FINAL REPORT
GEOTECHNICAL INVESTIGATION
PROPOSED CAMBRIDGE STREET
BETWEEN HOLCOMBE AND SOUTH MACGREGOR
HOUSTON, TEXAS

FOR

SCIENTECH ENGINEERS, INC.
5701 WOODWAY, SUITE 200
HOUSTON, TEXAS 77057

PREPARED BY
ASSOCIATED TESTING LABORATORIES, INC.
HOUSTON, TEXAS

JANUARY 2005
REPORT NO. G04-504

ASSOCIATED TESTING LABORATORIES, INC.

ESTABLISHED 1959

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Date: January 24, 2005

ATL Job No: G04-504

Scientech Engineers, Inc.
5701 Woodway, Suite 200
Houston, Texas 77057

Attention: Mr. Ricky Gonzalez

Reference: Final Report
Geotechnical Investigation
Proposed Cambridge Street
Between Holcombe and South MacGregor
Houston, Texas

Dear Mr. Gonzalez:

We have completed 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 Manager

GEOTECHNICAL INVESTIGATION

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0.0 EXECUTIVE SUMMARY

Associated Testing Laboratories, Inc. (ATL) has conducted a Geotechnical Investigation for the proposed Cambridge Street between Holcombe and South MacGregor including a new roadway bridge over Brays Bayou in Houston, Texas. We understand that it is planned to construct Cambridge Street between Holcombe and South MacGregor. For this purpose a new Cambridge Street bridge across Brays Bayou will be constructed along with the street pavement between Holcombe and South MacGregor. Additionally, a bikeway trail will be constructed on both sides of the bayou. The existing side slope of the bayou is not expected to be changed. The purpose of this study was to determine subsurface conditions in the project area and to develop recommendations for the design and construction of proposed roadway bridge, abutment walls, new asphalt or concrete pavement and to check the stability of the existing side slopes of Brays Bayou.

A total of seven (7) soil borings were drilled in the project area to depths ranging from 25- to 120-feet. The depth of the groundwater in the proposed project area was measured below the depths of 15- to 23-feet below the existing ground surface during drilling and at a depth of about 16.5-feet in the piezometer PZ-1(Boring GB-1) during the latest reading on 12/15/04. ATL's subsurface investigation disclosed the following details regarding the subsurface soil types in the proposed project area:

A - Cohesive Soils:

Cohesive soils are present in the subsurface throughout the project area. The cohesive soils consisted of fill sandy clay (CL), natural clay (CH) and natural sandy clay (CL). They were noted from the ground surface to varying depths ranging up to the maximum depth of the borings at 120-feet.

B - Granular Soils:

Granular soils were encountered at most boring locations at varying depths. At boring GB-5, the fill granular soils were encountered between the depths of 6- to 8-feet. In general, the granular soils were encountered at varying depths starting below the depths ranging from 17- to 42-feet and extending to varying depths ranging from 27- to 51-feet. At boring GB-3, these soils were encountered between 72- to 75-feet. These soils were again encountered at boring GB-5 between the depths of 101- to 111-feet. The granular soils consist of silty sands (SM).

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 active or known faults in the proposed project area.

- Abutment walls may be designed by using equivalent fluid pressure approach or the criteria given in the report. More information is given in the report.

- Based on the our preliminary slope stability analysis using the computer program PCSTBL5M and using topographic information supplied to us by the client, the existing southern side slopes of the Brays Bayou are stable for short term, long term and rapid draw down conditions. The existing northern side slopes of the Brays Bayou are stable for short term and rapid draw down conditions and marginally stable for long term condition.

- Our recommedations for the roadway bridge are given in section 5.3. Design curves have been developed to calculate the drilled pier compressive and uplift capacity at the top bank and for the interior bents. Recommendations for lateral capacity are also given. The drilled pier depths should be so selected that it is safe against compressive, uplift and lateral loads.

- Based on the AASHTO procedure for design of rigid pavements and for the design ESAL of 10,000,000; the recommended concrete pavement thickness is 10.0 inches over 8.0 inches of lime stabilized (5% by dry weight) subgrade. More recommendations are given in the report.

- Reinforcements for the concrete pavement may be inaccordance with the latest City of Houston Standard Specifications shown in Drawing No. 02751-01. More information is given in the report.

I. FACTUAL DATA

1.0 INTRODUCTION

1.1 General

This investigation was authorized by Scientech Engineers, Inc., with the acceptance of the **Associated Testing Laboratories, Inc.,** Proposal No. CP03-1001 dated October 28, 2004. Project details were provided to ATL by Mr. Ricky Gonzalez., of Scientech Engineers, Inc. This report includes results of the field investigation, laboratory testing, geotechnical engineering analysis, and recommendations for the proposed design and construction of drilled piers for the new bridge structure, results of slope stability analysis for the bayou side slopes for existing slope section at bridge locations, recommendations for abutment walls and recommendations for asphalt and concrete pavement.

1.2 Location and Description of the project

The project alignment is the proposed construction of the Cambridge Street between Holcombe and South MacGregor in Houston, Texas. The project alignment crosses the Brays Bayou between Holcombe and South MacGregor. A general site vicinity map of the project area is shown on Figure 1. It is planned to construct a new roadway bridge across Brays Bayou along the proposed Cambridge Street between Holcombe and South MacGregor. In addition bikeway trail will be constructed along the bayou on both sides. We understand that the existing bayou slope will remain the same. The new pavement for the proposed Cambridge Street between Holcombe and South MacGregor will consist of concrete. No other information is available to us at this time.

The project alignment includes a proposed roadway between the existing Holcombe Street and South MacGregor. It crosses over the Brays Bayou and includes about 300-feet along Brays Bayou to the east and west of the proposed bridge crossing. Businesses, Brays Bayou, golf course, Holcombe, South and North MacGregor Streets are located at or near the project alignment. The northern end of the project is currently fenced and used by construction personnel for some possible utility construction. The southern end of the project is adjacent the occupied McDonald House and also fenced. 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 design and construction of new roadway bridge, abutment walls and concrete pavement. Additionally, the stability of the existing bayou side slopes was also investigated.

Associated Testing Laboratories, Inc. (ATL) has completed a subsurface exploration program for this project, which consisted of the following scope:

- Drilling and sampling a total of seven (7) soil borings to depths ranging from twenty five (25) feet to one hundred and twenty (120) feet below the existing ground surface level. Two (2) borings (GB-3 and GB-5) were drilled to a depth of 120-feet each at the bridge location. Two (2) borings (GB-4 and GB-6) were drilled to a depth of 80-feet each along the Brays Bayou. Two (2) borings were drilled to a depth of 25-feet each between Holcombe and Brays Bayou (GB-1 and GB-2) and one (1) boring was drilled to a depth of 25-feet between South MacGregor and Brays Bayou (GB-7). Two (2) borings (GB-1 and GB-7) were converted into piezometers (PZ-1 and PZ-2, respectively) after completion of drilling and sampling.

- 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.

Based on results from the field investigation, laboratory testing and gathered geological information, ATL performed an engineering analysis to develop geotechnical recommendations for the design and construction of a new roadway bridge and abutment walls and for analyzing the stability of the existing bayou slope sections and for the new pavement.

2.0 SUBSURFACE INVESTIGATION PROGRAM

The field investigation for this project consisted of drilling and sampling of a total of seven (7) soil borings in the project area.

Boring locations as drilled for this geotechnical exploration are shown in Figure 2. The soil borings were drilled to depths ranging from twenty five (25) to one hundred and twenty (120) feet below the ground level for a total of 475 feet of drilling and sampling. The borings depths and locations were approved by the client. Two (2) borings (GB-1 and GB-7) were converted into piezometers (PZ-1 and PZ-2, respectively) after completion of drilling and sampling. The structure of the piezometer well is shown in Figure 3. Listed in Table 2 are the results of our groundwater readings. Dry auger drilling methods were adopted to drill the soil borings for the shallow 25-feet borings and till the encountering of ground water for the deeper borings. Below the groundwater depths, wash boring was used for the deeper borings. 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. At boring GB-2, 2-inch O.D. Shelby tube soil samples were collected using a hand operated portable rig due to access difficulties prohibiting the use of a truck mounted hydraulic drilling rig. 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), Consolidated Undrained Triaxial Compression tests (ASTM D-4767), California Bearing Ratio (ASTM D-1883) and Standard Proctor Density (ASTM D-698) tests. The results of laboratory tests are presented on the boring logs in Appendix 3 and summarized in tables of Appendix 4A. The results of California Bearing Ratio (CBR) and Proctor tests are presented in Table 2. The Consolidated Undrained Triaxial Compression tests were performed on our behalf by HTS, Inc. Their test results are presented in Appendix 4B. Overall numbers and types of tests performed or currently being performed for this study for this project are presented below:

TYPE OF TEST	NUMBER OF TEST
Dry Density	44
Moisture Content	116
Atterberg Limits	45
Unconfined Compression	44
Sieve Analysis thru #200	6
Consolidated Undrained Triaxial Compression	4
Optimum Moisture and Density (Proctor)	2
California Bearing Ratio	2
Consolidated Undrained Triaxial	4

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 upthrust 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. Conversion to surface water usage and the limiting of oil production has greatly reduced the subsidence rate in the area east of 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 any known faults present in the proposed project area. Figure 4 shows the principal active faults 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 different locations of the project area are presented below:

4.3.1 - Cohesive Soils:

Cohesive soils were encountered from the ground surface to varying depths ranging up to the maximum depth of the borings at 120-feet. The cohesive soils consisted of fill sandy clay (CL), natural sandy clay (CL) and natural clay (CH). Fill sandy clay (CL) soils were encountered to depths ranging from 4- to 26-feet. Natural clay (CH) soils were encountered at borings GB-1, GB-5, GB-6 and GB-7 beginning at depths varying from ground surface to 8-feet and extending to depths ranging from 12- to 25-feet. Natural sandy clay (CL) soils were encountered at borings GB-1, GB-4 through GB-7 at varying depths below 12- to 27-feet and extending to varying depths ranging from 21- to 43-feet. At boring GB-2, natural sandy clay (CL) were encountered between the depth of 4- to 8-feet. At boring GB-5, natural sandy clay (CL) soils were encountered between the depths of 51- to 81-feet and again between the depths of 88- to 101-feet. At borings GB-3, GB-4 and GB-6, natural clay (CH) soils were encountered at varying depths starting below depths ranging from 31- to 51-feet and extending to depths ranging up to the maximum depth of the borings at 120-feet. At boring GB-5, natural clay (CH) soils were encountered between the depths of 81- to 88-feet and again below the depths of 111-feet and extending to the maximum depth of the boring to 120-feet.

The fill sandy clay soils were found to have a liquid limit ranging from about 31 to 43 %, a plastic limit ranging from about 16 to 18 % and a plasticity index ranging from about 15 to 25. The moderately expansive sandy clay soils (plasticity index above 20) are not suitable for use as select fill in their present condition. These soils once lime stabilized, 5% by dry weight, should be suitable for use as select fill. However, these soils in their present condition should be suitable for use as random fill. The natural clay (CH) soils have a liquid limit ranging from 48% to 88%, plastic limit ranging from 18% to 26% and plasticity indices ranging from 30 to 62. The clay soils are not suitable for use as select fill in their present condition. These soils once lime stabilized (7 % by dry weight) should be suitable for use as select fill. However, these soils in their present condition should be suitable for use as random fill material. The natural sandy clay soil was found to have a liquid limit ranging from 25 % to 46 %, a plastic limit ranging from 15 % to 18 % and plasticity indices ranging from 10 to 28. The moderately expansive sandy clay soils (plasticity index above 20) are not suitable for use as select fill in their present condition. These soils once lime stabilized, 5% by dry weight, should be suitable for use as select fill. However, these soils in their present condition should be suitable for use as random fill.

4.3.2 - Granular Soils:

Granular soils were encountered at most boring locations at varying depths. At boring GB-5, fill granular soils were encountered between the depths of 6- to 8-feet. The granular soils were generally encountered at varying depths starting below the depths ranging from 17- to 42-feet and extending to varying depths ranging from 27- to 51-feet. The granular soils were also encountered at boring GB-3 between the depths of 72- to 75-feet and at boring GB-5 between the depths of 101- to 111-feet. The granular soils consisted of silty sand (SM) soils. The silty sands are medium dense to very dense with blow counts ranging from 18 to greater than 50 for negligible penetration. The granular soils are not suitable for use as 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-7 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 logs profiles are presented as Figures 5A through 5C.

4.4 Groundwater

Groundwater conditions were observed in open soil borings during the field investigation. Groundwater was encountered during drilling at depths ranging from 15- to 23-feet. Two (2) of the soil borings (GB-1 and GB-6) were later converted into piezometers (PZ-1 and PZ-2, respectively) and water level will be measured over a period of few weeks after drilling. At the piezometer locations, groundwater was encountered at a depth of about 16.5- feet at PZ-1 (GB-1) at the latest reading taken on 12/15/04. Groundwater was not encountered at the other piezometer location PZ-2 (GB-7).

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 depths measured during and at completion of drilling and at piezometer locations are shown on the respective boring logs and summarized in Table 1.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

5.1 OSHA Type Soils

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;
- 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.

Based upon the soil conditions revealed by the borings, ATL recommends the use of OSHA soil classification Type "C" for the determination of allowable maximum slope or selection and design of the protective system.

5.2 Stability of Bayou Side Slopes

The existing bayou slope at the proposed Cambridge Street bridge location over Brays Bayou was analyzed by using the topographic information supplied to us by the client. Existing slope sections at the north and south side of the bayou were plotted and the slope stability analysis performed by considering the modified Bishop approach using the computer software PCSTABL5M. The slope was analyzed for the short term, long term and rapid draw down conditions. The short term conditions represents the existing or end of construction stage conditions and is analyzed using total stress analysis. The long term conditions represents the condition of the slope after passage of several years and is analyzed using effective stress concept. The rapid draw down condition represents a stage when the soils are saturated and there is no water in the bayou. This type of condition may occur if there are heavy rains leading to the flooding of the bayou and saturating of the soils on the high bank followed by a sudden withdrawal of water from the bayou before the high bank is drained. This condition can occur immediately after construction, take several years or not occur at all. In most cases this condition occurs partially in the sense that some drainage of the high bank soils does occur before the water level in the bayou drops to the bottom.

Section A-A as shown on Figure 2A was analyzed for stability. Section A-A is at the proposed Cambridge Street bridge location. Both the northern and southern side slopes of the Brays Bayou were analyzed at this section. Appendix 5 shows the results of our slope stability analysis at section A-A for both northern and southern side slopes.

The soil parameters shown on the plots were used for our analysis. Soil information as obtained from borings GB-3 and GB-5 was considered for the analysis of the southern and northern slope of the bayou, respectively, at section A-A. A slope with a factor of safety of 1.0 or less is unstable and likely to fail. A factor of safety between 1.0 to 1.2 is generally considered marginal. A factor of safety of 1.4 or above is generally preferred for a slope with important structures located on top of it.

Our slope stability analyses using the computer program PCSTBLE5M indicated a factor of safety under short term, long term and rapid drawdown condition to be about 2.27, 1.11 and 1.58, respectively at northern slope of bayou at section A-A and about 3.87, 1.75 and 2.69, respectively at the southern slope of bayou at section A-A. Hence based on our analysis, it appears that the existing southern side slopes of the bayou is stable for all three conditions. At the northern side of the bayou, the side slope is stable for short term and rapid draw down condition and marginally stable for long term conditions.

5.3 Bridge Structure

We understand that a new Cambridge Street roadway bridge over Brays Bayou is planned for connecting the proposed Cambridge Street between Holcombe and South MacGregor. Based on the topographic information furnished by the client, we understand that the bayou is about 33-feet deep. The bridge piers at the bayou top bank should be founded at a depth of at least 15-feet below the bottom of the ditch or the depth necessary to achieve the required capacity, whichever is more. Side resistance to a depth of 10-feet should be ignored. For the interior bent design, the side resistance in the potential scour depth should be neglected. If the ditch bottom has a concrete lining then the scour depth may be taken as zero. The compressive capacity for drilled piers at the south and north top bank can be calculated using the design charts given in Figures 6 and 7, respectively. These figures were developed based on the soil information obtained at borings GB-3 and GB-5, respectively. The compressive capacity for drilled piers at the interior bents can be calculated using the design charts given in Figures 8 and 9. These figures were developed based on the soil information obtained at borings GB-3 and GB-5, respectively. For conservatism, the lower capacity obtained at any depth from Figures 8 and 9 may be used for the interior bents. Shown in these figures are the design curves for end bearing factor, E (ksf), and side resistance factor, F (kips/ft). The curves include a factor of safety of three for end bearing and two for side resistance. The end bearing factor and side resistance factor at any depth should be multiplied by the corresponding drilled pier tip area and perimeter, respectively, to obtain the drilled pier end bearing and side resistance capacity at that depth. The total drilled pier compressive capacity is obtained by adding the end bearing and side resistance capacities. The uplift capacity may be taken as 0.7 times the allowable side resistance value. Figures 6 through 9 also show design examples calculated for demonstrating the use of the design curves. The lateral bearing capacity may be taken as about 1000 psf. The lateral capacity of drilled piers may be analyzed using non-dimensional method as outlined in the drilled shaft manual (Ref. 12) or by using computer programs such as LPILE plus, etc. The modulus of subgrade reaction, K, may be taken as 40 pci in granular soils and 350 pci in cohesive soils. The final pier depth and diameter should be so selected that the pier is safe against compressive, uplift and lateral loads.

Deep drilled piers may be straight sided shafts without bell bottoms. Groundwater will be encountered below the depth of about 16.5- to 23-feet below the bayou high banks. This depth will vary with the changes in the water level in the bayou. A casing or slurry method of construction will be required for the drilled pier installation.

5.3.1. Abutment Walls

We understand that new abutment walls will be planned for the bayou side at the bridge crossing over Brays Bayou. The abutment walls may consist of either cantilever or gravity retaining walls. The cantilever walls may consist of either sheet piles or drilled pier walls. The drilled pier walls are basically a row of contagious drilled piers installed adjacent to each other. We understand that the bayou is about 33-feet deep.

The abutment retaining walls may be designed for an earth pressure equivalent to a fluid having a density of 102 pcf in clay soils and 82 pcf in sandy soils below the water table. Groundwater was assumed to be at the surface for the earth pressure recommendations given above and the hydrostatic component is included in the earth pressure values. In the event that ground above the abutment walls top is sloping, we recommend that for simplicity, the wedge of soil above the top of the retaining wall be considered as a surcharge pressure exerting an earth pressure on the wall equal to 0.5 times the surcharge pressure. Surcharge loads due to the construction machinary, etc., should be considered in the design. All future surcharge loads including traffic loads likely to act on the wall should also be considered. A typical earth pressure diagram for cantilever wall is given in Figures 1 and 2 of Appedix 6. If the drilled pier walls have edge to edge spacing, then the active pressures given in the figures should be proportionally adjusted upwards.

The anchored abutment walls may be designed for an earth pressure given in Figures 3 and 4 of Appendix 6. Surcharge pressure as discussed for the cantilever wall should be considered in the design. Ground water should be assumed to be at the ground surface and hydrostatic pressure of 62.4 pcf should be considered in design. The effects of surcharge loading on anchored walls can be analyzed by first increasing the height (H) of the pressure diagram by an equivalent height of soils and then using only the portion of the modified pressure diagram below actual ground surface. To compute an equivalent height of soil in feet for this purpose, the surcharge load reduced to load per unit area, in psf, should be divided by 125. The anchors should be located beyond the plane formed by a 4 horizontal to 1 vertical line drawn from the toe of the slope. The anchors should be designed as discussed in the tiebacks section of the report.

The backfill material may consist of on-site soils. In the event that off site soils are brought in then we recommend that they be cohesionless free draining granular soils. The backfill soils should be placed in eight-inch loose lifts and compacted at optimum moisture content to 95 percent of their maximum dry density as determined by the Standard Proctor Compaction Test.

Passive soil resistance for the walls may be taken as 2c (where c is cohesion and may be taken as 1000 psf) in cohesive soils and 70 h in sandy soils (where h is the embedded depth of the wall). The passive resistance of the soils in the potential scour depth of the bayou should be ignored unless the bottom has concrete lining. For cantilever walls, the embedded length calculated should be increased by 30% to include an additional factor of safety in the design. The drilled piers may be straight sided drilled piers and designed for an compressive and uplift capacity as given in Figures 6 through 9. A casing or slurry method of construction will be required for drilled pier installation.

The gravity retaining wall footings may be designed for a safe allowable bearing pressure of 2500 psf. In the event that the retaining wall is supported on drilled footings, then the straight sided drilled piers may be designed for the capacity as obtained from Figures 6 through 9. Casing or slurry method of construction will be required for the drilled footing installation.

The retaining wall designed should be safe against overturning and sliding.

5.3.2. Tiebacks

Tiebacks are used to withstand the lateral earth pressure on the retention system wall. The tiebacks consist of an anchor embedded into the earth, attached to a tendon which is connected to the retention wall. The anchors of the tiebacks should be installed at a distance behind the wall which is outside the plane formed by an 4 horizontal to 1 vertical line drawn from the toe of the slope. The anchors may consist of shafts which are grouted with concrete with or without pressure. The anchors may be straight shafts, be belled or may be concrete blocks. Other type of anchors such as driven piles or helical augers can also be used. A bond stress between anchor and soil of 1000 psf may be used for design purposes. A factor of safety of two should be used in the anchor design. The tendon used for the tiebacks should be protected against corrosion. The tendon design load should not exceed 0.6 times the tendon strength.

5.4 Pavement Design

We understand that a new Cambridge Street roadway will be constructed between Holcombe and South MacGregor. We understand that the new paving will consist of concrete paving. Our design recommendations for new concrete pavement are given in the following sections.

5.4.1 Traffic Information

Considering the location of the street and it's potential use, design ESALs of 10,000,000 were assumed for the new pavement design. In the event that the actual traffic is to be significantly different, we should be contacted. We will then revise our recommendations based on the actual anticipated traffic information.

5.4.2 Subgrade Preparation

The surficial soils at the proposed project area consisted of sandy clay (CL). These soils should provide an acceptable base for pavement construction when properly prepared as following:

- Strip existing ground to remove organics and other unsuitable materials. Proof roll the subgrade to detect any wet, soft, or pumping areas. Treat these areas with drying or stabilizing agents, as necessary, or remove and replace them with a suitable fill material. Lime stabilization of the subgrade with 5 % lime by dry weight extending to a depth of eight inches is recommended. This percentage should be confirmed by a lime series test at the time of construction.
- Good surface drainage should be provided away from the edges of paved areas to minimize lateral moisture transmission into the subgrade.
- Compact the subgrade to a minimum of ninety-five (95) percent of its maximum dry density at an moisture content within a range of plus or minus 2 percent of optimum, as determined by the Standard Proctor Compaction Test (ASTM D 698).

5.4.3 Subgrade Support

Our field exploration indicated that the subgrade soils below the existing pavement consisted of stiff to very stiff sandy clays. The California Bearing Ratio (CBR) tests indicated the surface soils to have CBR values ranging from 4 to 5. Additional information can be obtained from the Table 3. For this project, we recommend a design CBR value of 4 and resilient modulus M_R. of 6000 psi for use in the pavement design for this project.

5.4.4 Concrete Pavement

The concrete pavement was designed based on the AASHTO procedure. The following design parameters were used in the concrete pavement design for the proposed pavement.

Reliability, R : 95

Overall Standard Deviation, So : 0.30

Load Transfer Coefficient, J : 3.2

Drainage Coefficient, Cd : 1.0

Design Serviceability Loss, ΔPSI : 2.5

Initial Serviceability, po = 4.5, Terminal Serviceability pt = 2.0

Loss of Support : 1.0

Traffic : 10,000,000 ESAL (18-Kips) for design life of 20 years

Concrete Modulus of Rupture : 600 psi

Modulus of Elasticity of Concrete, Ec = 3.6×10^6 psi

Effective Modulus of Subgrade Reaction, k = 120 pci

Based on the above design parameters, the recommended concrete pavement section thickness is 10.0-inches for design ESAL of 10,000,000. The design chart for concrete pavement is shown in Appendix 7. The top eight-inches under the pavement should be lime-stabilized using 5% lime by dry weight. The lime stabilization should be in accordance with City of Houston Standard Specirfication, Section 02336. It should be noted that the pavement thickness will change as a function of traffic. If the actual traffic is going to be significantly different from that assumed, then we should be contacted for revised recommendation based on the actual traffic.

Concrete should meet the City of Houston standard requirements and/or the requirements of the AASHTO "Guide Specifications for Highway Construction and the Structural Specifications for Transportation Materials".

Longitudinal joints for concrete pavement are generally designed at distances between 40- to 80-feet. A longitudinal spacing of about 80-feet may be used.

5.4.5 Reinforcement Design

The reinforcement design may be in accordance with City of Houston standard specification shown in Drawing No. 02751-01. For a 10-inch thick concrete pavement with longitudinal spacing of 80-feet, pavement width of 24-feet, 28 day concrete compressive strength of 3000 psi and grade 60 steel, the longitudinal spacing may be 12.75-inches for No. 4 bars and 18.25-inches for No. 5 bars. The transverse spacing may be 36-inches for No. 4 and No. 5 bars. The minimum lap lengths should be 22-inches for No. 4 bars and 27-inches for No. 5 bars. For a different pavement width then that given above, the reinforcement details should be taken from the City of Houston Drawing No. 02751-01.

6.0 CONSTRUCTION REVIEW

6.1 Quality Control

Associated Testing Laboratories, Inc. (ATL) recommends implementation of a comprehensive quality control program under the supervision of a Professional Engineer. 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 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.

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.

6.2 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.

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.

7.0 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 features at the site not within the scope of this investigation, ATL is not responsible for any problems caused by these features.

8.0 REFERENCES

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6. Annual Book of ASTM Standards for Soils and Rock; Building Stones.
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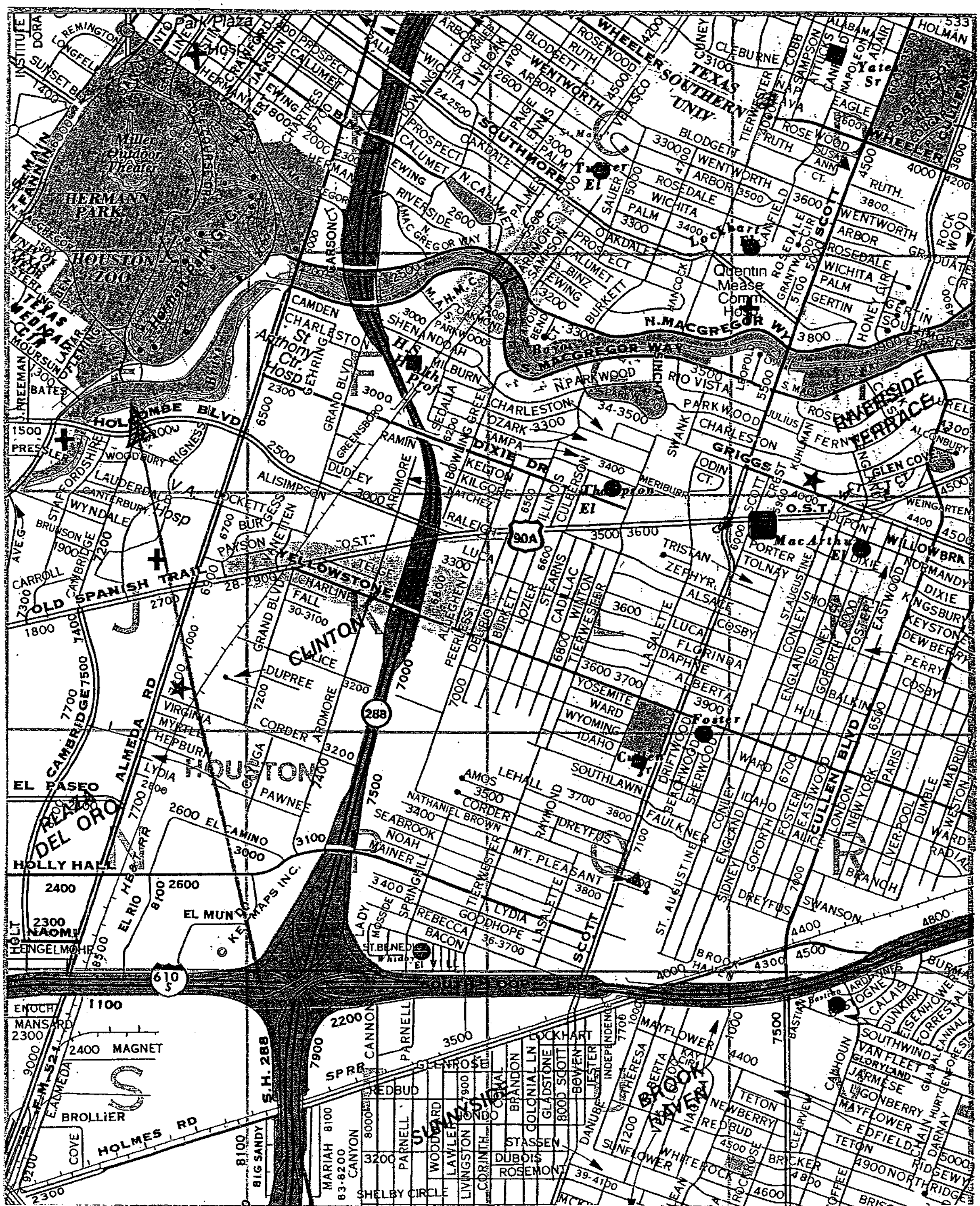
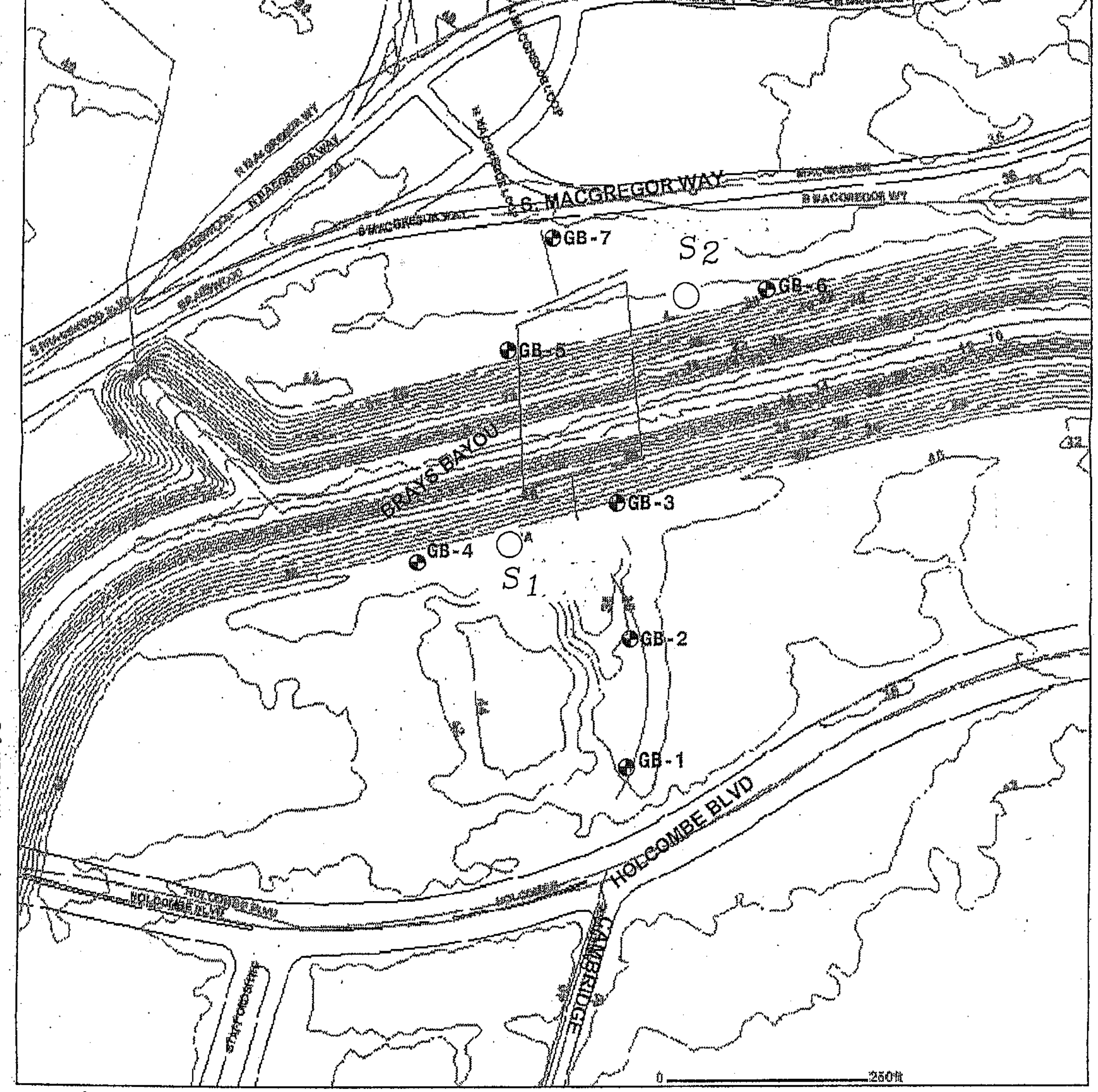
9. Groundwater Quality in Texas; Texas Natural Resources Conservation Commission.
10. 29 CFR PART 1926.
11. Greater Houston Waste Water Program Guidelines.
12. Yaung H. Huang (1993), APavement Analysis and Design,≅ Prentice Hall.
13. E.J. Yoder and M.W. Witczak (1975), Principles of Pavement Design,≅ John Wiley & Sons, Inc., Second Edition
14. Design of Pavement Structure; AASHTO 1993.

FIGURES

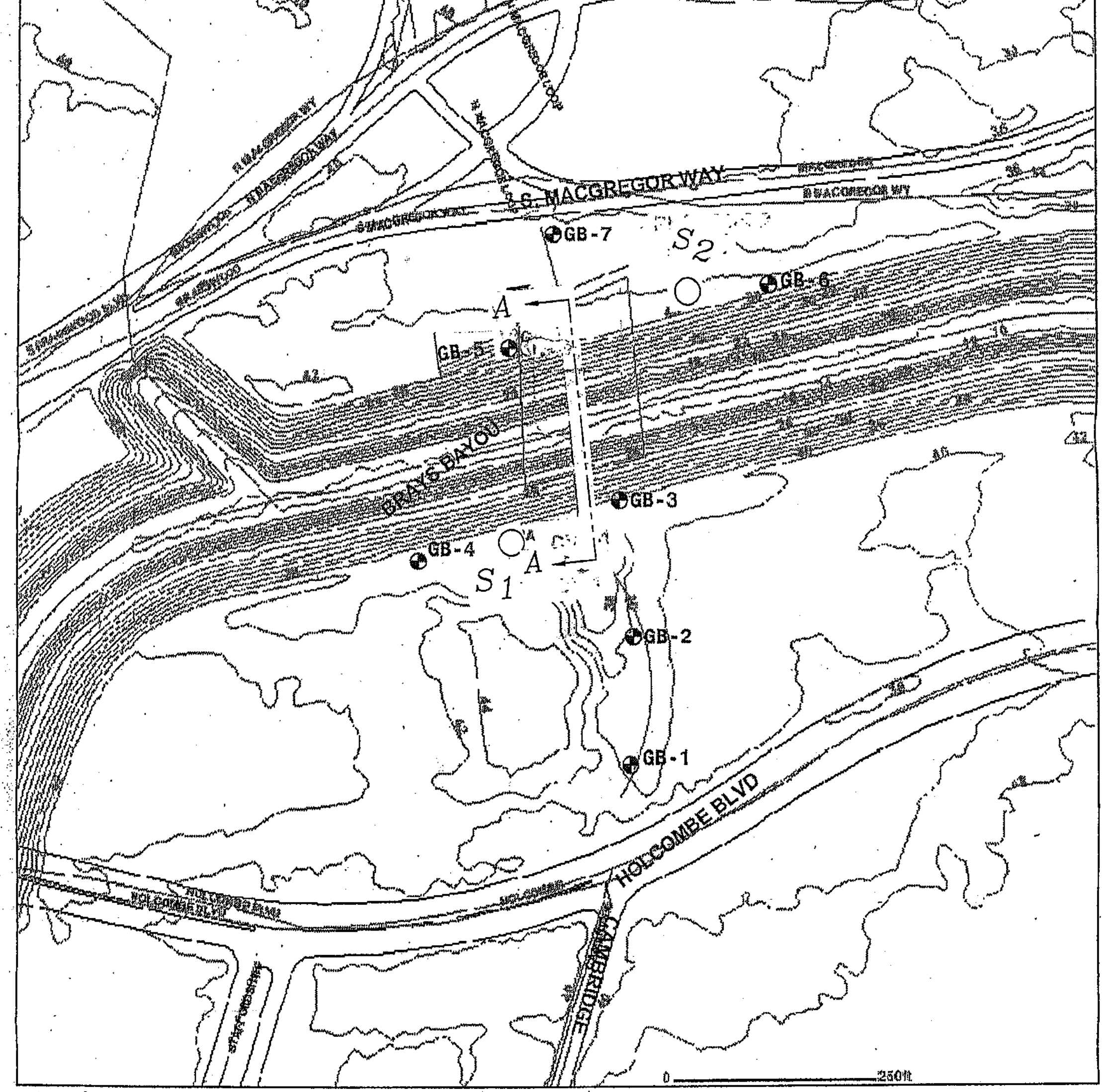


SITE LOCATION

SITE VICINITY MAP	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748
CAMBRIDGE STREET BETWEEN HOLCOMB AND SOUTH MACGREGOR HOUSTON, TEXAS	SCALE : N.T.S.
	PROJECT NO. G04-504 FIGURE. 1



SITE PLAN WITH BORING LOCATIONS	Associated Testing Laboratories, Inc. 3143 Yellowstone Blvd. Houston, Texas Tel: (713) 748-3717 Fax: (713) 748-3748	
CAMBRIDGE STREET BETWEEN HOLCOMB AND SOUTH MACGREGOR HOUSTON, TEXAS	SCALE : N.T.S.	
	PROJECT NO. G04-504	FIGURE. 2



PLOT SHOWING SECTION A-A
ANALYZED FOR SLOPE STABILITY

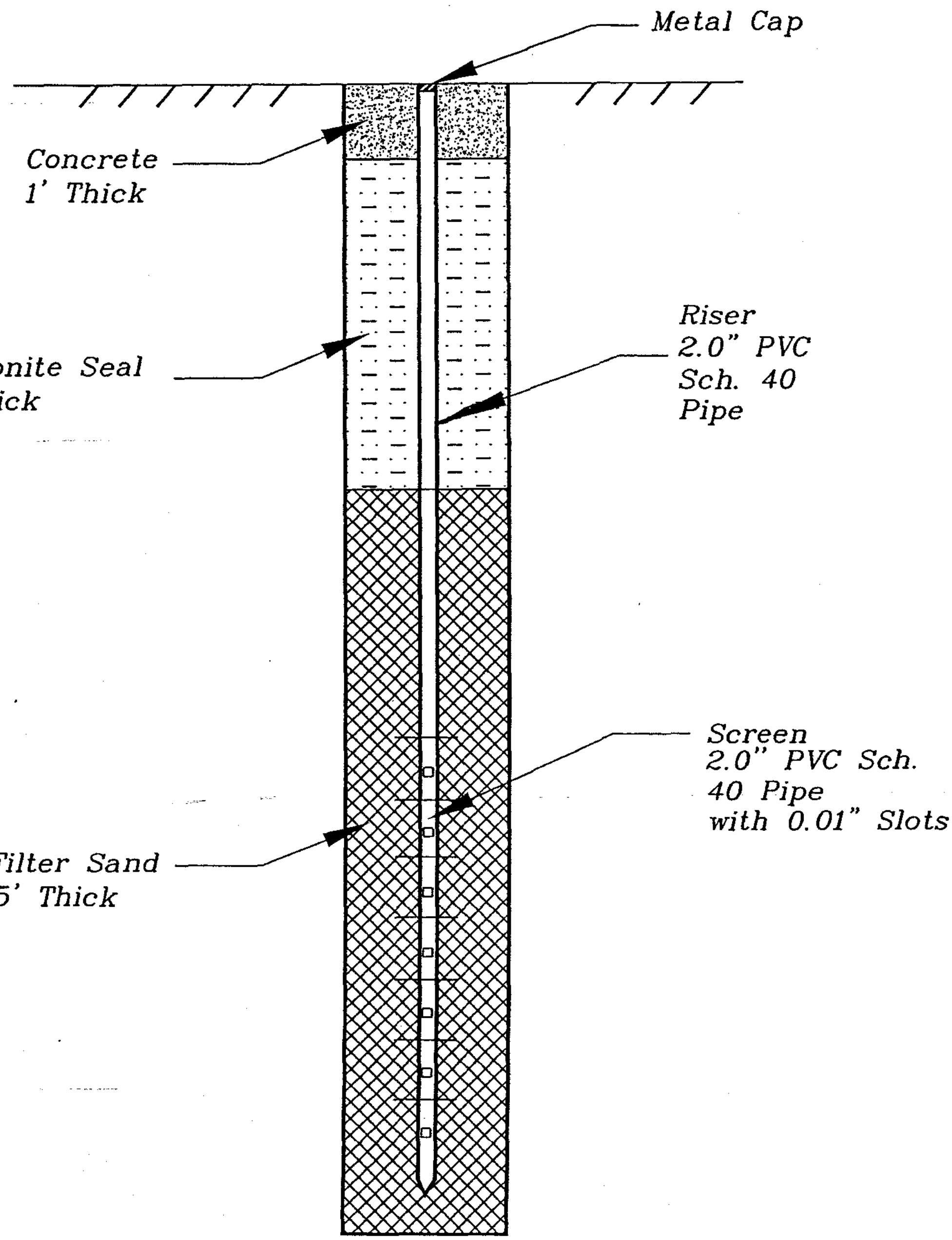
Associated Testing Laboratories, Inc.
3143 Yellowstone Blvd. Houston, Texas
Tel: (713) 748-3717 Fax: (713) 748-3748

CAMBRIDGE STREET BETWEEN HOLCOMB AND
SOUTH MACGREGOR
HOUSTON, TEXAS

SCALE : N.T.S.

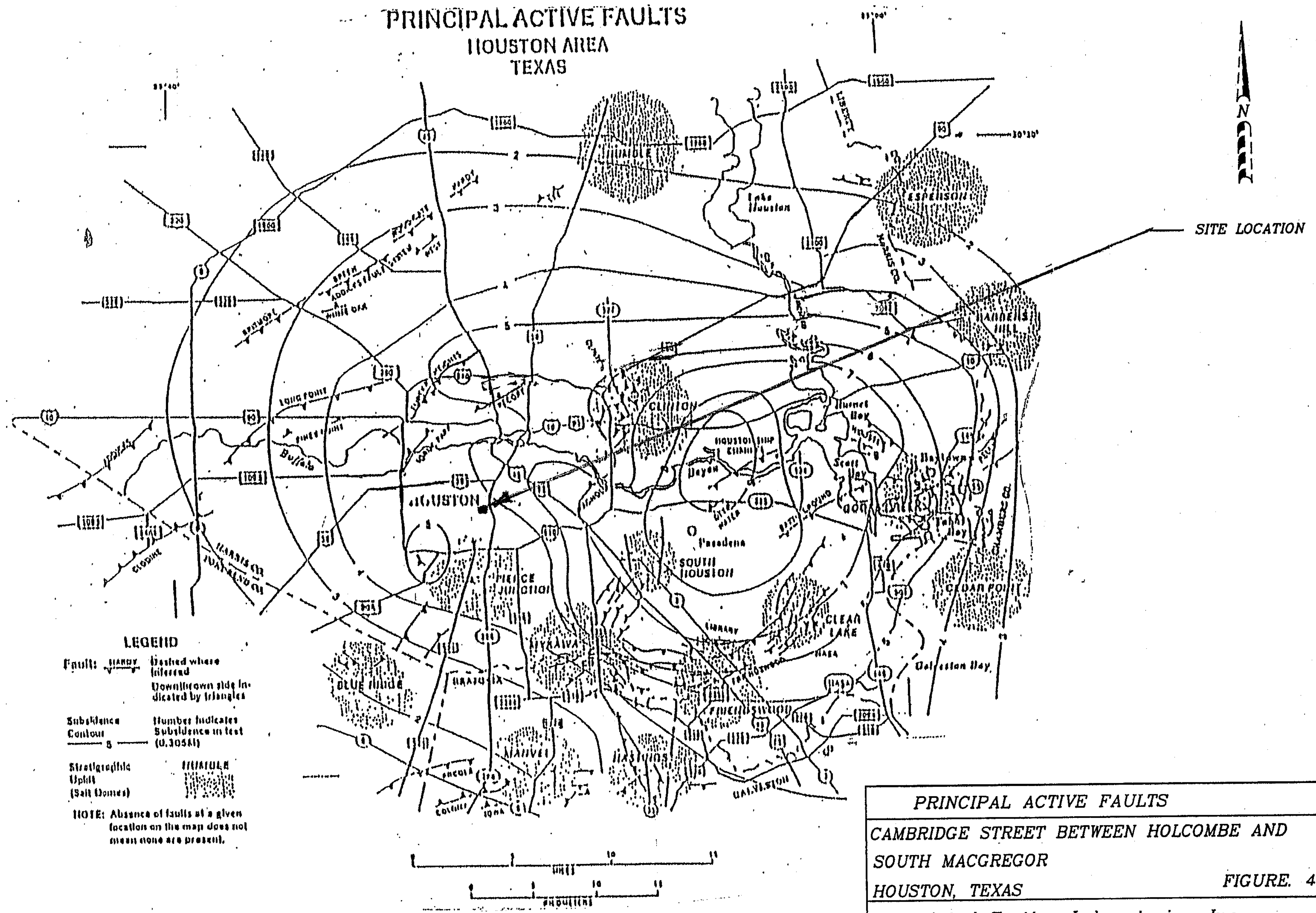
PROJECT NO. G04-504

FIGURE. 2A



PIEZOMETERS AT GB-1 & GB-7

FIG.3 G.W. MONITORING WELL
CAMBRIDGE STREET BETWEEN HOLCOMBE AND SOUTH MACGREGOR
ATL Job No. G04-504 FIGURE. 3
Associated Testing Laboratories, Inc.



PRINCIPAL ACTIVE FAULTS

CAMBRIDGE STREET BETWEEN HOLCOMBE AND SOUTH MACGREGOR

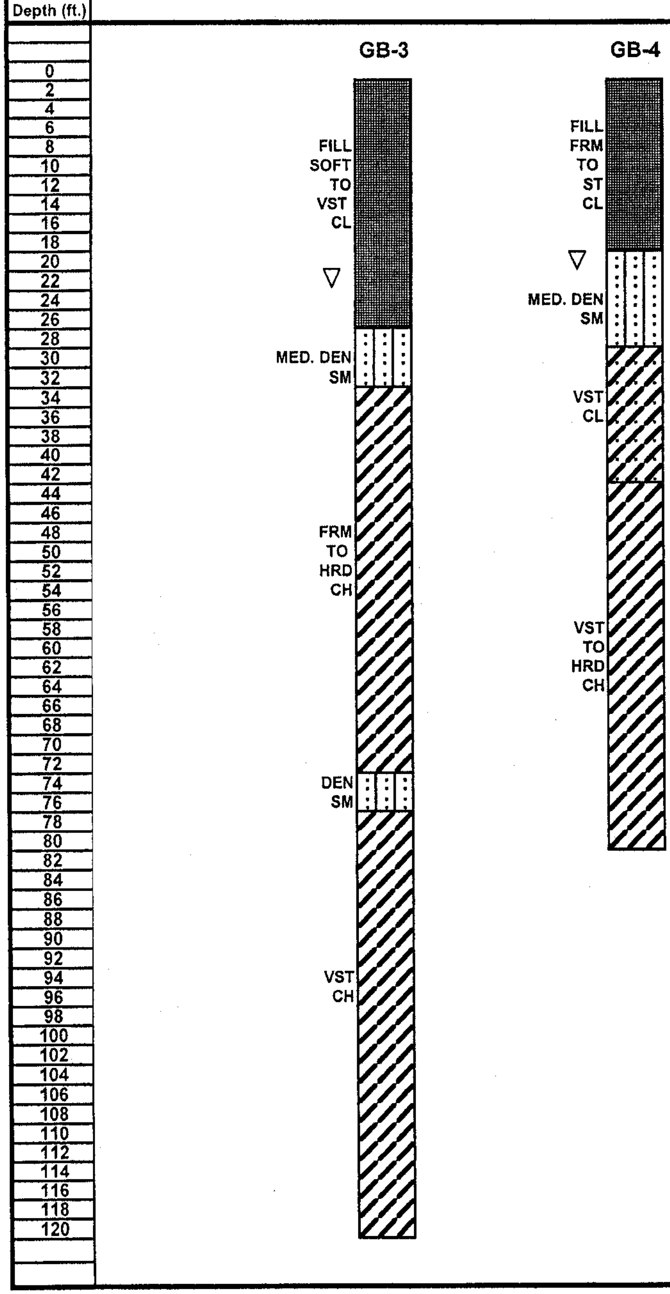
HOUSTON, TEXAS

FIGURE. 4

Associated Testing Laboratories, Inc.

ASSOCIATED TESTING LAB, INC.

Project Name: CAMBRIDGE STREET BETWEEN HOLCOMBE AND S. MACGREGOR — PROJECT NO. G04-504



KEY

- CH- Clay
- CL- Sandy Clay
- ST- Stiff
- VST- Very Stiff
- FRM- Firm
- HRD- Hard
- MED.DEN- Medium Dense
- DEN- Dense
- V.DEN- Very Dense

PROFILE ALONG SOUTH SIDE OF BRAYS BAYOU

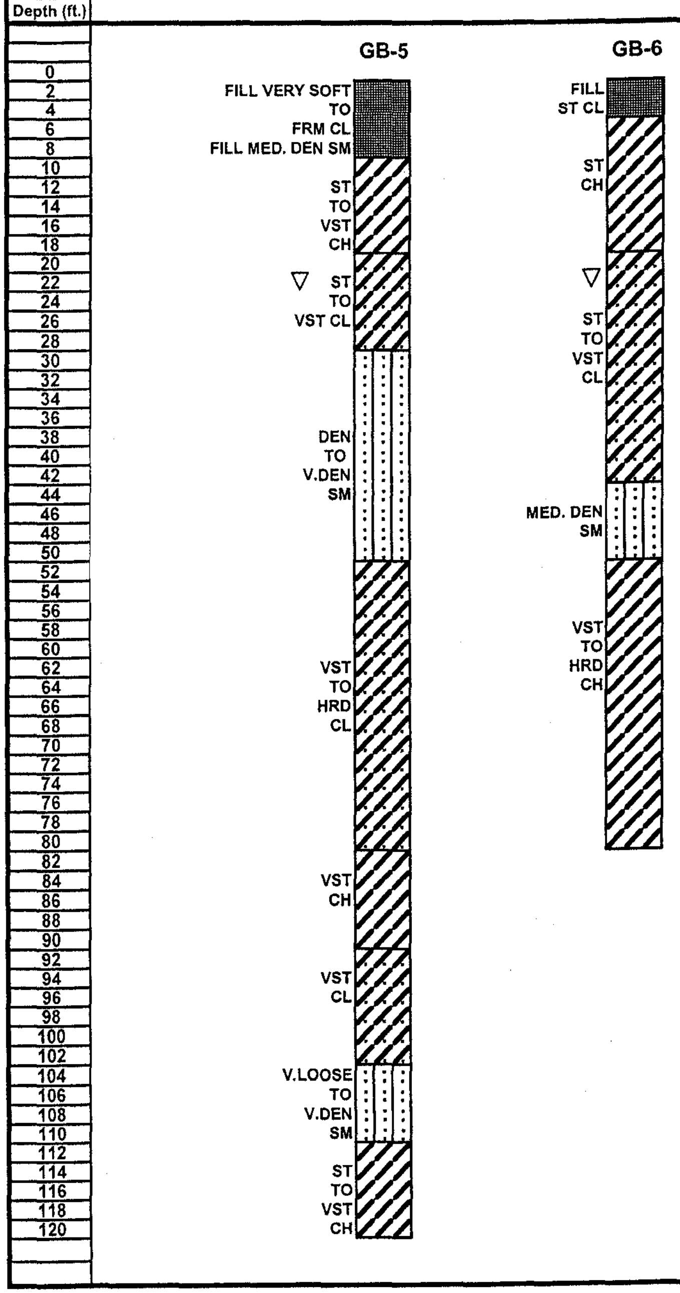
▽ Water First Noticed
▽ Depth to Water at Completion
▼ pz water level

SCALE:
Vertical Scale: 1" = 20'
Horizontal Scale: N.T.S.

Figure 5A

ASSOCIATED TESTING LAB, INC.

Project Name: CAMBRIDGE STREET BETWEEN HOLCOMBE AND S. MACGREGOR — PROJECT NO. G04-504



KEY
CH- Clay
CL- Sandy Clay
ST- Stiff
VST- Very Stiff
FRM- Firm
HRD- Hard
MED.DEN- Medium Dense
DEN- Dense
V.DEN- Very Dense

PROFILE ALONG NORTH SIDE OF BRAYS BAYOU

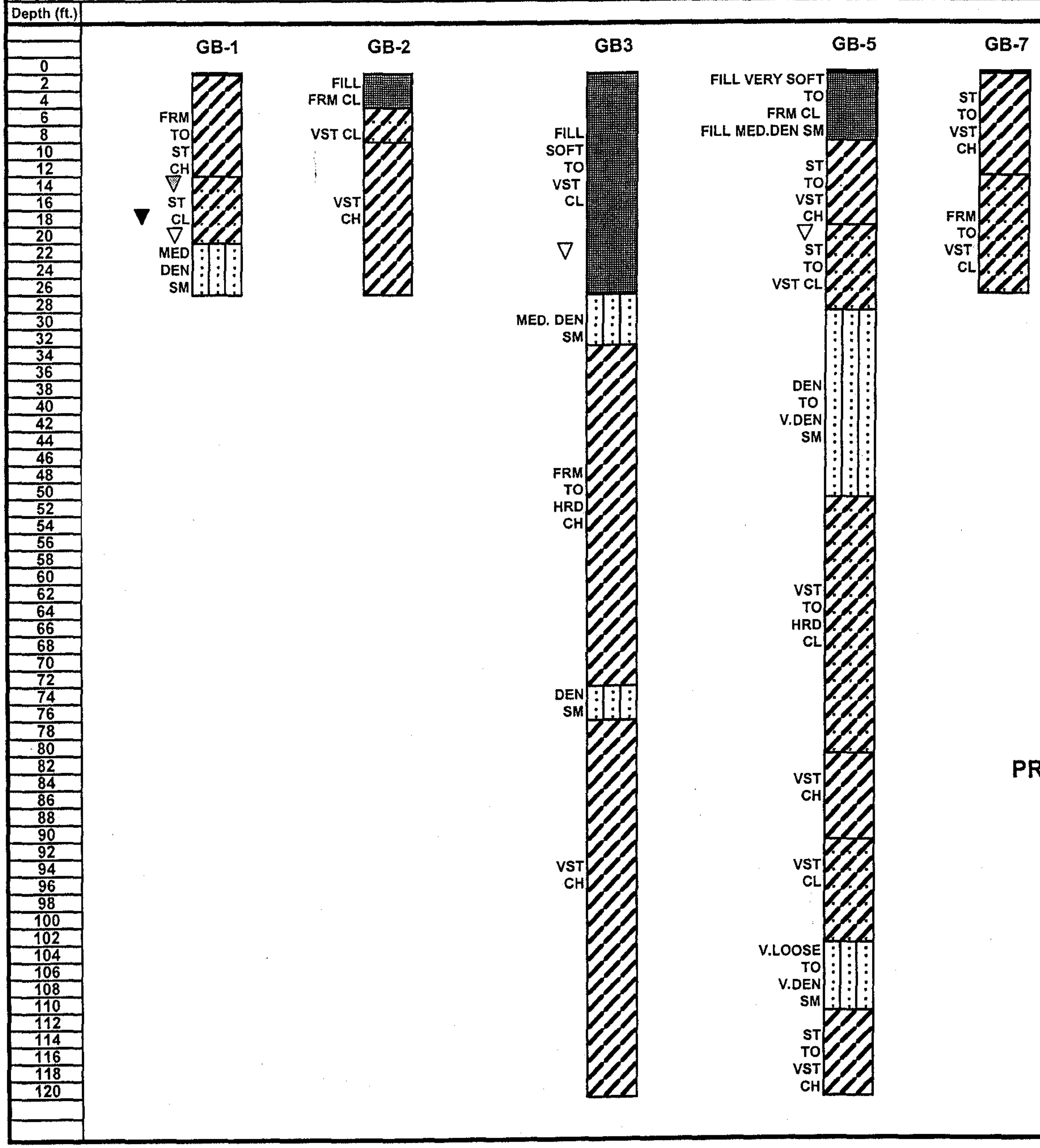
▽ Water First Noticed
▼ Depth to Water at Completion
▼ pz water level

SCALE:
Vertical Scale: 1" = 20'
Horizontal Scale: N.T.S.

Figure 5B

ASSOCIATED TESTING LAB, INC.

Project Name: CAMBRIDGE STREET BETWEEN HOLCOMBE AND S. MACGREGOR | PROJECT NO. G04-504



KEY
CH- Clay
CL- Sandy Clay
ST- Stiff
VST- Very Stiff
FRM- Firm
HRD- Hard
V.LOOSE- Very Loose
MED.DEN- Medium Dense
DEN- Dense
V.DEN- Very Dense

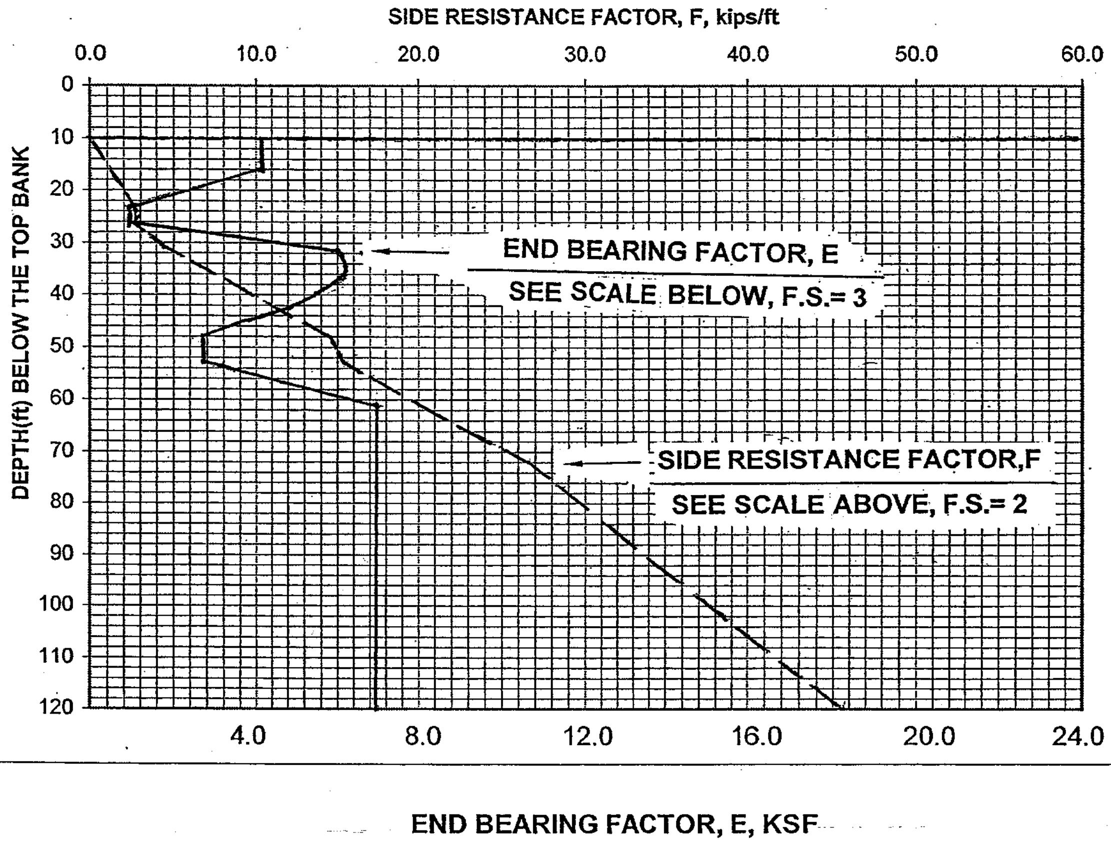
PROFILE ALONG PROPOSED CAMBRIDGE STREET

- ▽ Water First Noticed
- ▼ (shaded) Depth to Water at Completion
- ▼ pz water level

SCALE:
Vertical Scale: 1" = 20'
Horizontal Scale: N.T.S.

Figure 5C

FROM BORING GB-3AT TOP BANK
(FOR DRILLED PIERS)



DESIGN EXAMPLE (FROM BORING GB-3)

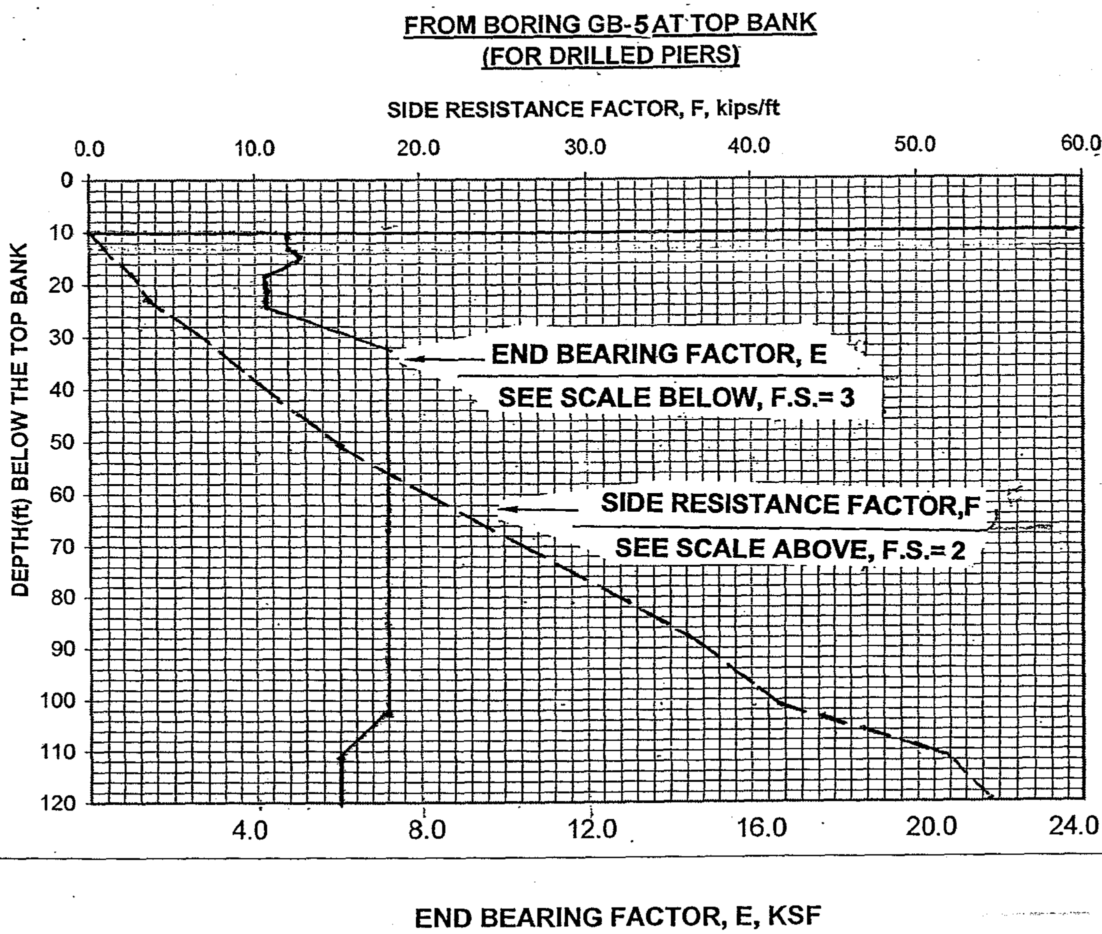
AT Top Bank

Say 48-inch diameter Drilled Pier
Area of 48-inch diameter pier = 12.56 sq. ft
Perimeter of 48-inch diameter Pier = 12.56 ft

COMPRESSIVE CAPACITY
(for drilled piers at 60-feet depth from high bank)
Qc = 12.56 (E) +12.56 (F)
= 12.56 (7.0) + 12.56 (19.5)
= 332.84 kips

TENSILE CAPACITY:
Qt = 0.7 * 12.56 (F)
= 0.7 * 12.56 (19.5)
= 171.45 kips

Project No. G04-504



DESIGN EXAMPLE (FROM BORING GB-5)

AT Top Bank

Say 48-inch diameter Drilled Pier
Area of 48-inch diameter pier = 12.56 sq. ft
Perimeter of 48-inch diameter Pier = 12.56 ft

COMPRESSIVE CAPACITY
(for drilled piers at 60-feet depth from high bank)
Qc = 12.56 (E) +12.56 (F)
= 12.56 (7.6) + 12.56 (20.5)
= 352.94 kips

TENSILE CAPACITY:
Qt = 0.7 * 12.56 (F)
= 0.7 * 12.56 (20.5)
= 180.24 kips

Project No. G04-504