits where the sheeting terminates at the base of the trench, lateral earth pressure, surcharge, and seepage pressure caused by a differential hydrostatic head moving upward to the bottom of the trench can cause trench bottom instability. Therefore, it is recommended that, if the bottom stability evaluation in cohesive soils yields a factor of safety less than 1.5, the sheeting should be extended below the base of cut. Dewatering is recommended in non-cohesive soils.

Before filling operations take place, representative samples of the proposed fill material should be tested by an independent laboratory to determine the compaction and classification characteristics. Additional fill material (used for backfilling) should meet the standard SWTP specifications. Fill materials should be placed and compacted to the requirements as specified in standard SWTP specification.

5.9 Monitoring

Despite the thoroughness of this geotechnical exploration, there is always the possibility that actual subsurface conditions may differ from the predicted conditions because conditions between soil borings can be different from those at specific boring locations.

Any excessive ground movements like settlement and lateral movement should be monitored and controlled. This can be done by performing a preconstruction survey including photography and documentation of existing conditions like elevations, cracks, etc., and by installing ground movement monitoring devices such as inclinometers, crack monitors, and establishing elevation monitor stations along the waterline alignment to monitor the ground movement after commencement of the excavation.

Associated Testing Laboratory, Inc.(ATL) recommends a regular inspection and overall project monitoring by a geotechnical engineer during the construction phase. The purpose of inspection is to provide sound engineering and judgement alternatives during construction, if unanticipated conditions occur.

6.0 LIMITATIONS

The recommendations contained in this report are based on data gained from test borings at the locations shown in figure 2, a reasonable volume of laboratory tests, and professional interpretation and evaluation of such data, from the project information furnished. Should it become apparent during construction that soil conditions differ significantly from those discussed in this report, this office should be notified immediately so that an evaluation, and any necessary adjustments can be made. Any analysis of slope stability, bulkhead or other buildings or features at the site, not within the scope of this investigation, ATL is not responsible for any problems caused by these features.

7.0 AUTHORIZATION AND CREDITS

This project of Accelerated Surface Water Transmission Program, Contract 6C-1, GFS No. S0900-64-3, and Water File No. WA 10637 was authorized by Thompson Professional Group, Inc., with the acceptance of the Associated Testing Laboratories, Inc., proposal no. GP00-1201 dated December 11, 2000. Project details were provided to ATL by Mr. David Kubala, P.E., and Mr. Ryan Simpson, P.E., of Thompson Professional Group, Inc. The field investigation and laboratory investigations were performed by Associated Testing Laboratories, Inc., in accordance with the ATL proposal referenced above. The soil characterization, analyses, recommendations and the formal written report were provided by Associated Testing Laboratories, Inc. Associated Testing Laboratories, Inc., staff which participated in the project were Mr. Jasbir Singh, P.E., Mr. Jaywant Vaghela, P.E., Mr. Jitendra Shah, E.I.T., and Mr. Sam Mohammed, BSCE.

8.0 REFERENCES

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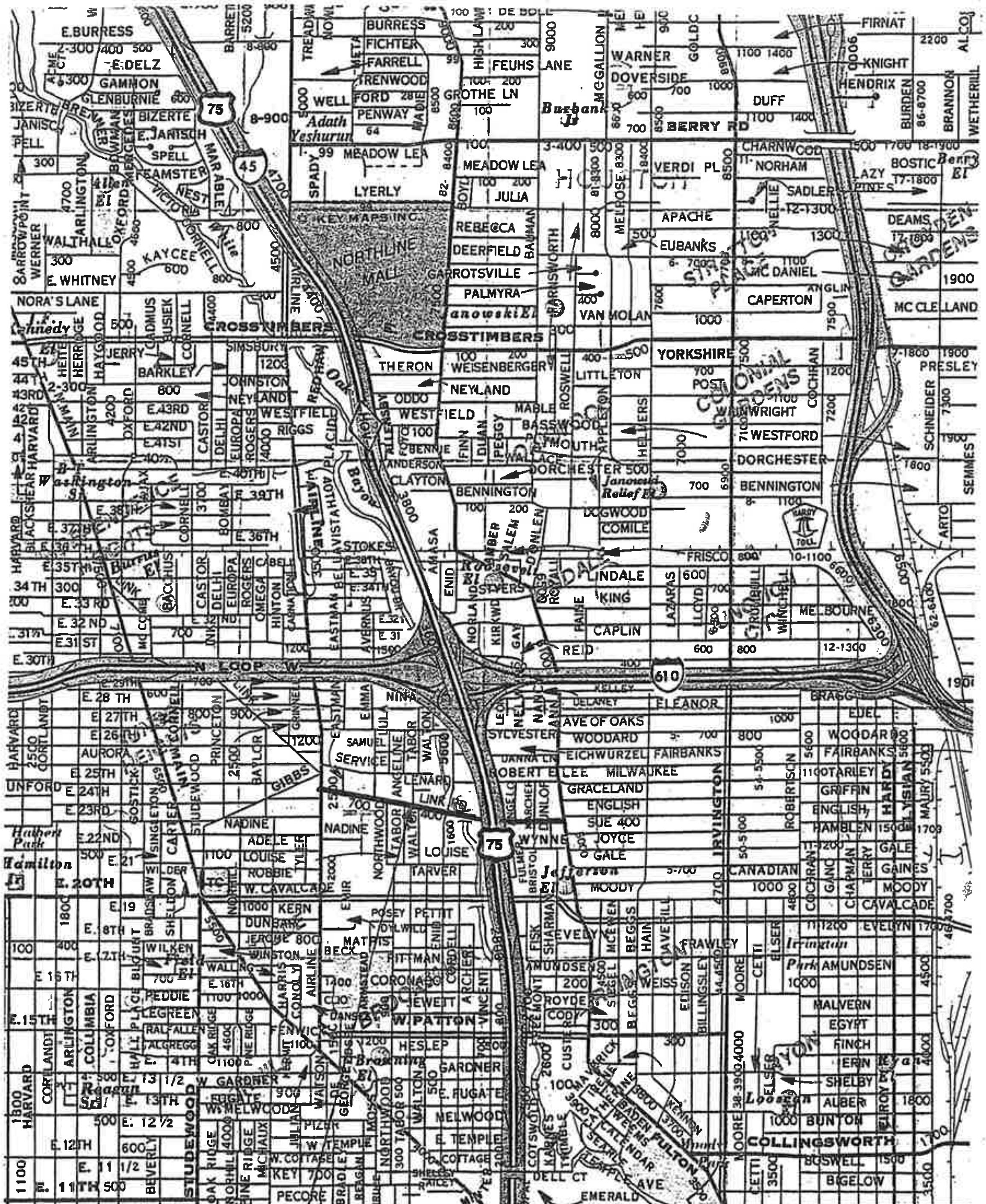
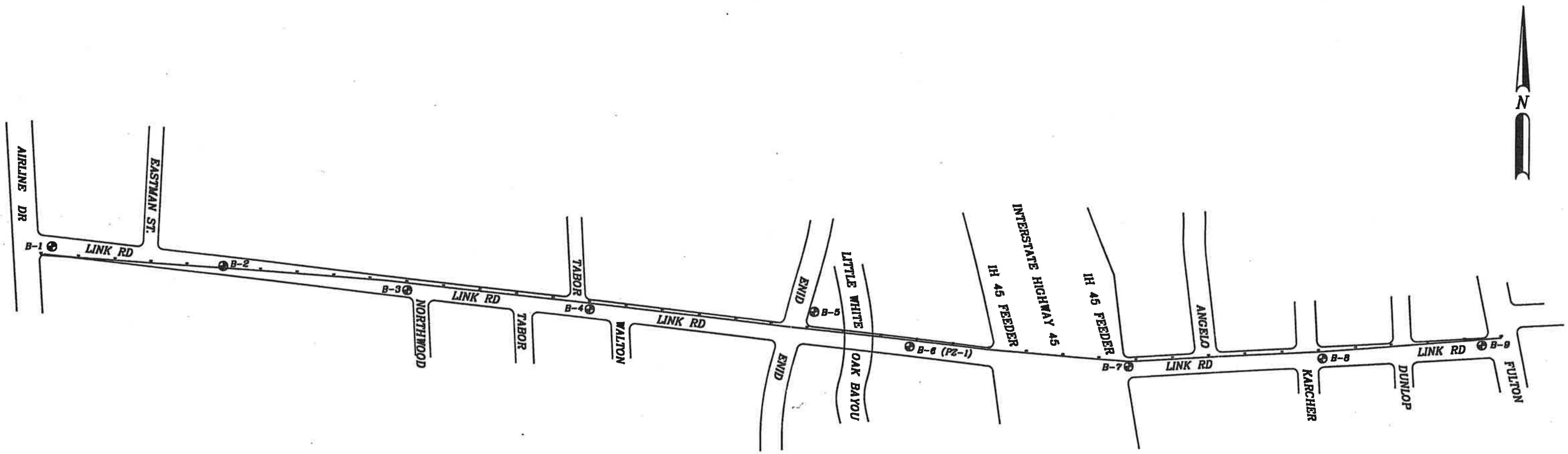
FIGURES



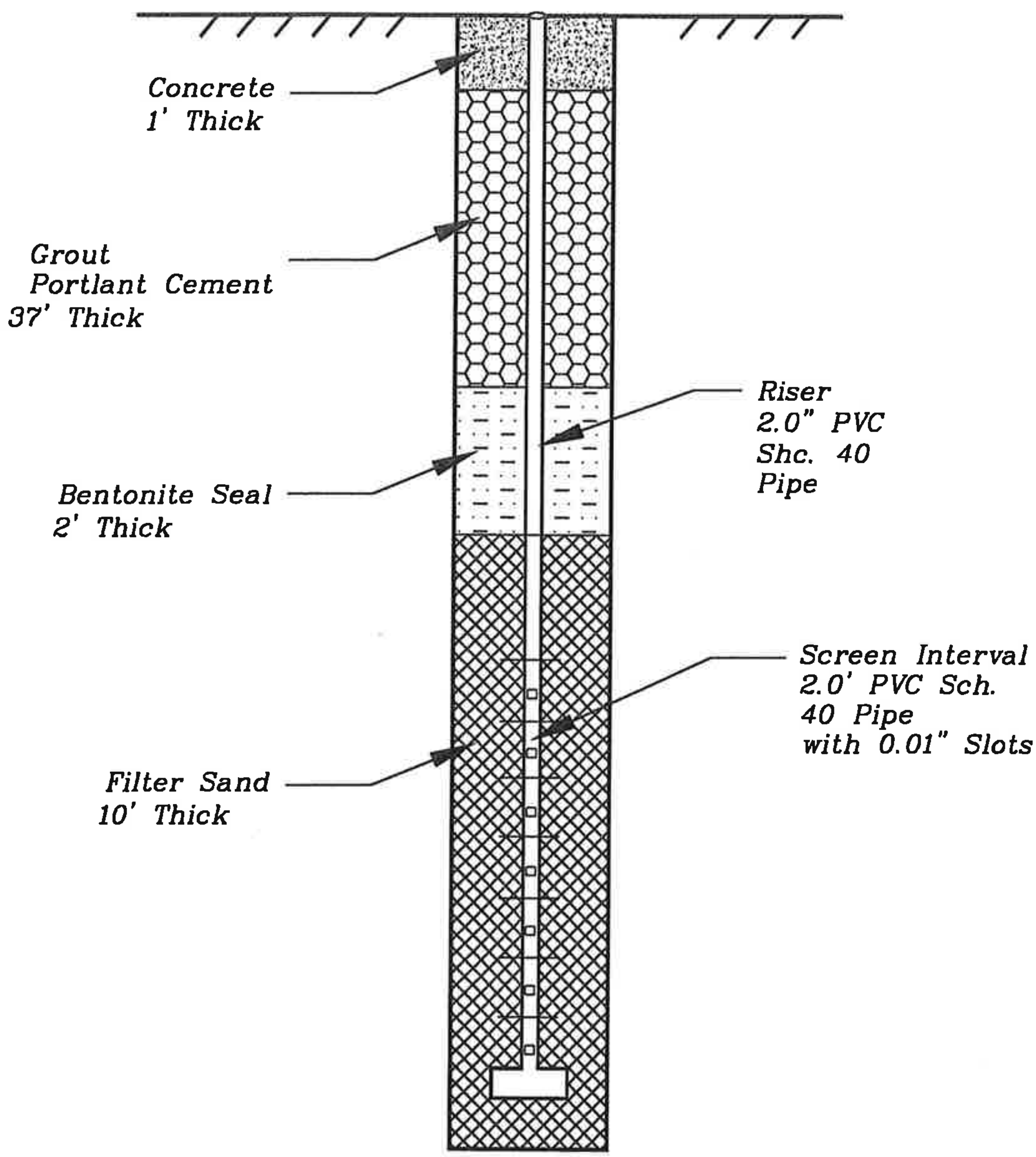
SITE LOCATION

N

SITE VICINITY MAP	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748	
ACCELERATED S.W.T.P. CONTRACT 6C-1 ALONG LINK ROAD HOUSTON, TEXAS	S.W.T.P. CONTRACT 6C-1	FILE NO. WA 10637
	PROJECT NO. G00-801	FIGURE. 1

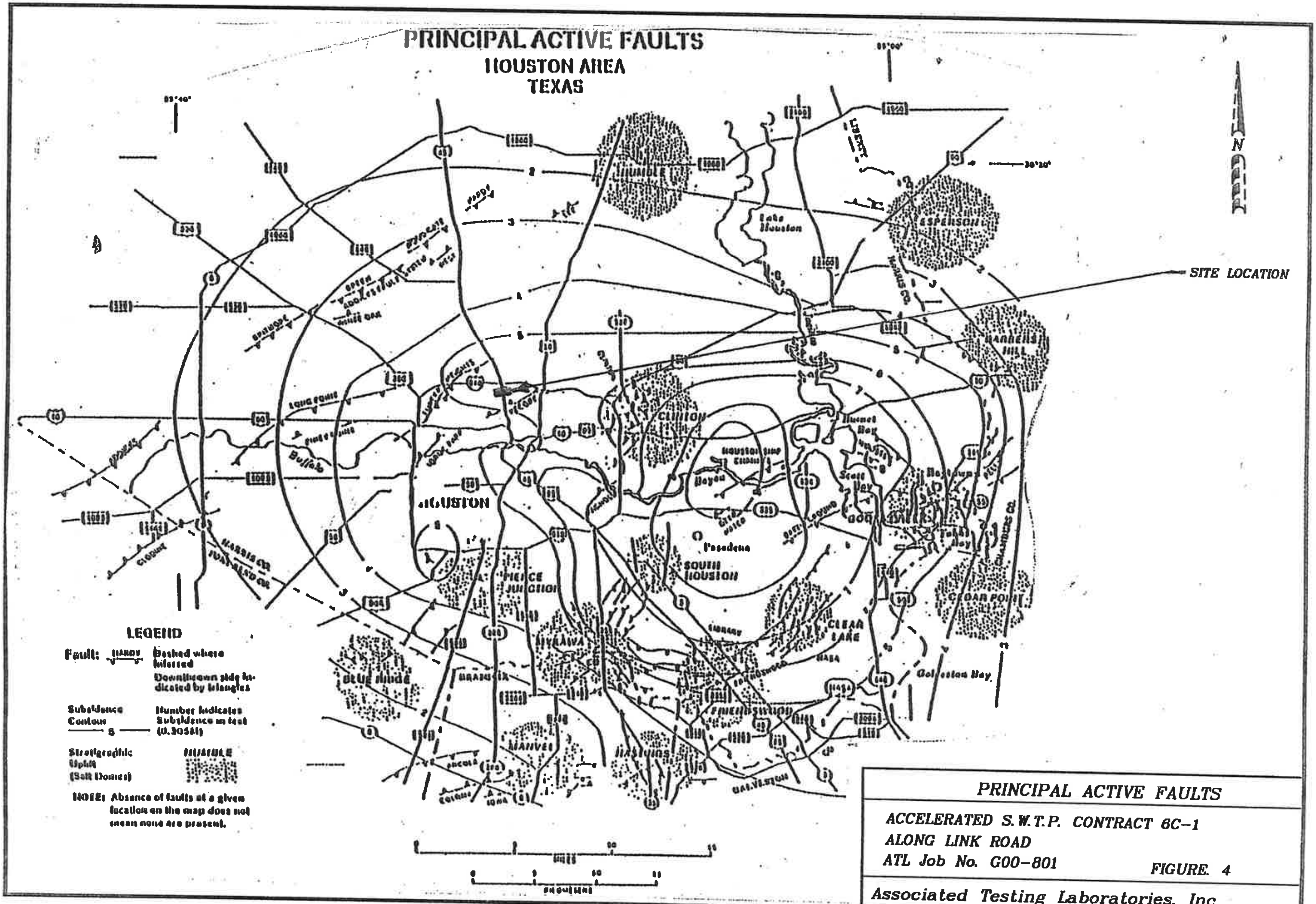


SITE PLAN	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748	
ASWTP (CONTRACT NO. 6C-1) ALONG LINK ROAD FROM AIRLINE TO FULTON HOUSTON, TEXAS	SCALE: N.T.S.	DRWN BY: SAM
	PROJECT: G00-801	FIGURE. 2



PIEZOMETER AT GB-6

FIG.3 G.W. MONITORING WELL
ACCELERATED S.W.T.P. CONTRACT 6C-1 ALONG LINK ROAD ATL Job No. G00-801 FIGURE. 3
Associated Testing Laboratories, Inc.



PRINCIPAL ACTIVE FAULTS

ACCELERATED S.W.T.P. CONTRACT 6C-1
ALONG LINK ROAD
ATL Job No. G00-801

FIGURE. 4

Associated Testing Laboratories, Inc.

ASSOCIATED TESTING LAB, INC.

Project Name: Accelerated Surface Water Transmission Program along Link Rd. G00-801

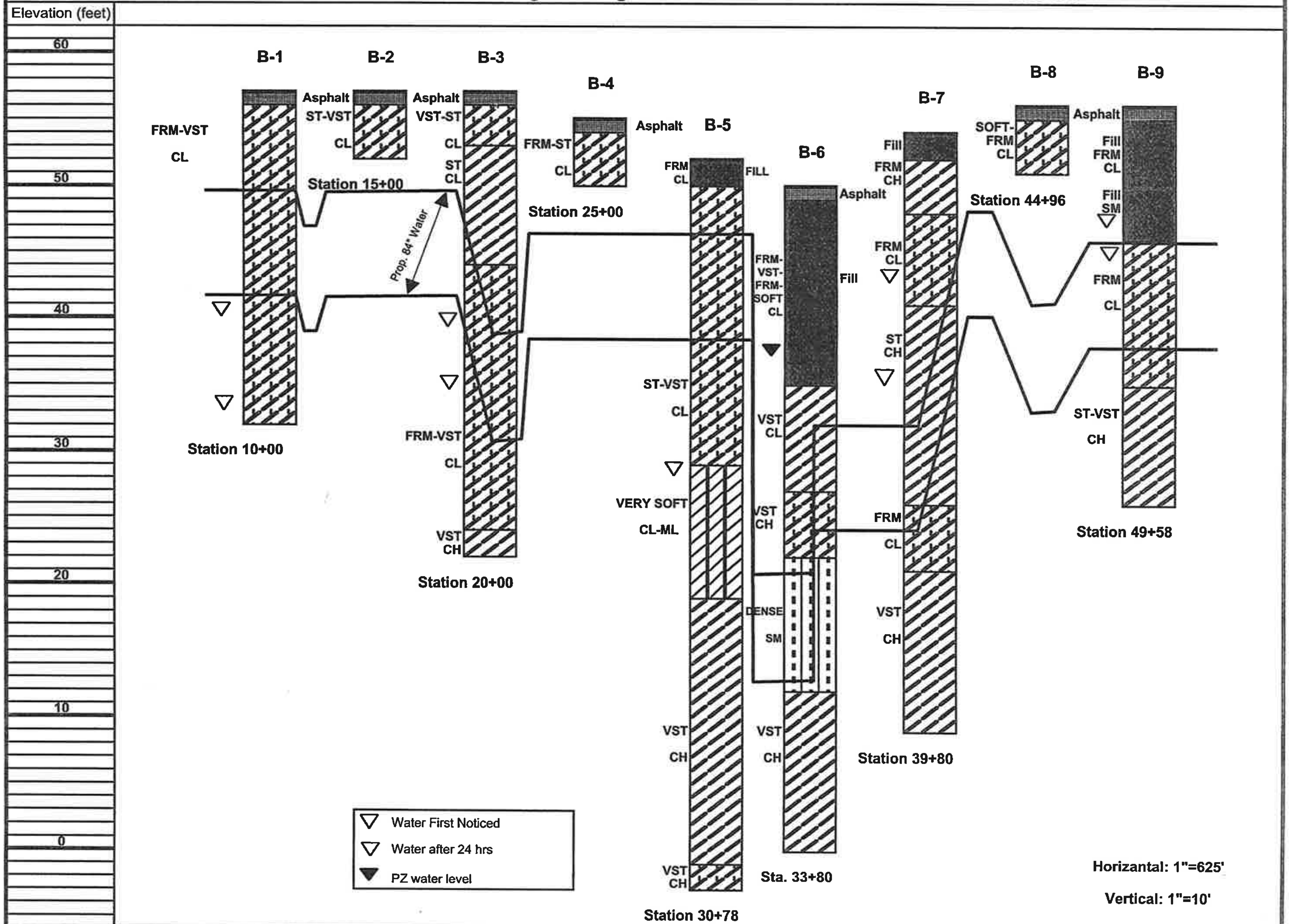
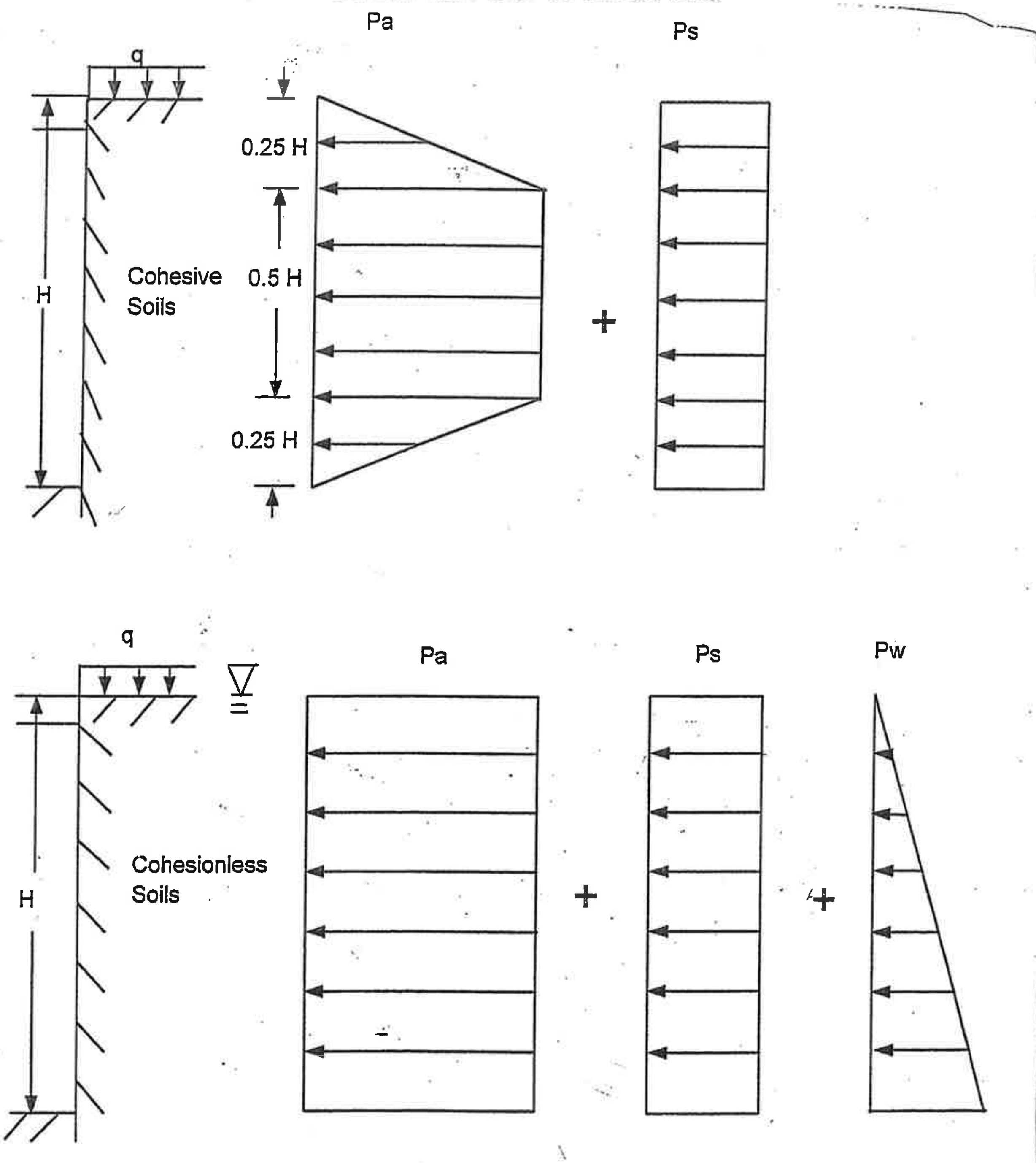


Figure 5



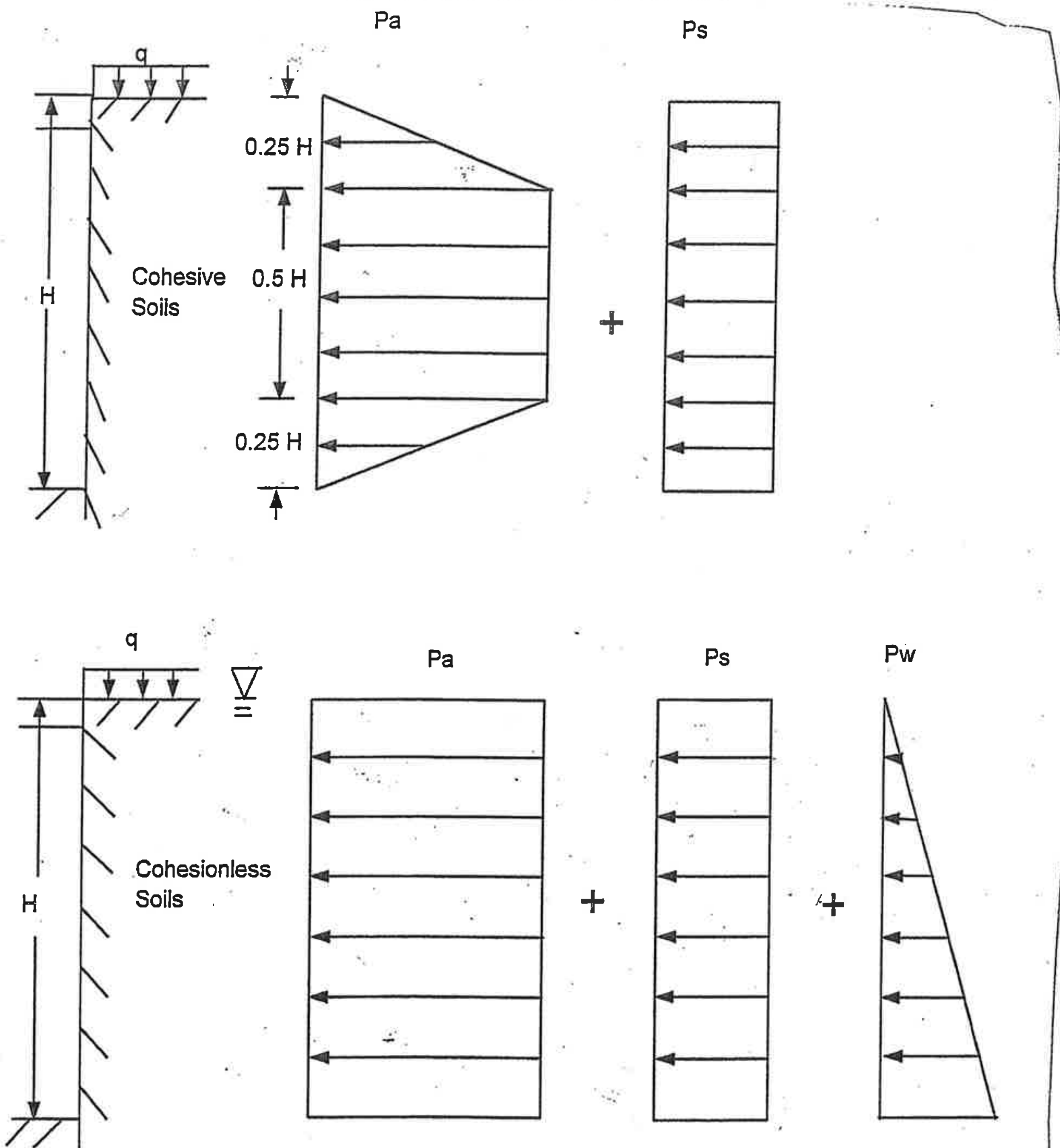
EARTH PRESSURE DIAGRAM

P = Pa + Ps + Pw

where P = Total lateral pressure (psf)
Pa = Active earth pressure (psf) = 0.4γH for Clays
= 0.25γH for cohesionless Sands
Ps = Lateral pressure due to surcharge load (psf) = 0.5q
Pw = Hydrostatic pressure (psf) = 62.4 * water depth
H = Depth of braced excavation (ft)

q = Surcharge load (psf) usually taken as 500 psf
γ = Average effective density of soils (pcf) = 120

Fig. 6A



EARTH PRESSURE DIAGRAM

P = Pa + Ps + Pw

where P = Total lateral pressure (psf)

Pa = Active earth pressure (psf) = 0.6yH for Clays

= 0.4yH for cohesionless Sands

Ps = Lateral pressure due to surcharge load (psf) = 0.7q

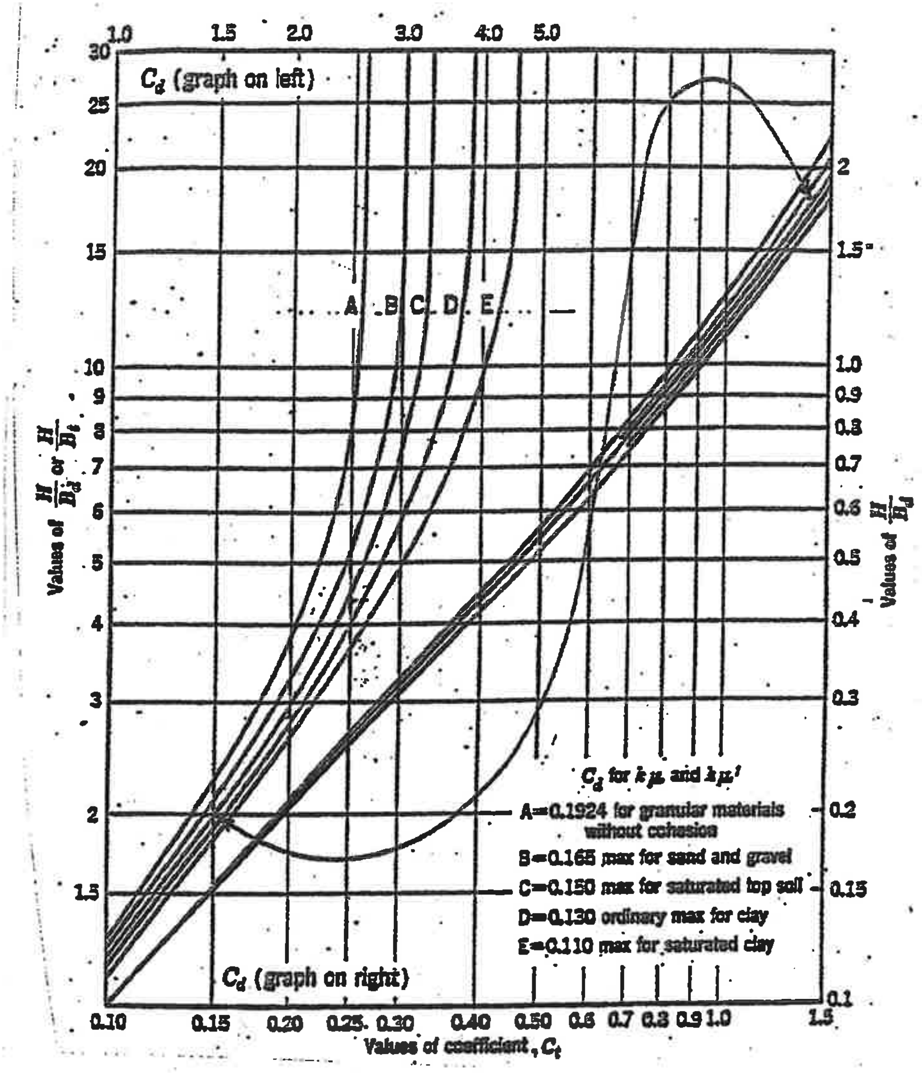
Pw = Hydrostatic pressure (psf) = 62.4 * water depth

H = Depth of braced excavation (ft)

q = Surcharge load (psf) usually taken as 500 psf

y = Average effective density of soils (pcf) = 120

Fig. 6B



LOAD COEFFICIENT CHART	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748	
ACCELERATED S.W.T.P. CONTRACT 6C-1 ALONG LINK ROAD HOUSTON, TEXAS	S.W.T.P. CONTRACT 6C-1	FILE NO. WA 10637
	PROJECT NO. G00-801	FIGURE. 7

$P1=\{[(H+D/2)*Y-(H-Dw)Yw$

FOR $Dw<H+D/2$

FOR $Dw>\_+D/2$

$P1=\{[(H+D/2)*Y]/1000\}+qs$

$P2=[(H*Y)/1000]+qs$

$P3=\{[(H+D)*Y]/1000\}+qs$

WHERE: P1, P2, P3=TUNNEL LINER LOAD, KSF

D=TUNNEL DIAMETER, FT

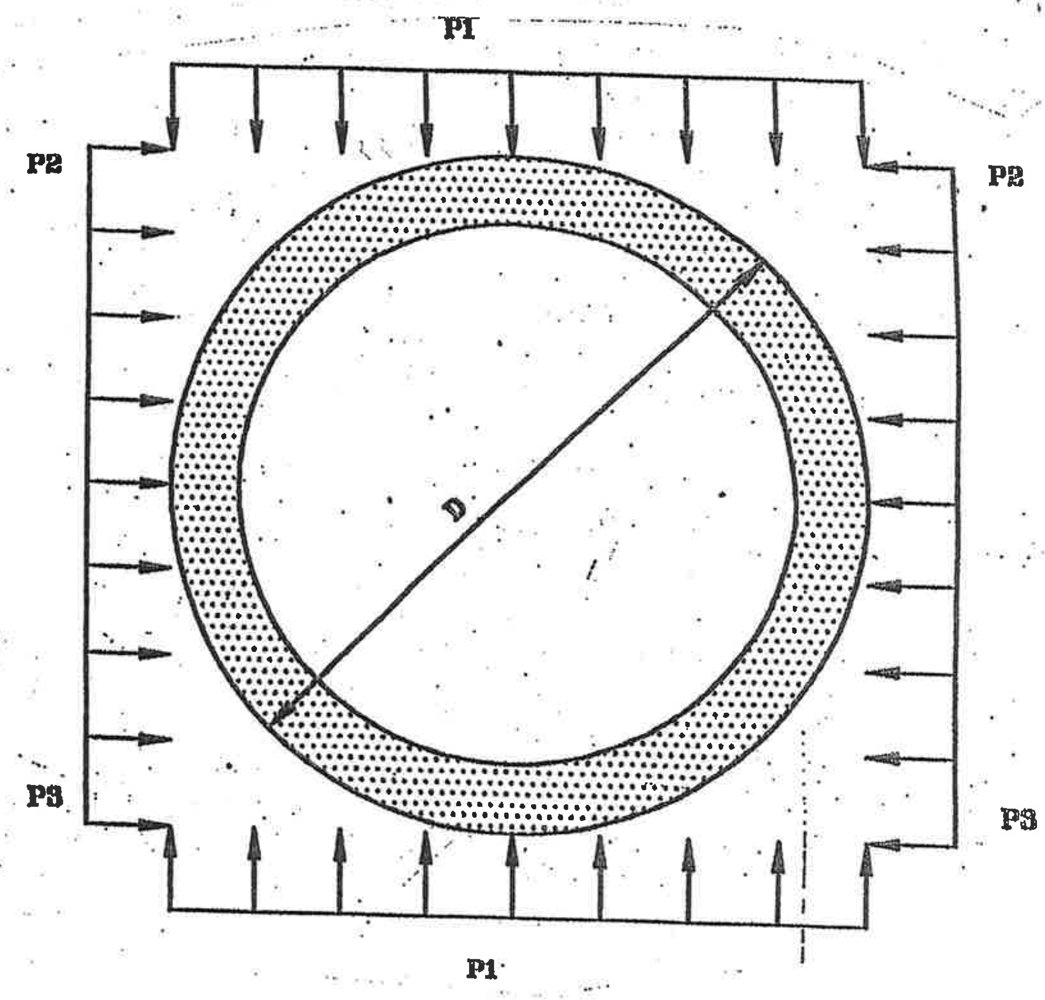
H=DEPTH TO TOP OF TUNNEL, FT

Dw=DEPTH TO GROUND WATER LEVEL, FT

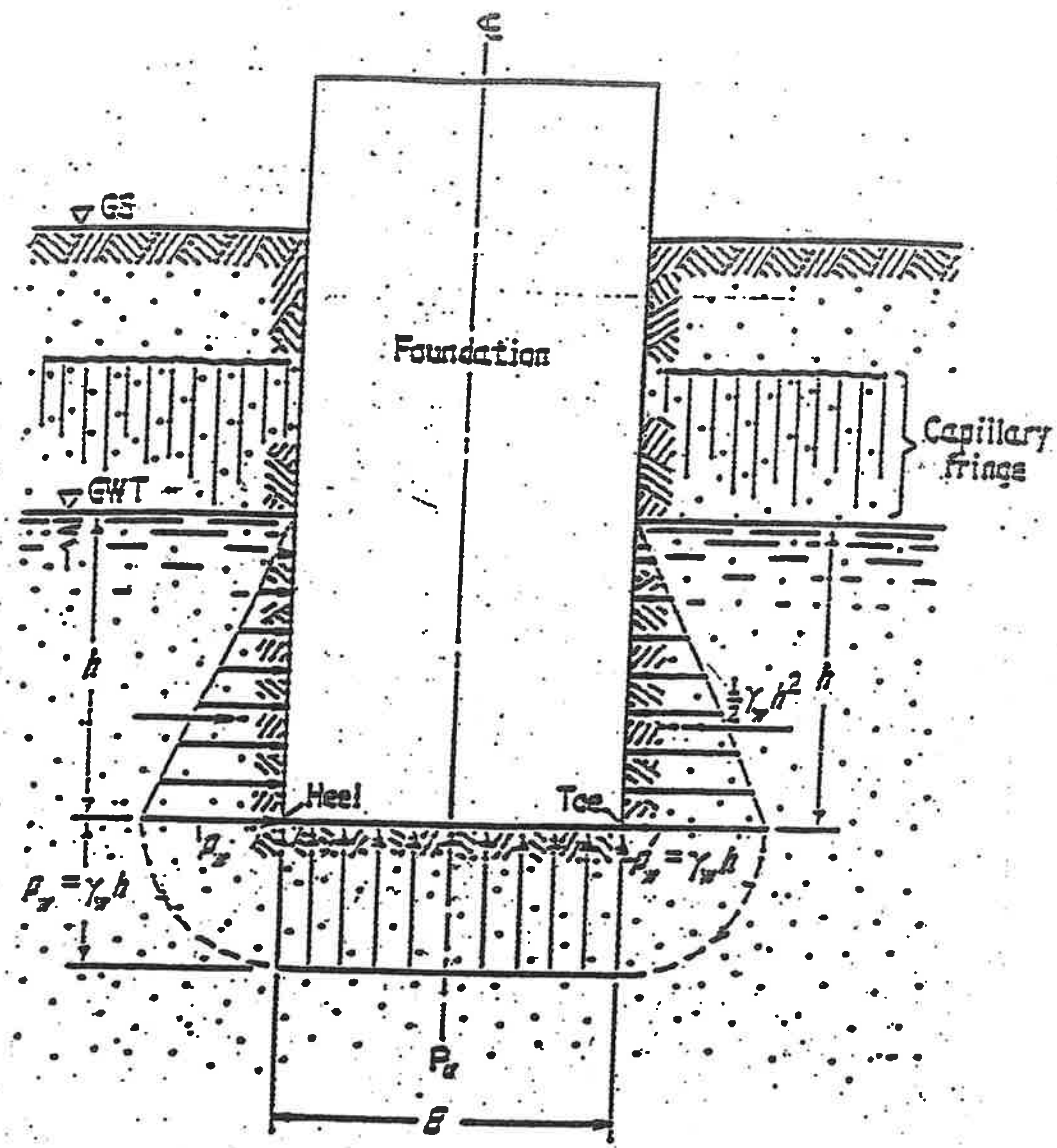
Y=WET UNIT WEIGHT OF SOIL VALUE 130 PCF);

Yw=UNIT WEIGHT OF WATER, 62.4 PCF

qs=SURCHARGE LOAD, KSF.
(INCLUDE FOR LIVE LOADS)



TUNNEL LINER LOAD
ACCELERATED S.W.T.P. CONTRACT 6C-1 ALONG LINK ROAD ATL Job No. G00-801 FIGURE. 8
Associated Testing Laboratories, Inc.



UPLIFT PRESSURE DIAGRAM FOR GROUND WATER AT REST

$P_w = \gamma_w * h$
$P_t = P_w * B$

P_w = Uplift Pressure (psf)
γ_w = Unit Weight of water (62.4 pcf)
h = Depth of water from ground water table to the bottom of foundation (ft)
P_t = Uplift force per unit length of foundation (lbs/ft)
B = Width of the foundation (ft)

UPLIFT PRESSURE DIAGRAM	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748	
ACCELERATED S.W.T.P. CONTRACT 6C-1 ALONG LINK ROAD HOUSTON, TEXAS	S.W.T.P. CONTRACT 6C-1	FILE NO. WA 10637
	PROJECT NO. G00-801	FIGURE. 9

FIGURE 10

SUMMARY OF CBR AND PROCTOR TEST RESULTS

PROPOSED WATER LINE REPLACEMENT

ALONG LINK ROAD

GFS NO. S-0900-64-2, FILE NO. WA10637, CONTRACT 6C-1

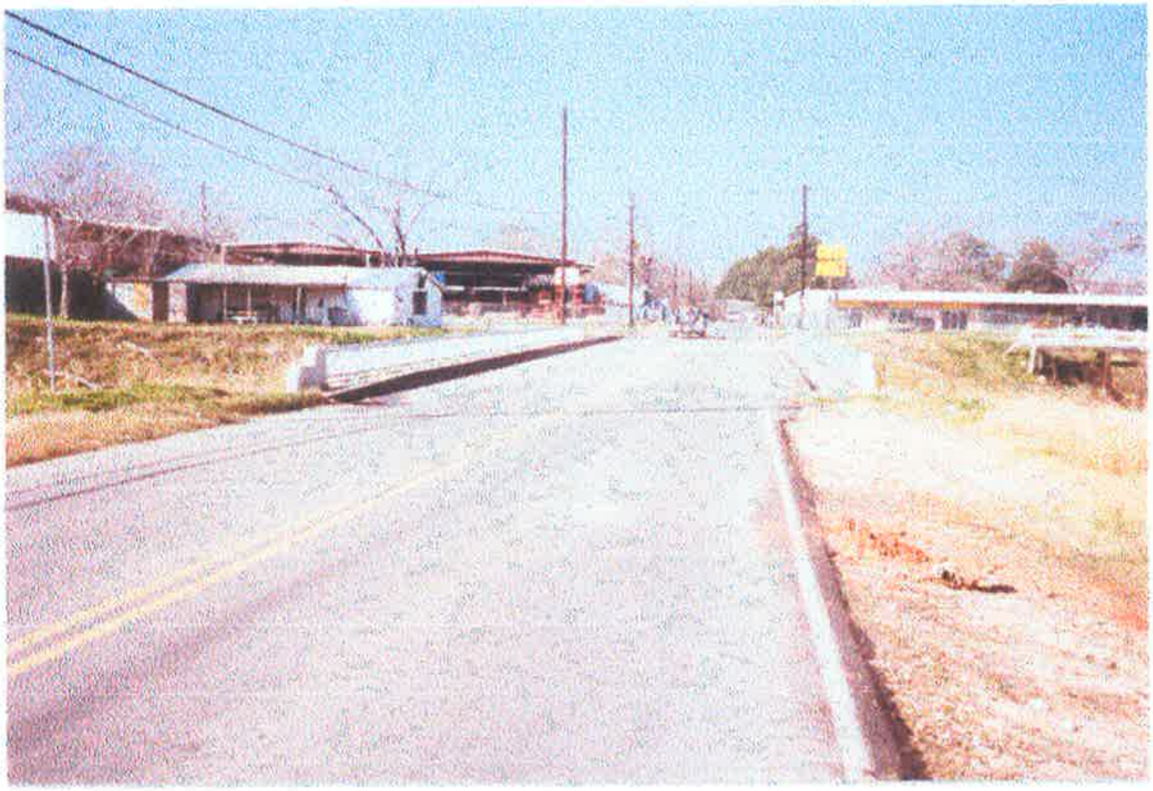
ASSOCIATED TESTING LABORATORIES, INC. JOB NUMBER G00-801

Test Number	Sample Location	CBR Value	Max Dry Density, pcf	Optimum Moisture %
1	Near Boring B-1	7	125	11
2	Near Boring B-9	4	110.1	14.8

APPENDIX 1

PHOTOGRAPHS OF THE PROJECT SITE

PHOTOGRAPHS OF THE PROJECT SITE



PHOTOGRAPHS OF THE PROJECT SITE

APPENDIX 2

DEFINITION OF TERMS AND KEY TO SYMBOLS

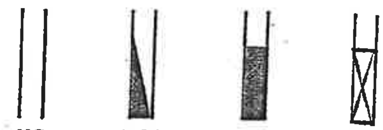
ASSOCIATED TESTING LABORATORIES, INC.

KEY TO LOG TERMS AND SYMBOLS

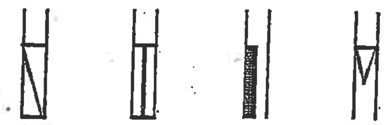
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D 2487

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS LESS THAN 50% PASSING NO. 200 SIEVE	GRAVEL & GRAVELY SOILS LESS THAN 50% PASSING NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		W/ APPRECIATA-BLE FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS,GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN 50% PASSING NO. 4 SIEVE	CLEAN SANDS LITTLE FINES	SW	WELL GRADED SAND, GRAVELLY SAND (LITTLE FINES)
			SP	POORLY GRADED SANDS, GRAVELLY SAND (L.FINES)
		SANDS WITH APPREA. FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS,SAND-CLAY MIXTURES
FINE GRAINED SOILS LESS THAN 50% PASSING NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS,ROCK FLOUR SILTY OR CLAYEY FINE SANDS OR CLAYEY SILT W/PI
			CL	INORGANIC CLAY OF LOW TO MEDIUM PI LEAN CLAY GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PI
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY FAT CLAYS
			OH	ORGANIC CLAYS OF MED TO HIGH PI, ORGANIC SILT
HIGHLY ORGANIC SOIL			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS
UNCLASSIFIED FILL MATERIALS				ARTIFICIALLY DEPOSITED AND OTHER UNCLASSIFIED SOILS FILL MATERIALS

SAMPLER TYPE



NO SAMPLE | AUGER SAMPLE | SHELBY TUBE | SPLIT SPOON



NO RECOVERY | ROCK CORE | 2" SHELBY TUBE | TXDOT CONE

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	UNCONFINED COMP. STRENGTH IN TSF
VERY SOFT	less than 0.25
SOFT	0.25 TO 0.5
FIRM	0.5 TO 1.0
STIFF	1.0 TO 2.0
VERY STIFF	2.0 TO 4.0
HARD	greater than 4.00

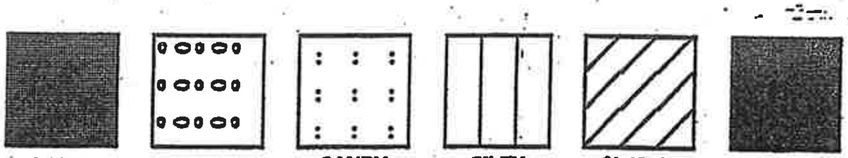
SOIL TYPE

ROCK | GRAVEL | SAND | SILT | CLAY | PEAT

RELATIVE DENSITY - GRANULAR SOILS

CONSISTENCY	N-VALUE (BLOWS PER FT)
VERY LOOSE	<4
LOOSE	5-10
MEDIUM DENSE	11-30
DENSE	31-50
VERY DENSE	> 50 OR 50+

MODIFIERS



STONE | GRAVELY | SANDY | SILTY | CLAYEY | FILL

CLASSIFICATION OF GRANULAR SOILS

U.S. STANDARD SIEVE SIZE(S)

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

BOUL- -DERS	COBBLES	GRAVEL		SAND			SILT OR CLAY	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 | 76.2 | 19.1 | 4.76 | 2.0 | 0.42 | 0.074 | 0.002

GRAIN SIZE IN MM

APPENDIX 3

BORING LOGS

Project Name: Accelerated Surface Water Transmission Program along Link Rd.
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1

BORING LOG

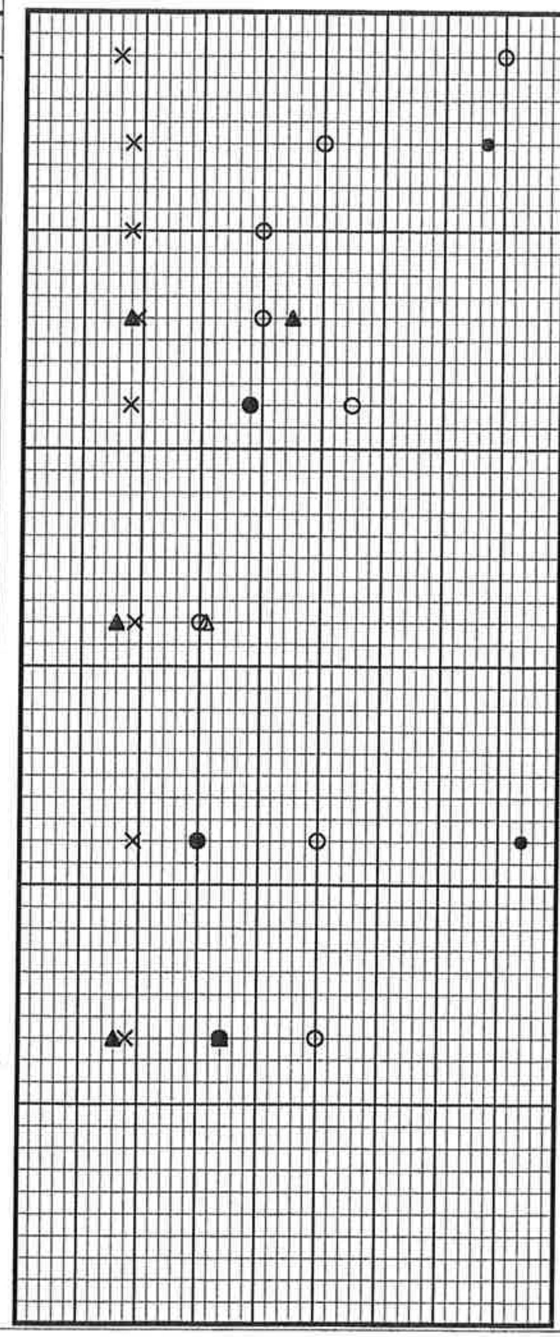
PROJECT NUMBER: G00-801 | BORING NUMBER: GB-1

GEOTECHNICAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC. | DESIGN CONSULTANT: Thomson Professional Group, Inc.

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					5" Asphalt and 4" Lime stabilized base
2.0		1			Very stiff dark gray **Sandy Clay (CL)**
4.0		2			.. stiff below 2 feet
6.0		3			.. gray and tan below 4 feet
8.0		4			.. light gray and tan with ferrous nodules below 6 feet
10.0		5			
15.0		6			.. firm with sand seams below 13 feet
20.0		7			.. stiff tan and light gray below 18 feet
25.0		8			
					Boring was terminated at 25 feet
30.0					

TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
△ LL ▲ PL
× MOIST.(%) ● #200 (%)



Water First Noticed: 17'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 10+00 OFFSET
Depth to Water at 24 hrs: 24.3'			GROUP LEVEL(MSL): 57'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 25'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-2
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

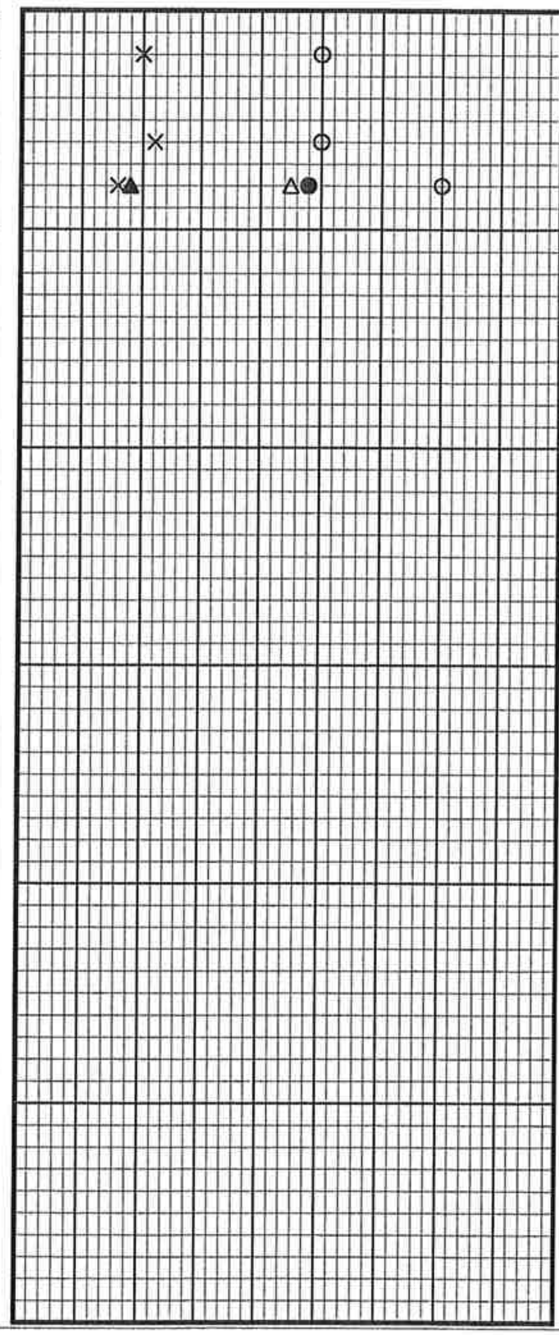
DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					3" Asphalt and 2" Lime stabilized base
		1			Stiff dark gray **Sandy Clay** (CL)
2.0		2			.. light gray and tan below 2 feet
4.0		3			.. very stiff with calcareous nodules below 4 feet
					Boring was terminated at 5 feet
6.0					
8.0					
10.0					
15.0					
20.0					
25.0					
30.0					

TEST RESULTS

O PENETROMETER TEST
● UNCONFINED COMP.
Δ LL
▲ PL
× MOIST.(%)
● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: None	DRILLED BY: H G.	STARTED: 01/15/01	STATION 15+00 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 57.3'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 5'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

ASSOCIATED TESTING LABORATORIES, INC.

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	**BORING LOG**
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801 / BORING NUMBER: GB-3
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.

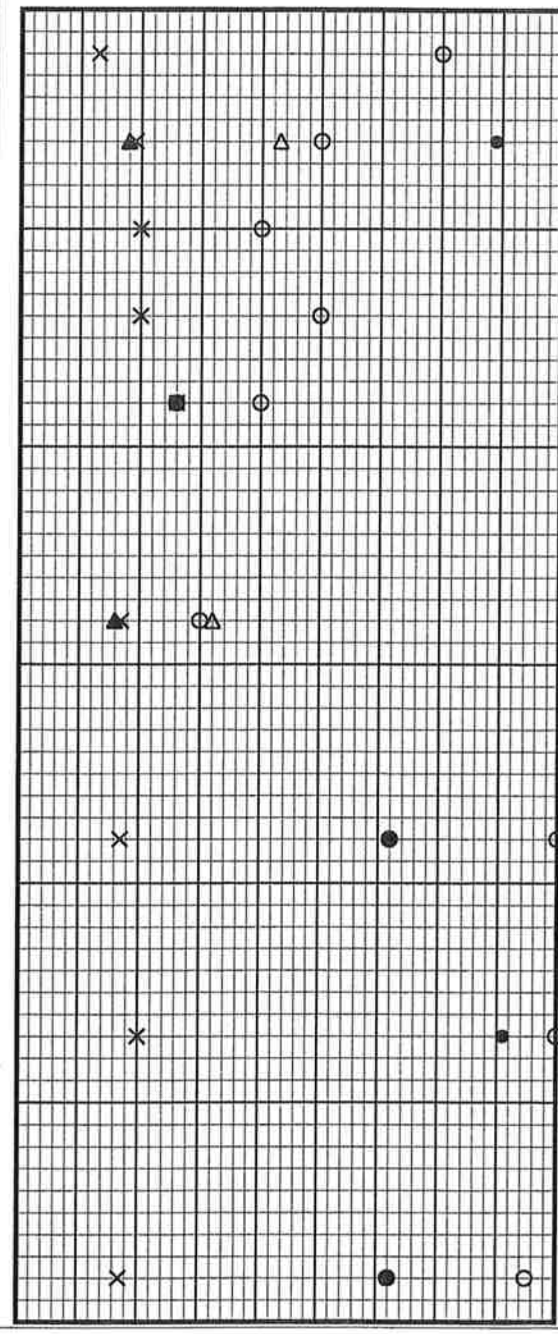
DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					2" Asphalt and 6" Lime stabilized Base
		1			Very stiff dark gray **Sandy Clay (CL)**
2.0		2			.. stiff below 2 feet
4.0		3			Stiff gray and tan **Clay (CH)**
6.0		4			.. light gray and tan with calcareous nodules below 6 feet
8.0		5			
10.0					
		6			Firm light gray and tan **Sandy Clay (CL)**
15.0					
		7			.. very stiff reddish brown and light gray below 18 feet
20.0					
		8			
25.0					
		9			.. reddish brown with calcareous nodules below 28 feet
30.0					continued

TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
X MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: 22'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 20+00 OFFSET
Depth to Water at 24 hrs: 18.4'			GROUP LEVEL(MSL): 56.6'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 35'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

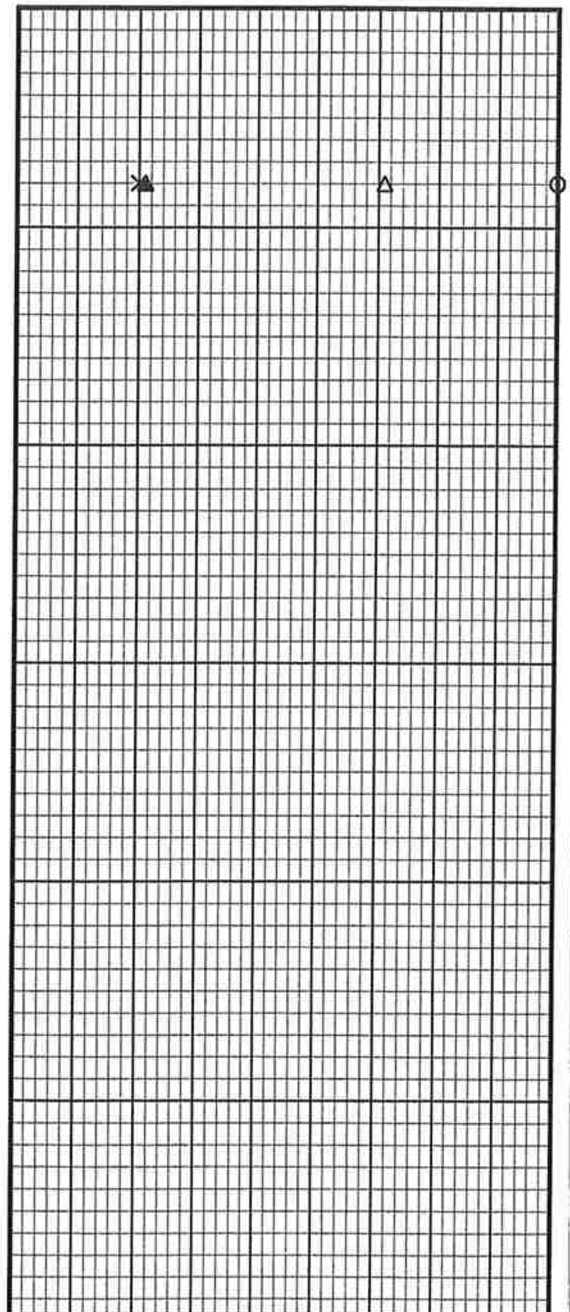
Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-3
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
35.0		10			Very stiff reddish brown **Clay (CH)** with calcareous nodules
					Boring was terminated at 35 feet
40.0					
45.0					
50.0					
55.0					
60.0					

TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
× MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90
0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: 22'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 20+00 OFFSET
Depth to Water at 24 hrs: 18.4'			GROUP LEVEL(MSL): 56.6'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 35'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 2 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	**BORING LOG**	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-4
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

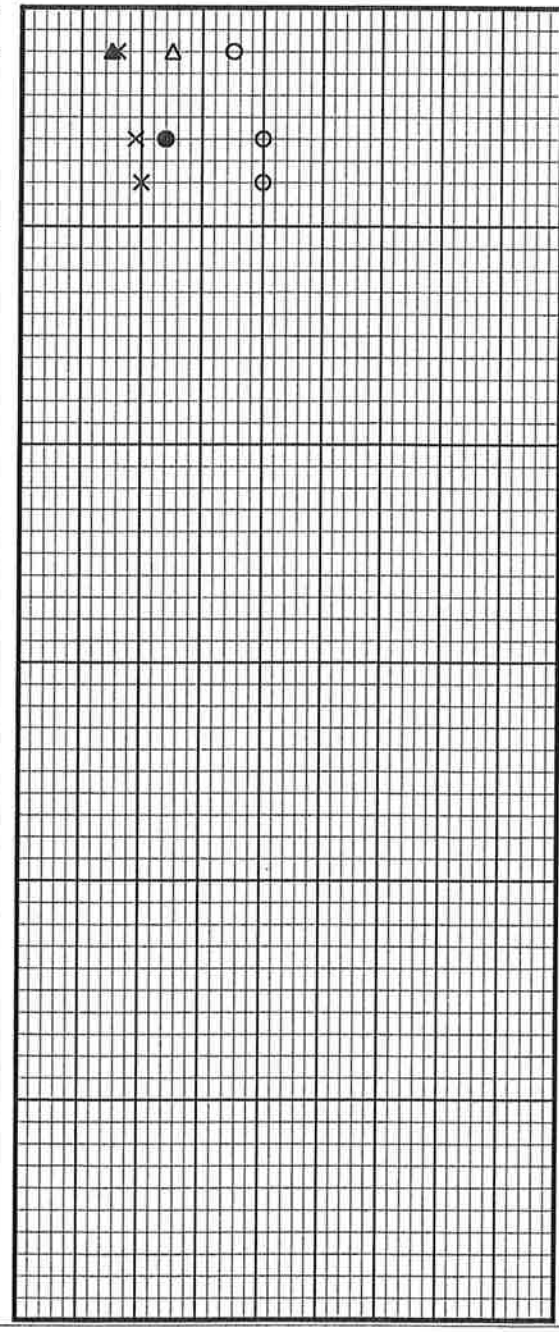
DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					4" Asphalt and 5" Lime base
		1			Firm gray and tan **Sandy Clay** **(CL)**
2.0		2			.. stiff below 2 feet
4.0		3			.. light gray and tan below 4 feet
					Boring was terminated at 5 feet
6.0					
8.0					
10.0					
15.0					
20.0					
25.0					
30.0					

TEST RESULTS

O PENETROMETER TEST
● UNCONFINED COMP.
Δ LL
▲ PL
× MOIST.(%)
● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: None	DRILLED BY: H G.	STARTED: 01/15/01	STATION 25+00 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 55.2'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 5'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

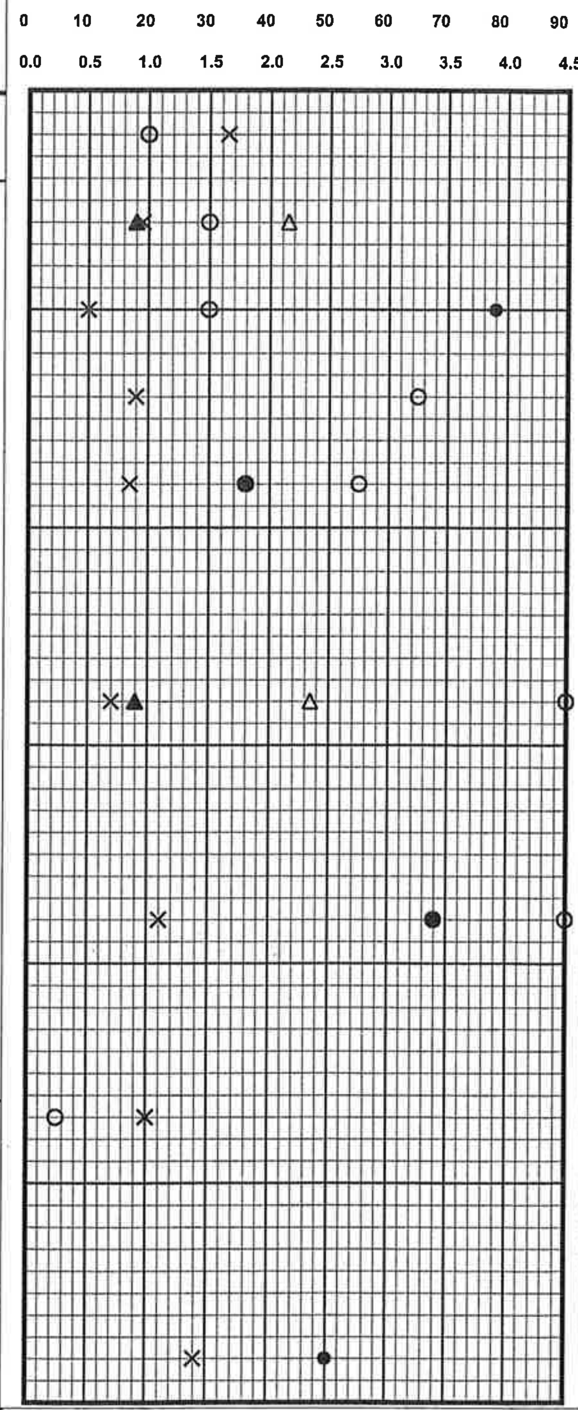
ASSOCIATED TESTING LABORATORIES, INC.

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-5
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
2.0		1			Firm gray and tan **Sandy Clay (CL)** with shells and asphalt (Fill)
4.0		2			Stiff light gray and tan **Sandy Clay (CL)**
6.0		3			.. with calcium deposite layer from 4 to 6 feet
8.0		4			.. stiff with ferrous nodules below 6 feet
10.0		5			
15.0		6			.. very stiff below 13 feet
20.0		7			.. tan and light gray below 18 feet
25.0		8			Very soft reddish brown **Clayey Silt (CL-ML)** (Wet)
30.0		9	24		.. with silty clay layers from 28 to 30 feet

continued

TEST RESULTS



Water First Noticed: 24'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 30+78 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 52.2'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: 22'			E:
COMPLETION DEPTH: 55'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

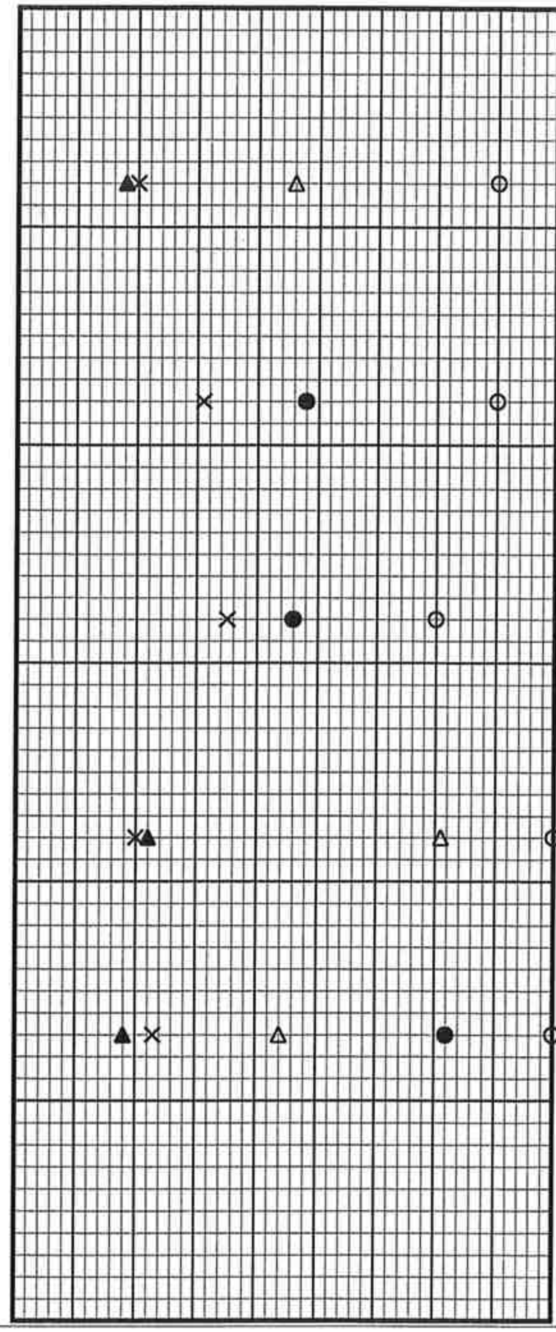
Project Name: Accelerated Surface Water Transmission Program along Link Rd.	**BORING LOG**
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801 / BORING NUMBER: GB-5
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
35.0		10			Very stiff reddish brown **Clay** (CH)
40.0		11			.. with slicken-sided layers below 38 feet
45.0		12			.. with slicken-sided layer below 43 feet
50.0		13			
55.0		14			Very stiff reddish brown **Sandy Clay** (CL)
					Boring was terminated at 55 feet
60.0					

TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
X MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90
0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: 24'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 30+78 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 52.2'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: 22'			E:
COMPLETION DEPTH: 55'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 2 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-6 (PZ-1)
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

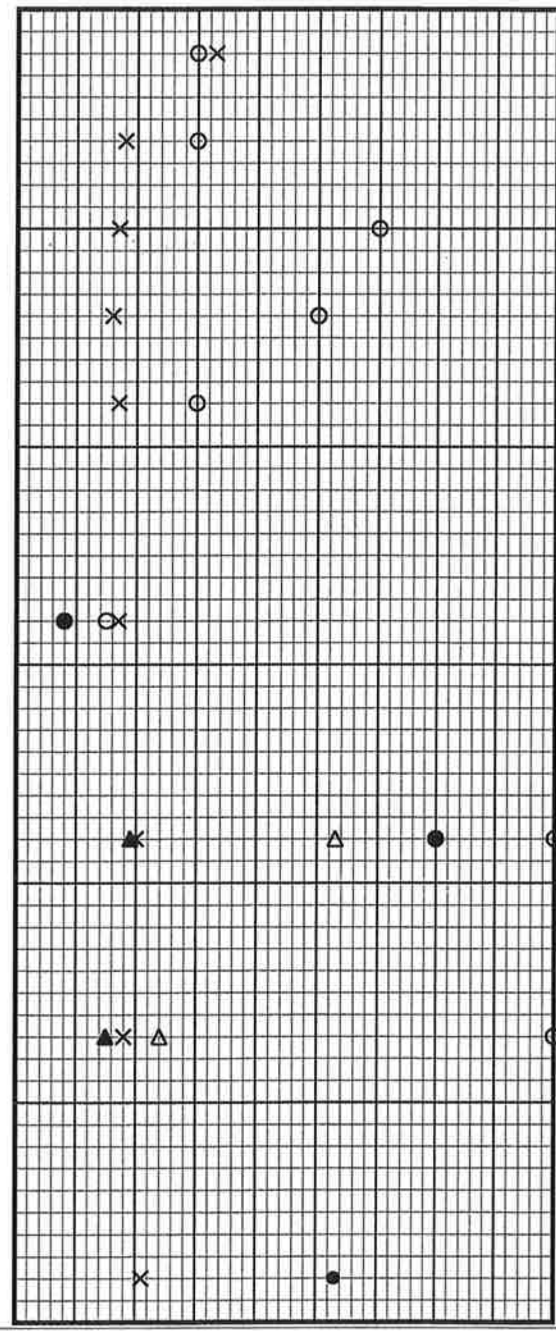
TEST RESULTS

O PENETROMETER TEST — ● UNCONFINED COMP.
Δ LL — ▲ PL
× MOIST.(%) — ● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					10.5" Asphalt and 4" crushed shell base
		1			Firm gray and tan **Sandy Clay (CL)** with roots (Fill)
2.0		2			.. with asphalt below 2 feet
4.0		3			.. very stiff with shells and gravels below 4 feet
6.0		4			
8.0		5			.. firm below 8 feet
10.0					
		6			.. soft below 13 feet
15.0					Very stiff light gray and tan **Clay (CH)**
		7			
20.0					
		8			Very stiff tan and light gray **Sandy Clay (CL)** with with calcareous nodules
25.0					
		9	31		Dense reddish brown **Silty Sand (SM)** with clay binder
30.0					



continued

Water First Noticed: None	DRILLED BY: H G.	STARTED: 01/20/01	STATION 33+80 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 50.2'
PZ WATER LEVEL: 14' (01/22/01)	LOGGED BY: H.G.	COMPLETED: 01/20/01	COORDINATES N:
HOLE CAVED AT: 30			E:
COMPLETION DEPTH: 50'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	**BORING LOG**	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-6 (PZ-1)
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

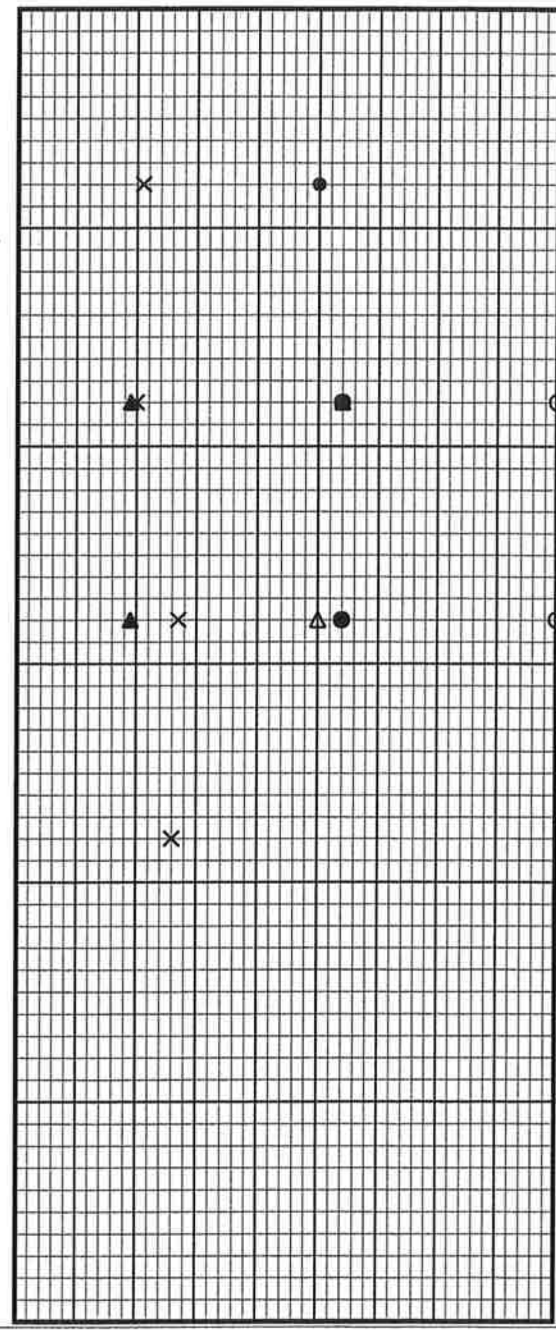
TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
X MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
35.0		10	25		
40.0		11			Very stiff reddish brown Clay (CH)
45.0		12			.. with slicken-sided layer below 43 feet
50.0		13			
55.0					Boring was terminated at 50 feet
60.0					



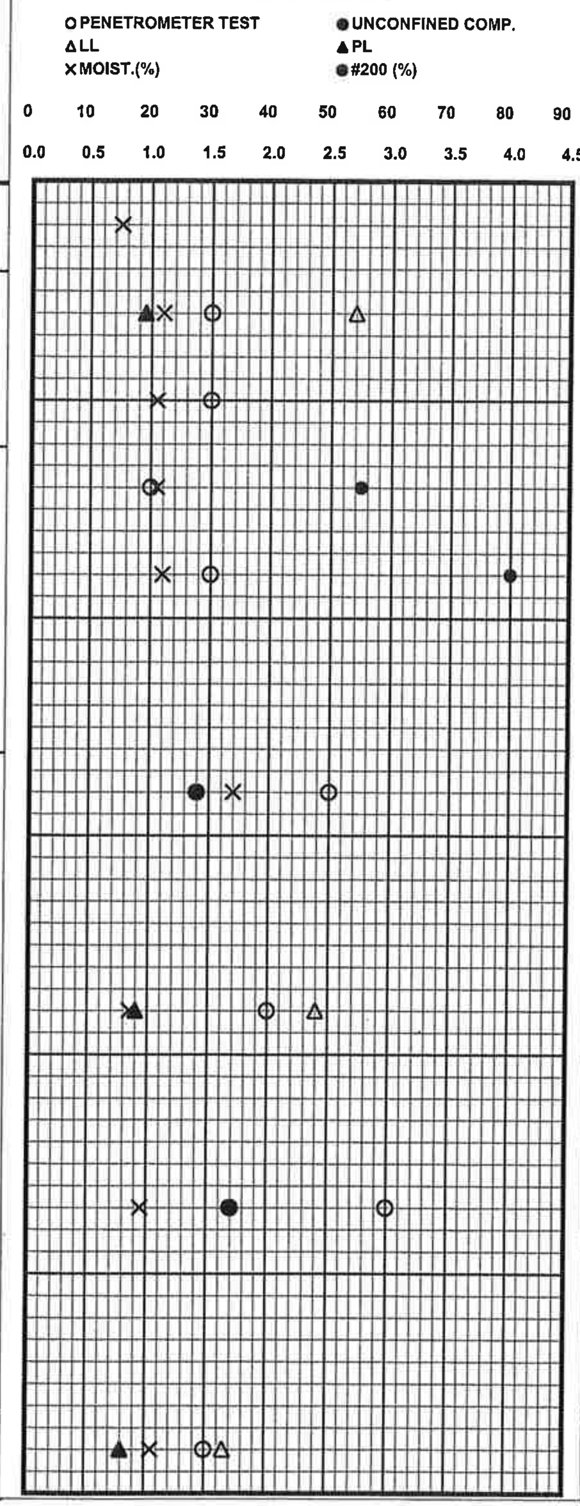
Water First Noticed: None	DRILLED BY: H G.	STARTED: 01/20/01	STATION 33+80 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 50.2'
PZ WATER LEVEL: 14' (01/22/01)	LOGGED BY: H.G.	COMPLETED: 01/20/01	COORDINATES N:
HOLE CAVED AT: 30'			E:
COMPLETION DEPTH: 50'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 2 OF 2
GROUT:			

ASSOCIATED TESTING LABORATORIES, INC.

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-7
GEOTECHNICAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
2.0		1			Gravel and asphalt (2' Fill)
4.0		2			Firm light gray and tan **Clay (CH)**
6.0		3			.. with calcium deposite layer from 2 to 6 feet
8.0		4			Firm light gray and tan **Sandy Clay (CL)**
10.0		5			.. reddish brown and light gray below 8 feet
15.0		6			Stiff reddish brown **Clay (CH)** with calcareous and ferrous nodules
20.0		7			.. light gray and tan below 18 feet
25.0		8			.. with calcareous nodules below 23 feet
30.0		9			Firm tan and light gray **Sandy Clay (CL)**

TEST RESULTS



continued

Water First Noticed: 11'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 39+80 OFFSET
Depth to Water at 24 hrs: 18.2'			GROUP LEVEL(MSL): 53.5'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: 22'			E:
COMPLETION DEPTH: 45'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-7
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

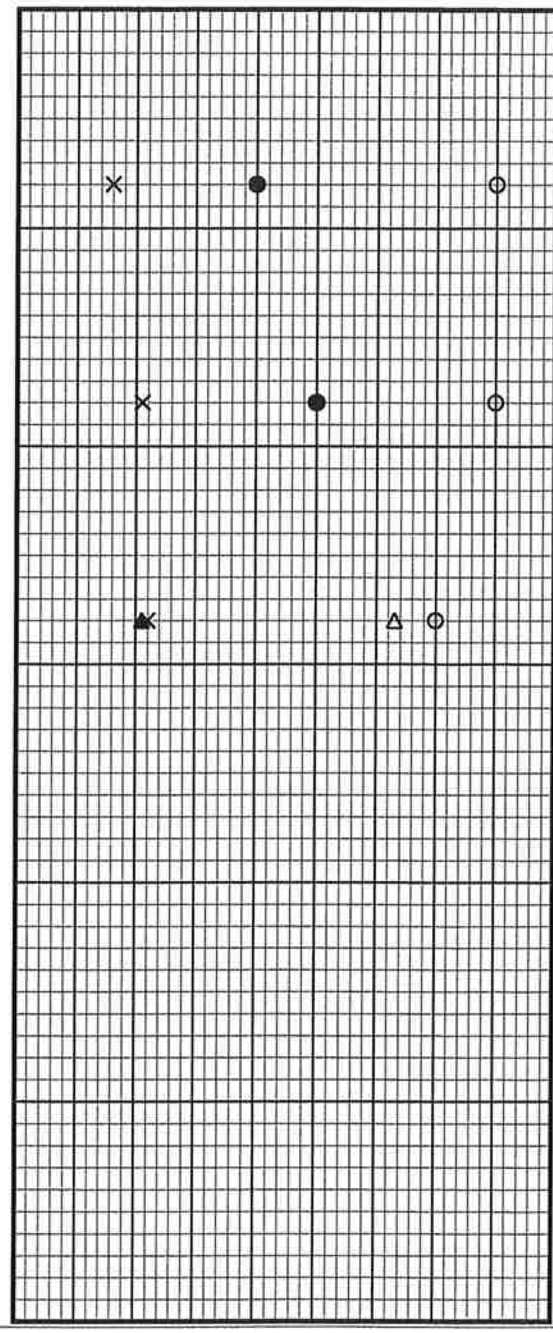
DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
		10			Very stiff reddish brown **Clay** (CH)
35.0					
		11			
40.0					
		12			.. with sandy clay layers at bottom
45.0					Boring was terminated at 45 feet
50.0					
55.0					
60.0					

TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
× MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: 11'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 39+80 OFFSET
Depth to Water at 24 hrs: 18.2'			GROUP LEVEL(MSL): 53.5'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: 22'			E:
COMPLETION DEPTH: 45'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 2 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-8
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

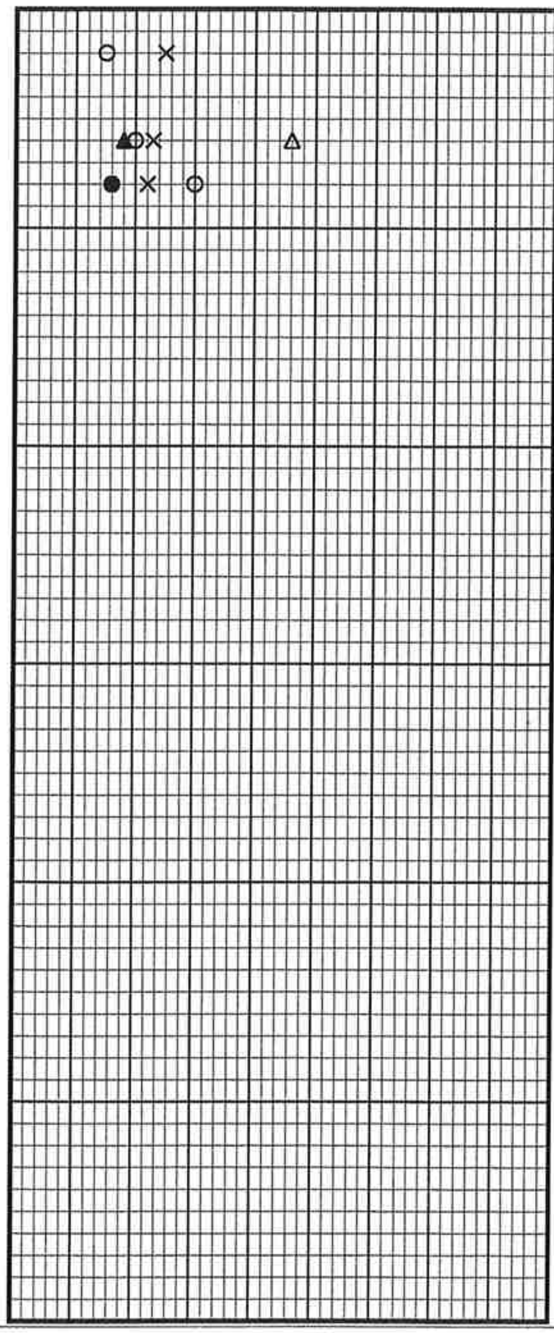
DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					2" Asphalt and 9" Lime stabilized with soil
2.0		1			Soft gray and tan **Sandy Clay** **(CL)** with gravel and shell (Fill)
		2			.. firm below 2 feet
4.0		3			.. light gray and tan below 4 feet
					Boring was terminated at 5 feet
6.0					
8.0					
10.0					
15.0					
20.0					
25.0					
30.0					

TEST RESULTS

O PENETROMETER TEST
● UNCONFINED COMP.
Δ LL
▲ PL
× MOIST.(%)
● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5



Water First Noticed: None	DRILLED BY: H G.	STARTED: 01/15/01	STATION 44+96 OFFSET
Depth to Water at 24 hrs: None			GROUP LEVEL(MSL): 55.8'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: None			E:
COMPLETION DEPTH: 5'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

Project Name: Accelerated Surface Water Transmission Program along Link Rd.	BORING LOG	
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1	PROJECT NUMBER: G00-801	BORING NUMBER: GB-9
GEOTECHNICIAL CONSULTANT: ASSOCIATED TESTING LABORATORIES, INC.	DESIGN CONSULTANT: Thomson Professional Group, Inc.	

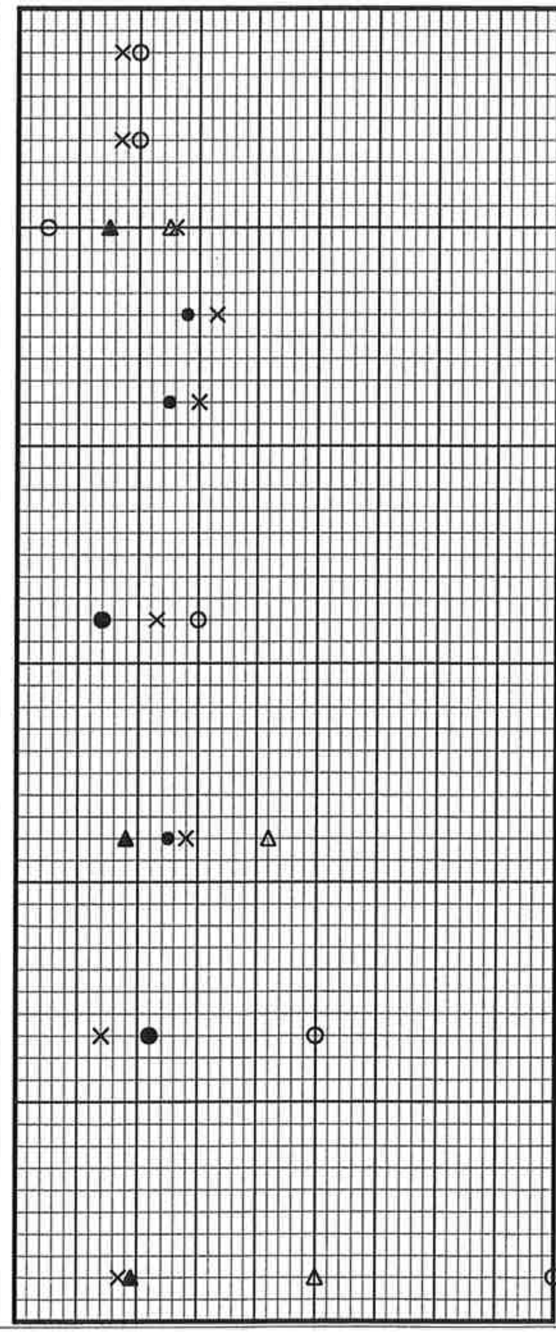
TEST RESULTS

O PENETROMETER TEST ● UNCONFINED COMP.
Δ LL ▲ PL
X MOIST.(%) ● #200 (%)

0 10 20 30 40 50 60 70 80 90

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5

DEPTH, FEET	SAMPLE TYPE	SAMPLE NUMBER	SPT	LEGEND	MATERIAL DESCRIPTION
					2.5" Asphalt and 4" Lime stabilized Base
		1			Firm light gray and tan **Sandy Clay** **(CL)** (Fill)
2.0		2			.. with calcareous nodules below 2 feet
4.0		3			.. gray and tan below 4 feet
6.0		4			Tan **Silty Sand** **(SM)** (Fill) (moist.)
8.0		5			
10.0					Firm tan and light gray **Sandy Clay** **(CL)**
		6			
15.0		7	21		.. sand layer at 18 feet
20.0					Stiff light gray and tan **Clay** **(CH)** with sandy clay
		8			
25.0		9			.. very stiff reddish brown below 28 feet
30.0					Boring was terminated at 30 feet



Water First Noticed: 10'	DRILLED BY: H G.	STARTED: 01/15/01	STATION 49+58 OFFSET
Depth to Water at 24 hrs: 13.3'			GROUP LEVEL(MSL): 56'
PZ WATER LEVEL: None	LOGGED BY: H.G.	COMPLETED: 01/15/01	COORDINATES N:
HOLE CAVED AT: 17'			E:
COMPLETION DEPTH: 30'	CHECKED BY: SAM	APPROVEN: J.A.	SHEET 1 OF 2
GROUT:			

APPENDIX 4

SUMMARY OF LABORATORY TESTING RESULTS

ASSOCIATED TESTING LABORATORIES, INC.
3143 YELLOWSTONE BLVD., HOUSTON, TEXAS 77052
TEL: (713) 748-3717 FAX: (713) 748-3748

Project Name: Accelerated Surface Water Transmission Program along Link Rd.
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1
Project Number: G00-801

Boring No.	Sample			Water Content (%)	Dry Density (pcf)	Atterberg Limits			Particle Size					Unconfined Comp. (TSF)	Triaxial Comp. Strength			Consolidation	Corrosion			
	No.	Depth (ft)	Type			LL	PL	PI	Grv. (%)	Sand (%)	Silt (%)	Clay (%)	#200 (%)		Figure No.	Max. Deviator Stress (ksf)	Conf. Press PSI		pH	R (ohm-cm)	SO4 (%)	Cl (%)
GB-1	1	0-2	UD	16																		
	2	2-4	UD	18									77									
	3	4-6	UD	18																		
	4	6-8	UD	19		45	18	27														
	5	8-10	UD	18	103									1.9								
	6	13-15	UD	19		31	16	15														
	7	18-20	UD	19	108								84	1.5								
	8	23-25	UD	18	106	34	16	18						1.7								
GB-2	1	0-2	UD	20																		
	2	2-4	UD	22																		
	3	4-5	UD	16	107	45	18	27						2.4								
GB-3	1	0-2	UD	13																		
	2	2-4	UD	19		43	18	25					79									
	3	4-6	UD	20																		
	4	6-8	UD	20																		
	5	8-10	UD	26	92									1.3								
	6	13-15	UD	17		32	16	16														
	7	18-20	UD	17	107									3.1								
	8	23-25	UD	20									81									
	9	28-30	UD	17	106									3.1								
	10	33-35	UD	20		61	21	40														

Legend:
UD - Undisturbed Sample Extruded in Field
UL - Undisturbed Sample Extruded in Lab
Designates consolidation test Performed
AU - Auger Cutting in Field
SS - Split Spoon Sample

ASSOCIATED TESTING LABORATORIES, INC.
3143 YELLOWSTONE BLVD., HOUSTON, TEXAS 77052
TEL: (713) 748-3717 FAX: (713) 748-3748

Project Name: Accelerated Surface Water Transmission Program along Link Rd.
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1
Project Number: G00-801

Boring No.	Sample			Water Content (%)	Dry Density (pcf)	Atterberg Limits			Particle Size					Unconfined Comp. (TSF)	Triaxial Comp. Strength			Consolidation	Corrosion			
	No.	Depth (ft)	Type			LL	PL	PI	Grv. (%)	Sand (%)	Silt (%)	Clay (%)	#200 (%)		Figure No.	Max. Deviator Stress (ksf)	Conf. Press PSI		pH	R (ohm-cm)	SO4 (%)	Cl (%)
GB-4	1	0-2	UD	16		25	15	10														
	2	2-4	UD	19	101									1.2								
	3	4-5	UD	20																		
GB-5	1	0-2	UD	33																		
	2	2-4	UD	19		43	18	25														
	3	4-6	UD	10									78									
	4	6-8	UD	18																		
	5	8-10	UD	17	111									1.8								
	6	13-15	UD	14		47	18	29														
	7	18-20	UD	22	104									3.4								
	8	23-25	UD	20																		
	9	28-30	SS	28									50									
	10	33-35	UD	20		46	18	28														
	11	38-40	UD	31	96									2.4								
	12	43-45	UD	35	93									2.3								
	13	48-50	UD	20		71	22	49														
	14	53-55	UD	23	104	44	18	26						3.6								

Legend:
UD - Undisturbed Sample Extruded in Field
UL - Undisturbed Sample Extruded in Lab
Designates consolidation test Performed
AU - Auger Cutting in Field
SS - Split Spoon Sample

ASSOCIATED TESTING LABORATORIES, INC.
3143 YELLOWSTONE BLVD., HOUSTON, TEXAS 77052
TEL: (713) 748-3717 FAX: (713) 748-3748

Project Name: Accelerated Surface Water Transmission Program along Link Rd.
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1
Project Number: G00-801

Boring No.	Sample			Water Content (%)	Dry Density (pcf)	Atterberg Limits			Particle Size					Unconfined Comp. (TSF)	Triaxial Comp. Strength			Consolidation	Corrosion			
	No.	Depth (ft)	Type			LL	PL	PI	Grv. (%)	Sand (%)	Silt (%)	Clay (%)	#200 (%)		Figure No.	Max. Deviator Stress (ksf)	Conf. Press PSI		pH	R (ohm-cm)	SO4 (%)	Cl (%)
GB-6	1	0-2	UD	33																		
	2	2-4	UD	18																		
	3	4-6	UD	17																		
	4	6-8	UD	16																		
	5	8-10	UD	17																		
	6	13-15	UD	17	105									0.4								
	7	18-20	UD	20	106	53	19	34						3.5								
	8	23-25	UD	18		24	15	9														
	9	28-30	SS	21									53									
	10	33-35	SS	21									50									
	11	38-40	UD	26	107	54	19	35						2.7								
	12	43-45	UD	27	99	50	19	31						2.7								
	13	48-50	UD	26																		
GB-7	1	0-2	AU	15																		
	2	2-4	UD	22		54	19	35														
	3	4-6	UD	21																		
	4	6-8	UD	21									55									
	5	8-10	UD	22									80									
	6	13-15	UD	34	87									1.4								
	7	18-20	UD	17		48	18	30														
	8	23-25	UD	19	102									1.7								
	9	28-30	UD	21		33	16	17														
	10	33-35	UD	16	101									2.0								
	11	38-40	UD	21	94									2.5								
	12	43-45	UD	22		63	21	42														

Legend:
UD - Undisturbed Sample Extruded in Field
UL - Undisturbed Sample Extruded in Lab
Designates consolidation test Performed
AU - Auger Cutting in Field
SS - Split Spoon Sample

ASSOCIATED TESTING LABORATORIES, INC.
3143 YELLOWSTONE BLVD., HOUSTON, TEXAS 77052
TEL: (713) 748-3717 FAX: (713) 748-3748

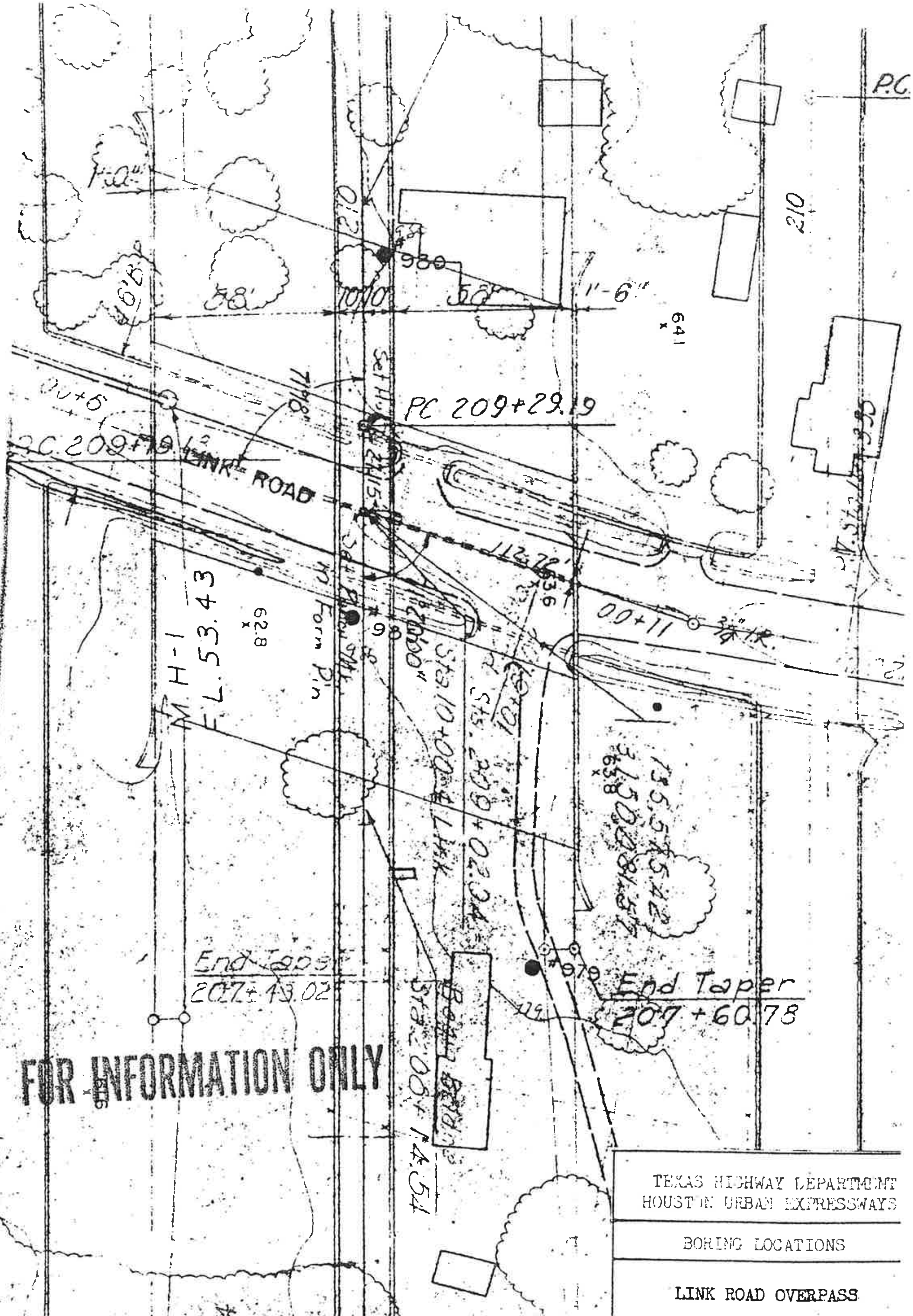
Project Name: Accelerated Surface Water Transmission Program along Link Rd.
GFS No. S-0900-64-3; File No. WA 10637; Contract No. 6C-1
Project Number: G00-801

Boring No.	Sample			Water Content (%)	Dry Density (pcf)	Atterberg Limits			Particle Size					Unconfined Comp. (TSF)	Triaxial Comp. Strength			Consolidation	Corrosion			
	No.	Depth (ft)	Type			LL	PL	PI	Grv. (%)	Sand (%)	Silt (%)	Clay (%)	#200 (%)		Figure No.	Max. Deviator Stress (ksf)	Conf. Press PSI		pH	R (ohm-cm)	SO4 (%)	Cl (%)
GB-8	1	0-2	UD	25																		
	2	2-4	UD	23		46	18	28														
	3	4-5	UD	22	99									0.8								
GB-9	1	0-2	UD	17																		
	2	2-4	UD	17																		
	3	4-6	UD	26		25	15	10														
	4	6-8	AU	33									28									
	5	8-10	AU	30									25									
	6	13-15	UD	23	94									0.7								
	7	18-20	SS	28		42	18	24					25									
	8	23-25	UD	14	111									1.1								
	9	28-30	UD	17		50	19	31														

Legend:
UD - Undisturbed Sample Extruded in Field
UL - Undisturbed Sample Extruded in Lab
Designates consolidation test Performed
AU - Auger Cutting in Field
SS - Split Spoon Sample

APPENDIX 5

BORING LOGS BY OTHERS



TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN EXPRESSWAYS

BORING LOCATIONS

LINK ROAD OVERPASS

LABORATORY LOG OF BORING No. 979
FOR
HIGHWAY I 45 LINK ROAD OVERPASS

DATE: 2-13-59 TYPE: 3" Shelby LOCATION: See Boring Loc.

ELEVATION + 63 ft.

Depth Feet	Description	Blows per Foot 1st 6"	Blows per Foot 2nd 6"	Cohesion P.S.I.	Friction Angle	Unit Dry Wt. Lbs/Cu. Ft.
0	Very Stiff Lt. Gray Sandy Clay w/Calc.					
10	-Becomes Lt. Gray Tan w/o Calc.	11	12			
				3	19	115
20		16	19			
30	-Becomes w/Shale					
	Hard Red Blue Silty Clay w/Shale Becomes w/o Shale @ 36'					
40		20	25			
				13	17	101
50		23	32			
	Hole Completed @ 55 Ft.					
60						
70						

POINT BEARING T.S.F.: 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT: 10 20 30 40 50

FOR INFORMATION ONLY

N O W A T E R

LABORATORY LOG OF BORING No. 985
FOR
HIGHWAY I 45 LINK ROAD OVERPASS

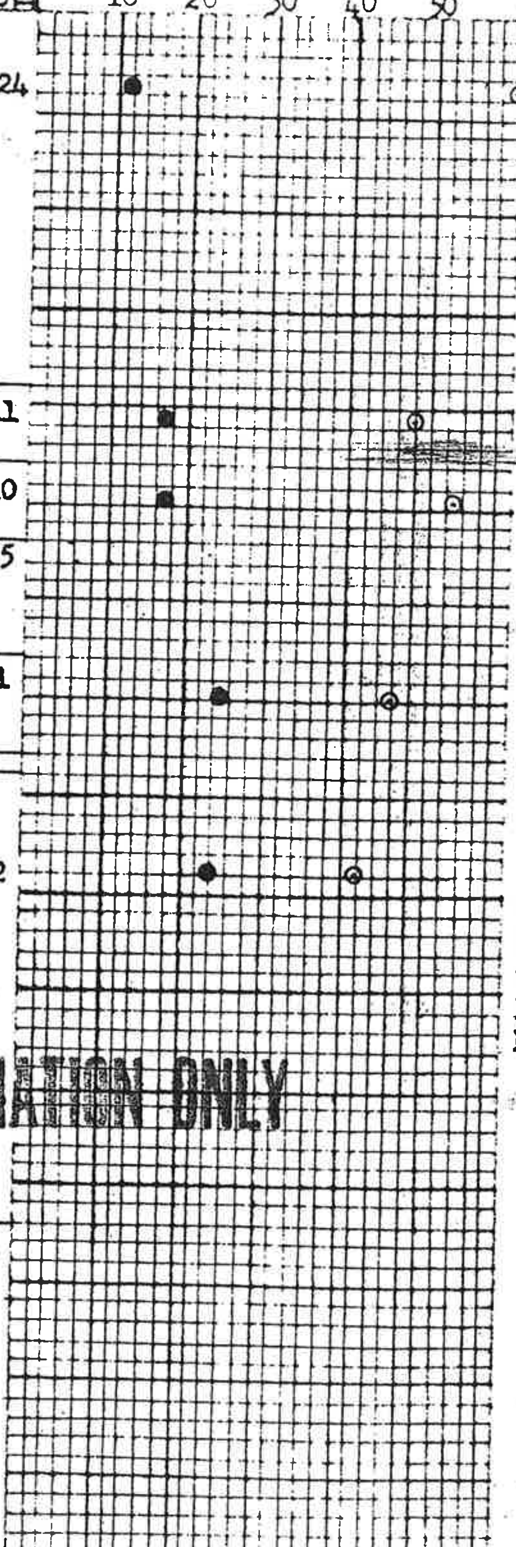
DATE: 3-18-59 | TYPE: 3" Shelby | LOCATION: See Boring Loc.

ELEVATION + 63.0

POINT BEARING T.S.F.: 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT: 10 20 30 40 50

DEPTH FEET	DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS/CU. FT.
0	Hard Lt. Gray Sandy Clay w/Calc.			10	26	124
10		12	12			
20	Hard Red Blue Silty Clay	16	24	15	18	111
	Hard Lt. Gray Tan Sandy Clay Slickensided			12	21	110
30	Dense Brown Lt. Gray Clayey Sand					105
		17	18			
	Hard Red Blue Silty Clay Slickensided			5	22	101
40	Very Dense Brown Lt. Gray Silty Sand					
	Hard Red Blue Silty Clay	19	24			
				12	13	102
50		37	38			
60		20	26			
	Hole Completed @ 62 Ft.					
70						



WATER LEVEL NOT NOTED

FOR INFORMATION ONLY

LABORATORY LOG OF BORING No. 980
FOR
HIGHWAY I 45 LINK ROAD OVERPASS

DATE: 3-17-59 TYPE: 3" Shelby LOCATION: See Boring Loc.

ELEVATION + 64.0

DEPTH FEET	DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS/CU. FT.
0	Very Stiff Lt. Gray Sandy Clay			5	16	112
	-Becomes Tan Lt. Gray Sandy Clay w/Calc.					
10		12	15			
20		16	20			
	Very Dense Brown Lt. Gray Clayey Sand			1	20	105
	Dense Brown Lt. Gray Clayey Silt w/Shale	20	32	5	19	100
30	Hard Red Blue Silty Clay w/Shale Slickensided			14	17	100
	Very Dense Brown Lt. Gray Silty Sand	50	5"			
40	Hard Red Blue Silty Clay Slickensided			18	17	103
		23	31			
50	-Becomes Hard					
60		19	29			
	Hole Completed @ 60 Ft.					
70						

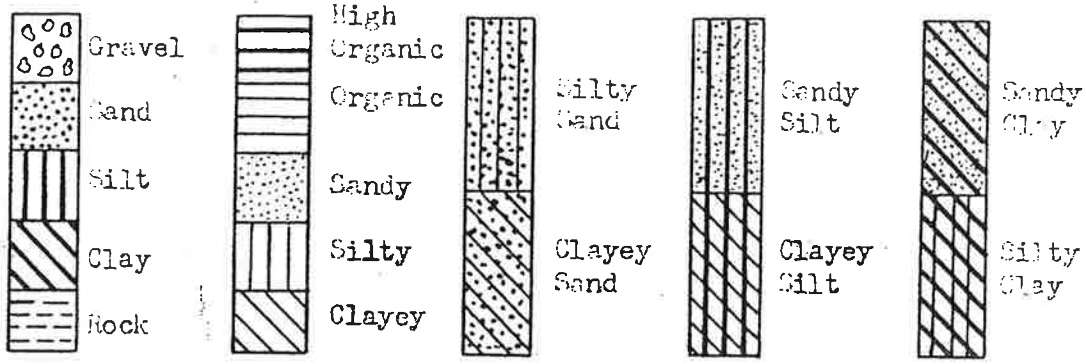
POINT BEARING T.S.F. 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT 10 20 30 40 50

NO WATER LEVEL NOTED.

FOR INFORMATION ONLY

Key to Soil Symbols and Description Used in Laboratory Logs



Color

In color description of sample, the predominating color is stated first

Consistency of Cohesive Soils

Classification:	Field Identification:	Laboratory Identification:
Very soft	- Tall core will sag under own weight	Less than 0.25 Tsf Unconfined Compressive Strength
Soft	- Core can be pinched between thumb and forefinger.	0.25 - 0.50 "
Med. Stiff	- Core can be easily imprinted with fingers.	0.50 - 1.00 "
Stiff	- Can be imprinted with considerable pressure from finger.	1.00 - 2.00 "
Very Stiff	- Can be imprinted very slightly with pressure from fingers.	2.00 - 4.00 "
Hard	- Cannot be imprinted with fingers; may be penetrated with pencil.	Over 4.00 "

FOR INFORMATION ONLY

Relative Density of Cohesionless Soils

Classification by Standard (THD) Penetration Resistance:

Loose	- 0 to 12 Blows per foot	Dense	- 30 to 50 blows per foot
Med. Dense	- 12 to 30 "	Very Dense	- 50 and above "

Soil Structure

Slickensided	- Cut by old fracture planes which are slick and glossy in appearance.
Fractured	- Containing old shrinkage cracks, frequently filled with fine sand, silt, or clay of color differing from main soil.
Varved	- Composed of thin laminae of varying color and soil types.
Interbedded	- Composed of alternate layers of different soil types.
Calcareous	- Contains deposits of calcium carbonate.