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# GEOTECHNICAL ENGINEERING REPORT

## CITY OF HOUSTON - EAST WATER PROGRAM

## PACKAGE 5B

REPORT NUMBER

275-85E

REPORTED TO

LOCKWOOD, ANDREWS, & NEWNAM INC.
HOUSTON, TEXAS

MAY 1986

# TABLE OF CONTENTS

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**(Continued)**

**The City of Houston East Water Program**
**Addendum to Geotechnical Engineering Report**
**Package 5B**
**Date: August 6, 1986**

I. Clarification Required for Engineering

A. Comment: For clarification, the working pressure is 96 psi, the test pressure is 175 psi and the maximum allowable surge pressure is 210 psi. Calculations for joint restraint, etc., should be based on 210 psi.

Response: Required thrust restraint for known unbalanced hydrostatic forces associated with the Segment 5B alignment have been recalculated using a maximum allowable surge pressure of 210 psi. A summary of the new thrust restraint requirements is provided on the attached Table 1.

B. Comment: No information has been provided on the friction coefficients to be used for joint restraint calculations. What value was used in the calculations? Can these coefficients be determined from information provided in the report? How is this affected by saturated soil conditions?

Response: Restrained joint calculations were made assuming a relatively clean sand would be used as bedding and backfill around the pipe and that the trench would be only as wide as required for pipeline construction. The report contained recommendations with respect to placement of the sand backfill and bedding, including that the sand backfill extend from the bedding to slightly above the top of the pipe. Compacted clay was recommended as trench backfill above the sand.

An angle of friction of 35° was conservatively assumed for the clean sand recommended for pipe bedding and backfill. In the restrained joint calculations, this friction angle was used in the calculation of the soil/pipe frictional resistance. However, for the assumption of saturated soil conditions, it was reduced by 20 percent and, for the recommended soil placement conditions and likely depth of cover, the calculated frictional resistance was reduced an additional 25 percent. The undrained strength parameters of the in situ soil were used in the restrained joint calculations for determining the resistance due to passive pressure. The undrained strength parameters are contained in Table 1 of the report.

C. Comment: Verify the required lengths for joint restraint and explain how they were calculated. Isolation valves are anticipated to be spaced every 5,000 feet. What impact will this have on the joint restraint calculations?

Response: Where restrained joints are used to provide reaction to unbalanced hydrostatic forces, resistance is developed through lateral passive resistance of the soil in addition to the frictional resistance between the soil and pipe. Typical calculations showing the methodology and assumptions used in calculating the required length of restraint to resist thrust forces are provided in the attached Design Calculation Package. Similar calculations are also provided for sizing thrust blocks to resist these unbalanced hydrostatic forces. The basis for these calculations was R. J. Carlsen's paper, entitled "Thrust Restraint for Underground Piping Systems" (Cast Iron News, Fall 1975). In situations where passive resistance of the soil cannot be developed, such as in resisting axial forces applied over short lengths, the use of restrained joints would not be feasible.

With respect to the isolation valves, it is our understanding that the valves will be operated from the ground surface and used to regulate flow within the pipeline. The valves should be connected to the pipeline with restrained joints to prevent separation of the pipe from the valves due to unbalanced hydrostatic forces. The length of pipe requiring restrained joints on either side of a valve should be calculated as described above when the valve operating procedures are designed and the pressures which develop from valve operation are determined. Additional restraint will be necessary if the pipeline length is less than the required length calculated using the attached Design Calculation Package. For unbalanced hydrostatic forces acting along the longitudinal axis of the pipe, a passive pressure ($P_p$) of 0 and bend coefficient (K) of 1.0 should be used in the equation.

D. Comment: Need more specific allowable bearing capacity for sand and clays (p. 19) to design the thrust blocks.

Response: Since the depth and angle of deflection is not known for any portion of the Segment 5B pipeline, only general comments can be made at this time regarding allowable soil bearing capacity with respect to providing reaction to thrust forces due to bends in the pipeline. These comments were included in the section on Thrust Restraint in the geotechnical engineering report. Specifically, if a vertical bend produces a resultant force which is downward with respect to a horizontal plane, thrust blocks may be designed with respect to bearing capacity by procedures developed for inclined footings. Parameters required for this design are:

- o Properties of the in situ soil at and below the bearing surface of the thrust block;

- o Depth to the bearing surface; and

- o Angle and magnitude of the resultant force.

If a vertical bend produces a resultant force which is upward with respect to a horizontal plane, thrust blocks should be designed to resist the resultant force by dead weight only. The dead weight of the thrust block may include the weight of the concrete and the weight of the soil prism above the block. The reaction capacity of thrust blocks designed to resist resultant forces which are inclined upward form the horizontal may be enhanced by the use of driven friction piling, straight shafts, or drilled, underreamed shafts.

E. Comment: Figure 4 shows over-excavation of the pipeline trench of one to 2 feet, while Figure 7 seems to show it should be limited to 6 inches. Which is correct? If excavation does occur beyond the acceptable limit, what must the contractor do to remedy the situation?

Response: Recommended bedding and backfill criteria for concrete embedded cylinder pipe (AWWA Standard C-301) are presented in Figure 7 of the Package 5B Report submitted to LAN. Figure 4 of this report is a generalized diagram showing typical dimensions for pipeline excavations and placement. Referring to Figure 7, the intent of this diagram was to specify that a minimum of 14 inches of clean sand bedding should extend, at minimum, from 6 inches below to 8 inches above the pipe invert. If over-excavation of the trench occurs during pipeline construction, additional sand bedding in excess of the required 6 inch thickness should be placed below the pipe to restore the trench bottom grade. The additional sand bedding, if required, should be placed in maximum 6 inch lifts and hand tamped.

F. Comment: All geotechnical investigations and analyses have been done specifically for concrete cylinder pipe. However, steel pipe will be bid as an alternate, and, therefore, information that relates specifically to steel (i.e., bedding and backfill, joint restraint, etc.) is needed.

Response: According to information provided by LAN, the steel pipe alternate for Segment 5B will have a wall thickness of 3/8 to 1/2 inch and will be continuously welded in the field. We have reevaluated each of the various design criteria for steel pipe and feel only two changes are necessary in our previous recommendations, namely: slightly reduced trench width/depth (approximately 12 inches less) due to the reduction in the outside diameter of the pipe, and lack of the need for thrust restraint due to the method of pipeline fabrication (continuous welds). However, the restrained joint calculations should be based on a coefficient of soil/pipe friction less than that assumed for the concrete pipe calculations because the soil/pipe interface is smoother. Other than these changes, the recommendations given previously for concrete embedded cylinder pipe may be applied to the steel pipe. The required length of steel pipe to resist thrust forces should, however, be verified using the information provided in response I.C.

G. Comment: On Table 1 of the report, what are $s_u$, $\phi$, $c'$ and $\phi'$? Why is $c'$ equal to zero for all soil types?

Response: Referring to the Design Soil Parameters shown in Table 1 of the report, $s_u$ and $\phi$ are undrained (i.e., short-term) shear strength parameters whereas $c'$ and $\phi'$ are drained (i.e., long-term) shear strength parameters. Specifically, $s_u$ and $c'$ are related to soil cohesion and are strength parameters normally associated with clays. The parameters $\phi$ and $\phi'$ are related to the angle of internal friction of soil particles and are strength parameters normally associated with granular soils. No laboratory testing was specifically conducted to evaluate drained shear strength parameters of clay soils. The values given for $\phi'$ in Table 1 are based on our experience with similar soils (the Beaumont Formation) in this area, in addition to review of available literature. In design calculations, c' is often conservatively assumed to be zero for clay soils in the Beaumont Formation. For most granular soils, the value of c' is zero.

H. Comment: Need specific recommendations for tunneling or jacking underneath the railroad.

Response: Segment 5B includes one reach of pipeline which crosses existing Houston Belt and Terminal Railroad Tracks. The crossing may be achieved by bridging or by installing the pipeline underneath the tracks. Potential methods for installing the 84-inch I.D. pipeline underneath the railroad tracks include open cut-and-cover, tunneling, and pipe jacking. Other methods commonly used to install pipeline under existing surface structures and facilities, such as horizontal augering and casing, are not considered appropriate for the relatively large 84-inch I.D. pipeline.

Open cut-and-cover installation of the pipeline is probably the most economical method of installation, but it requires that the railroad tracks be taken out of service and removed for the period of time required for pipeline installation. If the pipeline is installed by open cut-and-cover methods, backfilling of the trench should be controlled in the same manner recommended for backfilling a trench at a street crossing to minimize the potential for subsequent surface settlement.

Tunneling procedures appropriate for installation of a pipeline under existing railroad tracks include hand excavation, mechanical excavation and excavation using a tunneling machine. The procedure used will depend on the subsurface soil and groundwater conditions. Each of the procedures requires construction of a starting and a receiving shaft on opposite sides of the railroad track bed (and possibly outside of the railroad right-of-way), with the tunnel excavated between the shafts. A primary liner is installed during tunneling to support the excavation. If access to the exterior of the pipeline is required after its installation, a final liner will probably be placed or cast against the primary liner and the pipeline will be emplaced inside the final liner. Otherwise, the pipeline will be emplaced inside the primary liner, and the annular space between the primary liner and the pipeline filled with cement grout. A pipe-in-tunnel installation will probably require a 9.5 to 14 foot diameter tunnel bore, depending on the lining and subsequent inspection requirements.

The "pipe jacking" method of pipeline installation is similar to tunneling except the excavation is accomplished by tunnel boring machine (TBM) and the pipe is jacked into the tunnel excavation, or bore, immediately behind the TBM. For pipe jacking, the pipe provides thrust reaction for advance of the TBM. As with the pipe-in-tunnel method, shafts are required on either side of the track bed.

For pipe jacking, the TBM typically cuts an excavation diameter that is several inches larger than the outside diameter of the remainder of the TBM or the pipe. The overcut reduces friction between the pipe and soil during advance. To reduce friction on long drives and to help prevent movement of the soil into the resulting annular space, it is common to fill the annular space with bentonite slurry under pressure. When the drive is complete and the pipeline is in its final alignment, the annular space is filled with cement grout.

The stratigraphy at the Segment 5B railroad crossing, as represented by borings 5B-3 and 5B-4, appears to consist of up to 15 ft of high plasticity clay underlain by approximately 10 ft of low plasticity sandy clay and low plasticity silty clay (east side) and high plasticity clay to the terminal depth of the borings. To reduce the potential surface settlements, it is recommended that the pipeline be installed (whether by tunneling or jacking) with at least two excavation diameters of cover for the crown of the excavation, with at least 1/2 excavation diameter being clay. Under these recommendations, the invert of the excavation for the pipeline may be as deep as 30 to 40 feet. Tunneling or pipe jacking may be attempted at shallower depths with the risk of increased surface settlements.

The groundwater table through the length of the tunneling or jacking operation should be verified prior to construction. The contractor should be prepared to dewater in the event that lenses of water-bearing granular soils are encountered along the alignment of the pipeline railroad crossing.

Loading on the tunnel liner or pipe will result from three components which are additive and should be calculated as follows:

(1) The total weight of the ground from the surface to the springline;

(2) Hydrostatic pressure calculated from the ground surface to the springline (unless water table measurements indicate a smaller hydrostatic pressure is appropriate); and

(3) The distributed load of a single train passing over the tunnel/pipe as provided in the attached Table 3 which supersedes Table 3 of the report.

Rail car loading stress overlap resulting from trains on both tracks may be calculated by distributing the surface loads downward on 2(V) to 1(H).

Depending on the tunneling and lining technique used, from 1/2 inch to 3 inches, or more, of settlement above the tunnel centerline should be anticipated. A settlement trough may be expected to extend from 30 to 50 feet on either side of the tunnel centerline (that is, perpendicular to the tunnel alignment). If anticipated settlements are not tolerable, pre-excavation ground modification may be required.

I. Comment: Figure 3 is illegible.

Response: five (5) improved copies of Figure 3 have been transmitted to LAN under separate cover.

II. Clarification Required for Construction

A. Comment: What is the recommended density of compaction for backfill outside pavement areas (p. 15)?

Response: Compaction of backfill outside paved areas, such as where the pipeline is situated beneath a median or within the right-of-way on either side of a road, is less critical than beneath paved areas subject to traffic loads. We recommend the backfill in these trenches be placed in maximum 12 inch loose lifts and compacted using hand compaction equipment or the tires or tracks of available construction equipment. The backfill should be free of rocks, tree stumps, broken pavement, or other unyielding solid objects. The moisture content of the material during compaction should be such that the compaction equipment produces a dense, stable mass free of soft spots or depression areas. This compaction procedure should produce a backfill density equal to about 80-

85 percent of Standard Proctor Density. The surface of the backfill should be crowned to promote drainage and compensate for any settlement of the compacted backfill.

B. Comment: The recommendation on dewatering calls for lowering the water table approximately 2 feet below the bottom of the excavation. Vacuum well points are recommended, but are limited to 15 to 18 feet maximum uplift. What is recommended for deeper than 18 feet?

Response: In relatively shallow excavations (less than 10 feet deep), groundwater will generally be under low head and can be handled by sump collection for pumped disposal. Where excavation depth is on the order of 10 to 15 feet, groundwater will probably be under sufficient head to require dewatering. The common technique employed in this area for temporary dewatering is the use of vacuum wellpoints. Typically, wellpoints are installed in a row paralleling the trench, screened at a depth such that the water table will be drawn down at least 2 feet below the bottom of the excavation, and situated as close to the edge of the excavation as construction activities will permit. Experience in this area has shown vacuum wellpoints to be generally ineffective below about 15 to 18 feet, unless the wellpoint header is placed on a shallow bench within the excavation. Where vacuum wellpoints are used to dewater trench excavations, the wellpoints are often "walked" along the alignment; that is, when backfill is placed, the wellpoints along the backfilled portion of the alignment are deactivated and advanced to extend dewatering ahead of the excavation.

For excavations in excess of about 15 to 18 feet, wells with submersible pumps, either atmospheric or with vacuum assist, or eductor (jet ejector) wells, are generally required. Submersible pumps with vacuum assist have proved successful on some projects because of the resulting increase in gradient between the groundwater table and the screen.

C. Comment: What does the assumed 500 psf construction surcharge account for (p. 12, Figures 5 and 6 of the report)? Is this construction traffic only? Should excavation spoil be added to this number?

Response: The 500 psf construction surcharge mentioned on page 12 and shown in Figures 5 and 6 of the Package 5B Report was an assumed "typical" value. The actual construction surcharge will depend on the contractors' operations. The 500 psf surcharge is approximately equal to the surcharge which would be produced by placing 4 feet of excavation spoil adjacent to the open trench. With respect to equipment loads only, this value is probably high. In any case, we recommend this surcharge load be reevaluated prior to the design of an internally braced shoring system using more precise information provided by the contractor regarding his planned operations.

D. Comment: Some attempt has been made to identify the locations of possible wet sand conditions (p. 14 of the report). This is vague and general. These locations need to be tied down more specifically.

Response: The granular soils which require dewatering are likely to be encountered along the eastern reach of the Segment 5B alignment, between approximate Stations 60+00 and 92+50. The granular soils within this reach are predominantly clayey sands and clayey silts. We recommend that the water table be lowered at least 2 feet below the bottom of the trench excavation between Stations 60+00 and 92+50 of the Segment 5B alignment.

III. Additional Comment

Continuing internal review of the Package 5B Report indicated some discrepancies between the boring logs and the graphic logs shown on the Generalized Soil Profile, Figure C-1 of the report. Hence, Figure C-1 has been modified. A copy of the modified figure is attached to this addendum.

## TABLE 1
## THRUST RESTRAINT (C-301 PIPE)
## PACKAGE 5B

| Description | Thrust Force and Direction | Size of Thrust Block (1) | Length of Pipe Requiring Restrained Joints (2) |
|---|---|---|---|
| Blind Flange at the West End of 84" Line near Everton Street | 1,163,700 Pounds along longitudinal axis of pipe | 250 sq. ft. | N/A |
| $30^{o}$ Bend in 84" Line near Engle Street | 602,400 Pounds at $105^{o}$ angle to either reach of pipeline | 125 sq. ft. | 37 ft. either side of bend |

(1) Assumes block bears on soil between 10 and 20 ft. depths.

(2) Assumes use of restrained joints only at bends in pipeline.

# TABLE 3
## (Revised)
## PIPELINE LOADS DUE TO SINGLE RAILROAD AT GROUND SURFACE

| Depth of Cover, ft | Vertical Pressure* ksf | Length of Tunnel Affected by Load, ft |
|---|---|---|
| 1 | 1.36 | 8 |
| 3 | 0.99 | 10 |
| 5 | 0.75 | 12 |
| 7 | 0.59 | 14 |
| 9 | 0.48 | 16 |
| 11 | 0.40 | 18 |
| 13 | 0.34 | 20 |
| 15 | 0.29 | 22 |
| 17 | 0.25 | 24 |
| 19 | 0.22 | 26 |

* Load based on a maximum capacity 70 ton railcar with a loaded weight of 212 kips.

Project EWPP - MURILLO

File 85C6214-2

Phase Engineering

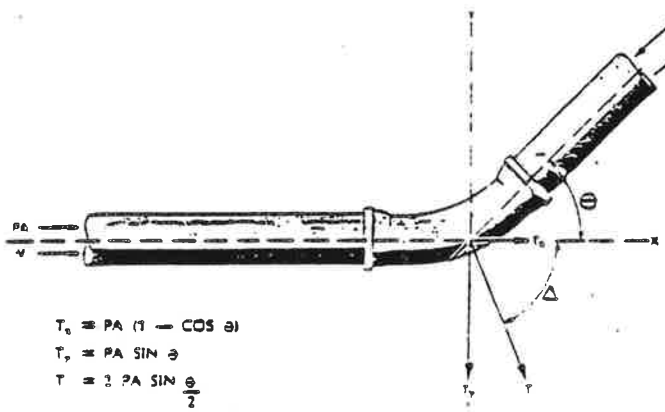
Made by CWS Date 6-4-86

Subject Typical Restraint Calculations

Checked by CVB. Date 6-6-86

# Thrust Force Calculations

## Equations:




$T_x = PA\,(1 - \cos\theta)$

$T_y = PA \sin\theta$

$T = 2\,PA \sin\frac{\theta}{2}$

$\Delta = (90 - \frac{\theta}{2})$

After Carlsen

where:
- $T$ = Resultant thrust force (lbs)
- $P$ = Maximum test or surge pressure (psi)
- $A$ = Inside pipe cross-sectional Area ($in^2$)
- $\Theta$ = Bend deflection angle (degrees)

## Typical example calculation:

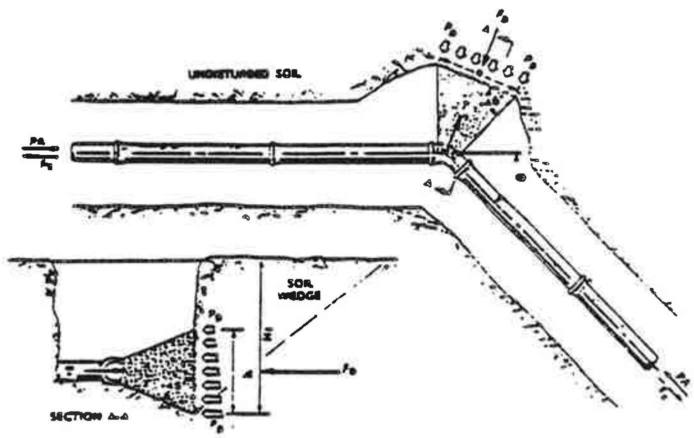
Calculate thrust force produced by 90° bend in concrete PRESSURE pipe with 90" I.D. & 105" O.D. Assume maximum allowable surge pressure of 210 psi.

$$T = 2PA \sin\frac{\Theta}{2} = 2(210)(6362)\sin\frac{90°}{2} = 1{,}889{,}418 \text{ lbs}$$

for design purposes say 1,889,400 lbs

## • Thrust Block Calculations

Equations:




**Required Bearing Area**

(1) General

$$A_b = hb = \frac{2PA \sin\frac{\theta}{2}}{P_p}$$

(2) If $h = 1/2\ H_T$

$$b = \frac{2PA \sin\frac{\theta}{2}}{3/8 w H_T^2 N_\phi + C_s H_T \sqrt{N_\phi}}$$

and

$$N_\phi = \tan^2 (45° + \phi/2)$$

- After Carlsen

where:

$A_b$ is the bearing area, $ft^2$

$P_p$ is the passive soil pressure, psf

$w$ is the soil unit weight, pcf

$\phi$ is the soil angle of internal friction, degrees

$C_s$ is the soil cohesion, psf

$h$ is height of thrust block, ft

$H_T$ is depth to bottom of block, ft

$b$ is required length of bearing surface (thrust block), ft.

## Typical example calculation:

Calculate the required size of thrust block to resist a 90° horizontal bend in pipline with 90" I.D. & 105" O.D. The maximum allowable surge pressure is 210 psi.

Assume: $^*h = 10'$

$H_T = 20'$

$\therefore h = \frac{1}{2} H_T$

Insitu clay soil parameter (Depth = 10'-20')

$C_s = 1500$ psf

$w = 125$ pcf

$\phi = 0°$ (Conservative)

$N_\phi = \tan^2(45°) = 1.0$

* Assume block bears on soil from 10' to 20' depth

Typical example calc. (cont.)

$$b = \frac{T}{3/8\, w H_T^2 + C_s H_T}$$ (simplified Equation for $N_\phi = 1$ case.)

$$b = \frac{1,889,400}{3/8(125)(20)^2 + 1500(20)} = \underline{38.8 \text{ ft}}$$

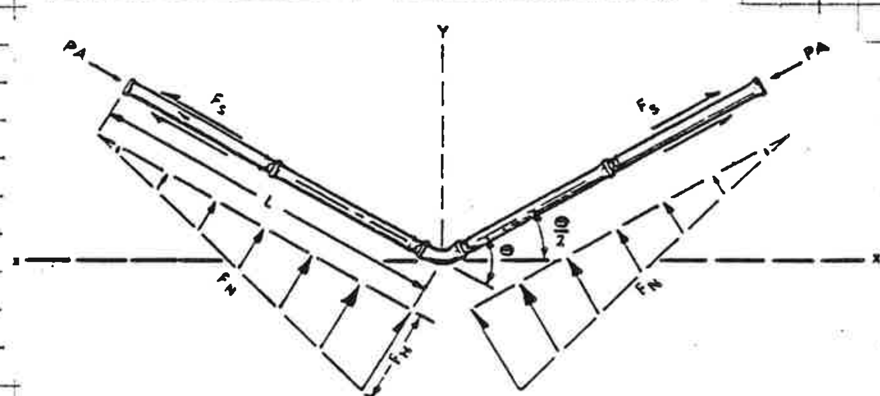
For design purposes required bearing length is approx. 40 ft.

∴ Area (Ab) required is on the order of 400 ft²

where: $A_b = b \times h$

- **RESTRAINED JOINT CALCULATIONS**

Equations:




Required length on pipe restraint (L)

$$L = \frac{S_f K P A}{K F_s + D P_p}$$

## Restrained Joint Calc. (Cont.)

Where:

$$K = 4 \tan \frac{\theta}{2}$$

$$F_s = A_p C + W \tan \delta \qquad \delta = f_\phi \phi; \; A_p = 2\pi r L \; (L = 1\text{ft.})$$

$$P_p = w H_c N_\phi + 2 C_s \sqrt{N_\phi} \qquad N_\phi = \tan^2(45° + \phi/2)$$

$$W = [\gamma \, D \, H_u \, w] \, R$$

$$C = C_s f_c$$

where:

$A_p$ = Outside surface area of pipe per unit length ($ft.^2$/L.F.)
$C$ = Pipe cohesion (psf)
$D$ = Pipe outside diameter (ft)
$f_c$ = Ratio of pipe cohesion / soil cohesion (see Table 1)
$f_\phi$ = Ratio of pipe friction angle / soil friction angle (see Table 1)
$H_c$ = Depth of cover to conduit center line (ft)
$H_u$ = Depth of cover above conduit (ft)
$K$ = Bend coefficient
$R$ = Reduction factor (see Table 2)
$W$ = Normal force on pipe (plf)
$\delta$ = Pipe friction angle
$S_f$ = Safety factor (normally 1.25)
$F_s$ = Conduit frictional resistance neglecting bell resistance (plf)
$A, \theta, \phi, C_s, w$ = As previously defined

## Typical Example calculation:

Calculate lenght of restraint required to resist the thrust developed by a 90° bend in pipeline with 90" I.D. & 105" O.D. Maximum allowable surge pressure is 210 psi.

Assume: $H_u = 10'$
$R = 3/4$
$S_f = 1.25$
$H_c = 15'$
Clay Cover [ $w = 125$ pcf, $\phi = 0°$ ]

Soil parameters: Assume clean sand bedding & backfill around pipe
∴ $\phi = 35°$ (conservative)
$f_\phi = 0.80$ (saturated conditions)
$C_s = 0.$
$w = 130$ pcf

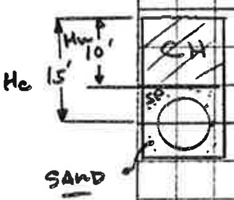
Example calc. (cont.)

4.125 8.25' 0.8

$$F_s = 2\pi(4.375)(\cancel{C_s}\,f_c) + \left[\left[\pi(8.75)(10)(125)\right](3/4)(\tan 0.8(35^\circ))\right]$$

12,919.6

$$\underline{F_s = 13{,}703 \ plf}$$




$$P_p = \underbrace{125(10')(\tan^2(45^\circ + \tfrac{0}{2}))}_{\text{CLAY}} + \underbrace{130(5')(\tan^2(45^\circ + \tfrac{35}{2}))}_{\text{SAND}} + 2\,\cancel{C_s}\,\sqrt{N_\phi}$$

2306

$$\underline{P_p = 3{,}649 \ psf}$$

$$\therefore L = \frac{1.25(4)(210)(6{,}362)}{4(13{,}703) + 8.75(3{,}649)} = 77.0 \ ft$$

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$\therefore$ Five Restrained Joints Required on Either Side of Bend (Assumes 16' Laying Lengths for Pipe).

TABLES OF FACTORS - Restrained Joints.

**Table 1**

**Soil Friction and Cohesion Factors**

| Soil Description | Friction Angle $\phi$ (Degrees) | Cohesion $C_s$ (psf) | $f_s$ | $f_c$ |
|---|---|---|---|---|
| ✓ Well graded sand: | | | | |
| Dry | 44.5 | 0 | 0.76 | 0 |
| Saturated | 39 | 0 | 0.80 | 0 |
| Silt (passing 200 sieve) | | | | |
| Dry | 40 | 0 | 0.95 | 0 |
| Saturated | 32 | 0 | 0.75 | 0 |
| Cohesive granular soil | | | | |
| Wet to moist | 13-22 | 385-920 | 0.65 | 0.35 |
| Clay | | | | |
| Wet to moist | 11.5-16.5 | 460-1,175 | 0.50 | 0.50 |
| At maximum compaction | | | 0.50 | 0.80 |

**Woodward-Clyde Consultants**

Eastern Region -- Houston Office



Project EWPP - Murillo

File 85C6214-2

Phase Engineering

Made by CWS Date 6-4-86

Subject Typical Restraint Calculations

Checked by CVB. Date 6-6-86

TABLES (Cont.)

Table 2

| Existing Condition | Reduction Factor R |
|---|---|
| ✓ 1. General construction | 2/3 |
| 2. Well-compacted backfill and selected backfill | 3/4 |
| 3. Shallow cover—Depth of cover less than 1/2 of the outside diameter | 1/2 |


Navigation Boulevard Alignment

GENERAL NOTES

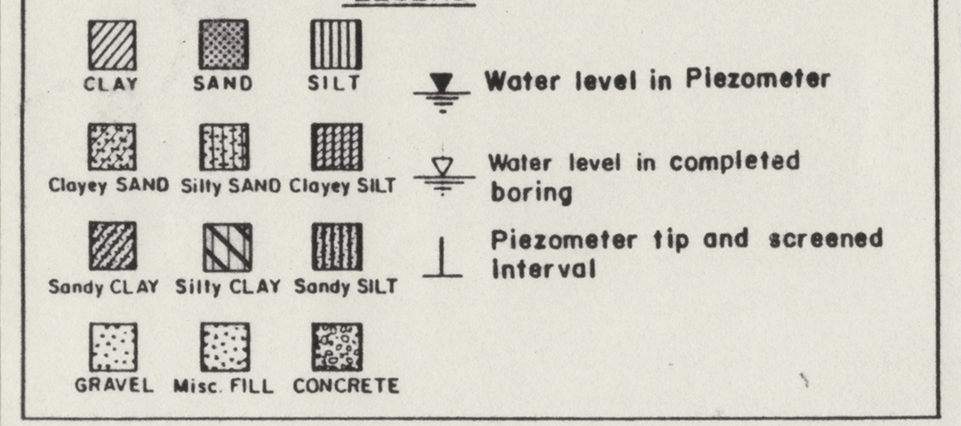
1. The interpretation of stratigraphic conditions along this alignment is based on widely spaced borings, consequently local variation in the stratigraphy should be expected.

2. The water table in this general area is known to vary with climatic conditions. The water table elevations shown on this profile are those measured shortly after each individual boring was completed. Consequently the actual water table at the time of construction should be expected to vary from that shown on this profile.

3. Boring locations are offset from the baseline at various distances. When the horizontal alignment of the pipeline is established the relationship of these boring locations to the pipeline should be determined in order to determine the degree of reliability which may be placed on each boring.

4. Project stationing was arbitrarily chosen to facilitate discussion of borehole locations. Locations and elevations of boreholes were surveyed by Cadastral Surveying.

LEGEND




SCALE : HORIZONTAL 500 0 500

VERTICAL 10 0 10

| | | | |
|---|---|---|---|
| NAME CITY OF HOUSTON EAST WATER PROGRAM | Murillo Engineering, Inc. | GENERALIZED SOIL PROFILE | FILE No. 85C6214 |
| FOR LOCKWOOW, ANDREWS & NEWNAM | SCALE: NOTED — MADE BY: J.P.W. DATE: 5-16-86 — CHECKED BY: CVB DATE: 5-23-86 | PACKAGE 5B | FIGURE C-1 |

# GEOTECHNICAL ENGINEERING REPORT
# WATER DISTRIBUTION SYSTEM IMPROVEMENTS
# EAST WATER PURIFICATION PLANT
# PACKAGE (DESIGN SEGMENT) 5B
# HOUSTON, TEXAS

## INTRODUCTION

The City of Houston has undertaken a capital improvement program involving improvements to the distribution system of the expanded East Water Purification Plant (EWPP). With the expansion of the EWPP, it is desired that more of the municipal water system demands be met by the use of surface water and that the service areas of the five existing groundwater facilities be incorporated into the service area of the EWPP to the maximum extent possible. In addition, system improvements are proposed to improve service levels within areas already served by the existing surface water treatment plant.

Additional conveyance lines, distribution pumping facilities, and storage capacity for treated water will be needed to distribute the increased supply of treated surface water. The East Water Program of Improvements includes more than 100 miles of new conveyance lines ranging in size from short segments of 12-inch lines to several miles of 96-inch lines. Large conveyance lines will function as supply lines during off-peak periods to carry treated water to storage tanks at the existing pump stations. These same lines will be connected to the existing distribution network at selected locations along their routes where they will aid in meeting the peak period demands in those areas.

Lockwood, Andrews, and Newnam, Inc. (LAN) has been contracted by the City of Houston to provide program management for the design engineering and contract development for this capital improvement program. LAN has entered into a subcontract with Murillo Engineering, Inc. (MEI) to provide geotechnical services for the Segment 5B improvements. Woodward-Clyde Consultants (WCC) has been retained by MEI to assist in providing geologic/geotechnical characterization and geotechnical analyses for the proposed improvements using field and laboratory data provided by MEI. The results of the geologic/geotechnical characterization and geotechnical analyses for the Segment 5B improvements are presented herein. Design Segment 5B has also been designated as Package 5B.

## DESCRIPTION OF PROJECT

The existing EWPP and facility expansion are located in far east Houston near the intersection of Federal Road and Clinton Drive. As shown in Figure 1, the alignment for Design Segment 5B extends from the intersection of North Wayside Drive and Navigation Boulevard west along Navigation Boulevard to Everton Street, a distance of approximately 11,000 feet. A trench of suitable width to accommodate a new 84-inch I.D. water distribution line will likely be excavated along the center median of Navigation Boulevard. An exception to this will likely occur in the vicinity of the Navigation Boulevard Bridge, at which location the pipeline will probably be placed along either the north or south right-of-way of Navigation Boulevard. The line will be installed in approximate 16 foot sections and will probably consist of prestressed concrete embedded cyclinder pipe. The operating pressure of the line will typically vary from 50 psi - 90 psi, although the system was analyzed assuming a design operating pressure of 150 psi. 96 psi

The scope of work for the Design Segment 5B alignment included a geologic study, a field investigation, a laboratory testing program, and engineering analyses. The purpose of this geotechnical study was to evaluate the subsurface stratigraphy and to assess the engineering characteristics of subsurface soils along the proposed alignment. Included in this study are design recommendations for vertical and horizontal anchorage, dewatering, bedding and backfill requirements, and other engineering considerations which may impact the proposed construction. The entire alignment of Design Segment 5B is expected to be constructed using open-cut (trenching) techniques, with the possible exception of the single railroad crossing which intersects the alignment.

## FIELD INVESTIGATION

An exploratory subsurface investigation for the Navigation Boulevard (Segment 5B) alignment were conducted by MEI in April 1986. The subsurface investigation consisted of 13 borings (5B-1 through 5B-13) drilled to depths ranging from 25 to 40 feet. The spacing of individual borings along the alignment generally varied from approximately 800 to 1,000 feet. The purpose of the borings was to assess soil conditions along the proposed alignment and to obtain soil samples for laboratory evaluation of soil properties. All borings were sampled continuously for the first 10 feet and at 5 foot intervals thereafter.

Following completion of drilling, borings were bailed and observation wells installed for short-term monitoring of groundwater in boreholes. Piezometers were also installed by MEI near Borings 5B-2 and 5B-11 for long-term monitoring of groundwater. Approximate boring and piezometer locations for Segment 5B are shown on the Boring Location Map in Figure 2. The locations and elevations of borings and piezometers were surveyed by Geogram Corporation. Surveyed locations of the borings related to baseline stationing are provided on the logs of borings. Logs of borings and piezometer data prepared by MEI are given in Appendix A. A subsurface profile along the alignment is given in Appendix C.

## LABORATORY TESTING

A laboratory evaluation of soil properties was conducted with samples obtained during the field investigation to evaluate the index and engineering properties of the subsurface soils. All tests were conducted by MEI. The results of the laboratory tests are provided in Appendix B and are summarized on the boring logs in Appendix A.

Index Properties

Index properties tests consisting of Atterberg limits, natural moisture content, dry unit weight, and particle size distribution relative to the No. 200 sieve were conducted to classify the soils encountered in the borings. The results of these tests are shown on the boring logs in Appendix A and are summarized on Tables B-1 through B-3.

Engineering Properties

Laboratory tests to evaluate the undrained shear strength of the subsurface soils included unconfined compression (UC) tests. The results of the strength tests are shown on the boring logs in Appendix A and are summarized in Tables B-1 through B-3.

## SUBSURFACE CHARACTERIZATION

Geology of the Coastal Plain

The Houston area is situated on the Quaternary Coastal Plain of Texas. Most of Houston is located on the nearly level, rather featureless depositional plains of the Montgomery and Beaumont Formations, while portions of the city are within the alluvial valley of Buffalo Bayou and its tributaries. The Beaumont and Montgomery depositional plains are two of five such surfaces recognized in the Quaternary Coastal Plain of Texas. Four of the five plains represent depositional surfaces that existed during Pleistocene interglacial stages, with the Beaumont (fourth) being the youngest. The fifth plain represents Holocene deposition during the current high sea level stage.

Pleistocene interglacial stages were periods when glaciers were melting or at minimum size (see Figure 3). The release of glacial water resulting from changes in climatic conditions produced high runoff with a corresponding high rate of sedimentation and the deposition of large quantities of sediment on or near the shoreline, or in the Gulf of Mexico. The five depositional plains in the coastal areas are separated by time gaps representing glacial stages when sea level was much lower and primary deposition took place on what is now the continental shelf.

The Quaternary formations of the Gulf Coast consist of sediments of similar depositional consistency. In order to understand the depositional sequence and depositional environments of the Quaternary, a knowledge of the geologic history and geologic processes related to their nature and genesis is of importance. Of primary interest is the understanding of glacial and interglacial stages (Figure 3).

Glacial stages were periods when there was a net lowering of the sea level due to the build-up of vast continental glaciers. Water which would normally reach the sea was stored as glacial ice and thus was unavailable to replenish the sea. Sea level during periods of maximum glaciation was as much as 450 feet lower than present. As the sea level was falling, streams adjusted their base level downward, cutting valleys. Drainage systems developed on the newly exposed soft sediment surface along the Gulf Coast. Also, as the sea level fell, the exposed surface was subjected to weathering and mature soil profiles developed. The mature soil profiles thus developed are characterized by a reddish color due to oxidation; by abundant nodules of iron, manganese, and calcium carbonate due to leaching and precipitation; and by two dimensional slickensided surfaces due to seasonal shrink-swell.

Interglacial stages occurred when the glaciers began to melt and retreat resulting in increased flow of water to the sea causing the sea level to rise. The increase in the amount of water led to greater sediment transport by streams. Along the coast, the valleys that were entrenched began to fill with sediment. Where sedimentation rates were small and could not keep pace with the rising sea level, the valleys were drowned often many miles inland. This was the case of the San Jacinto-Trinity Valleys, which now form Galveston Bay.

Rapid deposition of stream-born materials occurred when the stream intersected relatively still-standing bodies of water such as a bay or the Gulf of Mexico. Thus coastlines were centers of deposition. In the case where the sediment load of a stream was great, the valleys were initially filled with river (alluvial) sediments. Eventually streams built a land mass out into the gulf known as a delta. If the sediment load of a stream was low, as in the case of drowned valleys, the valley floor contained alluvial sediments overlain by bay or marine sediments. Deltaic sedimentation occurred initially at the head of the valley and progressed seaward thus overriding the bay marine sediment.

## Site Stratigraphy, Package (Design Segment) 5B

The stratigraphic unit which outcrops and is present in the subsurface along the alignment of Design Segment 5B is the Pleistocene Beaumont Formation. Man-placed fill was encountered in most of the borings along this alignment but the thickness generally did not exceed 4 feet. No fill was encountered in Borings 5B-5 and 5B-6 along the eastern reach of the alignment. The Beaumont Formation along this alignment consists of stiff to hard, low to high plasticity clays and medium to stiff clayey sands and clayey silts. The depositional environment of the Beaumont in this area was on a lower alluvial-deltaic plain of the ancestral Brazos and Trinity Rivers.

Soils encountered in the upper 20 to 30 feet along the alignment predominantly consisted of low to high plasticity clays with occasional clayey sand and clayey silt strata. Clayey sand strata were encountered below a depth of 10 feet in Borings 5B-5 and 5B-6 (Sta. 75+90 and Sta. 64+90) along the eastern reach of the alignment, while a clayey silt layer was encountered in Boring 5B-3 (Sta. 88+60) at the east end of the alignment between a depth of 17 and 23 feet. All borings along the alignment were terminated in low to high plasticity clays. These soils most likely represent overbank flood basin deposits of the deltaic system.

## Regional Structural Geology

Progressively older Pleistocene depositional plains outcrop farther inland and dip seaward under sediments forming the next successively younger plain. The differential vertical movement is the result of the very slow and progressive development of the Gulf Coast Geosyncline and of greater compaction rates in the offshore areas where finer grained sediments have been deposited. The Gulf Coast Geosyncline is an elongated structural trough that subsided through geologic time and has been receiving sediments concomitant with subsidence.

Regional offshore subsidence and inland uplift have resulted in gradual seaward tilting of the prism of sediments in the geosyncline and have continuously taken place throughout the Quaternary period. The Houston area is located in the north flank of the Gulf Coast Geosyncline.

Faulting is present in the Quaternary sediments on the Gulf Coast. Principal classes of faults within and beneath the Quaternary sediments are "growth faults" and faults associated with intrusive salt domes. Growth faults are non-tectonic fractures that develop contemporaneously with deposition. As the Gulf Coast Geosyncline subsides gulfward, the dip of the depositional surface increases and thus subsidence, combined with the overloading of deltaic materials, increases the tendency for gulfward slumping of the sediments. The water-saturated, unconsolidated sediments slump downdip much like slump-block landslides, creating growth faults along the trend of the dip changes. The principal mechanisms of these processes are differential compaction and gravitational sliding. Faults associated with salt domes are similar to growth faults, but the driving mechanism consists of mobile salt masses which form the domes.

Differential surface displacement related to active fault movement has taken place throughout historic time in the Houston Area. The natural movements related to geosyncline shifts and faulting have occurred over a long period of geologic time. Of themselves, these structural features would have had little bearing on the project. Man's activities, however, have served to accelerate ground movements that may impact the project. These are discussed below.

Natural Hazards

Natural hazards, as used herein, are defined as those geologic hazards that impact man or his activities. The natural hazards that appear to pose the most severe impact to the EWPP water distribution system improvements are subsidence and growth faults. However, no growth faults were identified which would intersect or otherwise impact the Design Segment 5B alignment.

One of the most notable phenomenon (hazard) in the Houston-Galveston area is regional differential surface displacement due to surface subsidence. Subsidence has occurred at a maximum average rate of about 0.5 feet per year, with a total subsidence of over 7-1/2 feet being observed between 1943 and 1973 in the southeast Houston area. Most of this subsidence is related to the withdrawal of groundwater in the normally pressurized aquifers. Piezometric levels of groundwater in the shallow aquifers have declined as much as 325 feet between 1943 and 1973. Subsidence, as recorded by resurveys of surface benchmarks throughout the area and by deep compaction recorders to depths of about 3,000 feet at selected locations, show a direct correlation with declines in the piezometric head. Subsidence would principally affect the system improvements through changes in grade over the life of the project. Subsidence, at least at present and projected rates, is not expected to adversely impact the project considering the fact that the water conveyance lines form a pressurized system.

The Texas and Louisiana portions of the gulf coastal region, including Houston, are characterized by a very low level of exposure to seismic hazards. The largest historical earthquakes in the region have occurred in east Texas and southern Louisiana. The quakes only produced minor damage to structures. Consideration of seismic forces are not considered relevant to this project.

## Geotechnical Characterization

The following paragraphs describe our interpretation of the subsurface soil and associated laboratory data along the alignment of Design Segment 5B. This interpretation of subsurface conditions was made in order to provide LAN with general geotechnical parameters pertinent to this alignment for their use in design of water conveyance lines, development of an engineer's construction cost estimate, and to aid in the evaluation of construction bids. No other use of these interpretations is intended, as uses other than those described may lead to erroneous conclusions.

Soil Characterization. An evaluation of the field and laboratory data indicates that the subsurface conditions along the Segment 5B alignment may be described in terms of two generalized strata: (1) random fill and (2) the Beaumont Formation. A subsurface profile along the Segment 5B alignment is included in Appendix C.

Random fill was encountered in most of the borings along the alignment of Segment 5B from the ground surface to a maximum depth of approximately 4 feet. No fill was encountered in Borings 5B-5 and 5B-6 along the eastern reach of the alignment. The random fill generally consists of a low to high plasticity clay. Due to the relatively shallow depth of fill along the alignment, extensive laboratory testing of the fill was not conducted. The cohesive fill material had measured undrained shear strengths which ranged from approximately 1,000 to 4,500 psf.

Pleistocene soils (the Beaumont Formation) were encountered throughout the entire alignment from below the random fill (where present) to the termination depth of borings. The soils within the Beaumont Formation generally consist of interbedded layers of stiff to hard clay, silty clay, and sandy clay with occasional medium to stiff clayey sand and clayey silt strata. Clay and silty clay soils encountered in the upper 30 feet along the alignment had measured undrained shear strengths which ranged from 600 to 4,400 psf and plasticity indices which ranged from 29 to 54. Below a depth of 30 feet, these clay soils had measured undrained shear strengths on the order of 1,100 to 3,300 psf and plasticity indices which ranged from 9 to 55. A pronounced secondary structure, i.e., joints and fissures which are often slickensided, is noted at locations in the clay soil strata. Sandy clay soils encountered in the upper 30 feet along the alignment had measured undrained shear strengths on the order of 400 to 3,800 psf and plasticity indices which ranged from 3 to 22.

The granular soils which were encountered in the subsurface along the alignment of Design Segment 5B consisted of clayey sands and clayey silts. The clayey sand strata which were encountered below a depth of 10 feet along the eastern reach of the alignment (Borings 5B-5 and 5B-6) had measured undrained shear strengths on the order of 100 to 200 psf and plasticity indices which ranged from 4 to 5. A sample of the clayey sand which was tested from Boring 5B-5 had a blow count measured by a Standard Penetration test of 42 blows per foot of penetration and a measured value of 19 percent finer than the No. 200 sieve. The clayey silt layer which was encountered in Boring 5B-3 at the east end of the alignment between a depth of 17 to 22 feet had a measured undrained shear strength of 200 psf and a plasticity index of 4.

Values of soil properties for use in design of braced excavations and reactions to unbalanced hydrostatic forces are given in Table 1.

Groundwater Conditions. Groundwater elevations measured at the time the field investigation was conducted are indicated on the interpreted profile (Appendix C) and on the logs of borings in Appendix A. The measured static groundwater level at most boring locations is well above the anticipated bottom of excavation for the proposed new water conveyance pipeline.

## GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Geotechnical recommendations and parameters which may be required for the design and open-cut construction of this design segment of the distribution system improvements are discussed herein. Specifically, the recommendations and parameters addressed are:

- Excavation slopes or bracing and dewatering, where appropriate, for the pipeline placement;
- Bedding and backfill requirements;
- Special considerations for surcharge loads;
- Thrust reaction at the horizontal curve in Design Segment 5B near the midpoint of the alignment, at the possible offset in the alignment in the vicinity of the Naviation Boulevard Bridge, and at the terminal (west) end of Design Segment 5B; and
- Thrust reaction and vertical anchorage at the single railroad crossing at approximate Sta. 88+00, and at other locations where existing utilities and/or unidentified obstructions may intersect the alignment.

Excavation for Pipeline Placement

It is assumed that the excavation for the 84-inch pressure pipeline for this design segment will be on the order of 15 to 20 feet. This assumption is based on the maximum outside diameter of prestressed concrete embedded cylinder pipe on the order of 8-1/2 feet, 1 to 2 feet of over excavation for bedding placement, and 5-1/2 to 9-1/2 feet of backfill cover. The trench width (at the elevation of the pipe centerline) should be as narrow as possible consistent with sufficient clear distance between the pipe and the trench walls to allow proper placement of backfill along the sides of the pipe. For this design segment, with pipe on the order of 8 to 8-1/2 feet in outside diameter, we recommend that the trench width be in the range of 10 to 11 feet.

We understand that a majority of the pipeline will likely be placed below the median of Navigation Boulevard for this reach of the distribution system improvements. Assumed excavation and pipeline placement dimensions in relation to the median along Navigation Boulevard are provided on Figure 4a. For the portion of the pipeline which will likely be placed along either the north or south right-of-way of Navigation Boulevard (offset at Navigation Boulevard Bridge), assumed excavation and pipeline placement dimensions are shown on Figure 4b. Due to top width requirements, safe excavation slopes of less than 1-1/2 or 2(H) to 1(V) are not feasible in the stratigraphy along the alignment of Design Segment 5B. Further, numerous underground utilities and other obstructions are likely located beneath the median and within the right-of-way along either side of Navigation Boulevard. Consequently, unless a portion of Navigation Boulevard is closed to traffic a shored or braced excavation with vertical walls is, in our opinion, the only feasible method of laying this reach of the pipeline.

It is possible that a trench shield (drag shield) could be employed to lay the pipe for this alignment, however, trench shields are typically not designed to support the walls of the excavation. Their primary purpose is to protect workers and the work area from sloughs and cave-in of the excavation walls. Consequently, the use of a trench shield would not prevent sloughs which could damage the existing pavement of Navigation Boulevard. It is also possible that major sloughs could "freeze" the shield requiring additional braced excavation to free it.

In our opinion, the most practical methods of maintaining an open excavation for placement of the pipeline would be by bracing the excavation walls with either cantilevered sheet piling or internally braced shoring. The contractor should be responsible for design (by a registered professional engineer) of his intended bracing system consistent with his methods, equipment, and experience. Shop drawings of his bracing design should, of course, be reviewed by the owner's engineer.

To assist in developing conceptual designs for cost estimating purposes, an earth pressure diagram for an internally braced shoring system of a 20 foot excavation in the median (or along the right-of-way) of Navigation Boulevard is provided on Figure 5. A similar earth pressure diagram for a cantilevered sheet piling system is provided in Figure 6. The pressures indicated on these figures have assumed a typical 500 psf construction surcharge and no dewatering. The construction surcharge will depend on the contractors actual operations, i.e., if the contractor plans to store excavation spoil adjacent to the open trench, the surcharge load should reflect the unit load of this windrowed soil. Also, in areas which are dewatered the hydrostatic load will be substantially reduced or eliminated.

After placement, if sheeting is removed, the lateral pressure on the pipe will likely increase with time until the lateral pressure on the pipe is approximately equal to the overburden pressure at that depth. Vibratory extraction of sheeting may cause surface settlement above the pipe, especially if granular backfill is used.

Dewatering

With the exception of the east end of the alignment, a majority of the trench excavation for Design Segment 5B will be through clay soils (see Generalized Soil Profile, Appendix C). At the east end of the alignment, a "wet" sand or silt may be encountered within or slightly below the trench excavation. Where granular deposits are anticipated, we recommend that the water table be lowered to approximately 2 feet below the bottom of the excavation.

It is common practice in this area to excavate trenches into water bearing granular deposits, place free-draining granular material in the bottom of the trench, and collect the seepage in sumps in the bottom of the trench for pumped disposal. There is, however, at least one documented case in which this procedure led to failure of the pipe. In this case the upward seepage so loosened the soil below the bedding that when the pipe was laid and backfill placed, the pipe settled breaking at and between joints. Consequently, we recommend that these water bearing granular strata be dewatered prior to excavation.

Based on the interpreted soil profile (Appendix C), granular soils below the water table which may impact the excavation and subsequent pipelaying are identified at the east end of the alignment between approximately Sta. 59+00 and Sta. 92+50. Because of the wide spacing of these borings, it is likely that additional granular lenses, layers, or channels may be encountered which require dewatering.

Strata which are likely candidates to be dewatered between approximately Sta. 59+00 and Sta. 92+50 are predominately clayey sands and clayey silts. We recommend that the contractor conduct one or more pumping tests in these deposits to determine the most feasible method of dewatering. The most efficient method of dewatering this material will likely be with vacuum wellpoints. The maximum lift with a vacuum wellpoint system is on the order of 15 to 18 feet. Consequently, the contractor may have to set the wellpoint header on a shallow bench within his excavation.

Pipe Bedding and Backfill Requirements

The load-carrying capacity of a ditch conduit in the field is influenced to a large extent by the bedding and backfill conditions. Due to the very large size (84-inch I.D.) of the water conveyance pipeline to be installed on this project, various bedding conditions were evaluated in terms of load factors to establish the proper bedding and backfill criteria for various external loading conditions in the field. The load factor for a given bedding condition is defined as the ratio of the load-carrying capacity of the conduit in the field to the strength of the conduit measured in a standard three-edge bearing test in a laboratory.

For concrete embedded cylinder pipe (AWWA Standard C-301), we recommend a "Type 3" bedding be employed for the alignment along Navigation Boulevard. A typical section showing this recommended bedding and backfill is provided on Figure 7. We recommend that a relatively clean sand be used for bedding and to provide backfill to slightly above the top of the pipe. The bedding should be uniformly placed and hand tamped. Backfill should be placed in lifts not exceeding 6 inches and tamped. If sluicing or flooding is used to densify the backfill, water draining from the backfill should be collected and disposed of immediately.

Sand used for bedding and backfill around the pipe should consist of select sandy soil or other granular material free from clay lumps, organic matter, construction rubble, stones, or other deleterious substances. The City of Houston's "Specifications for Water Main Construction" recommends a sandy soil with a plasticity index of less than

7 and not more than 40 percent passing the No. 200 Sieve. In our opinion, this specification is adequate, however, if sluicing or flooding is used for backfill densification, we recommend that a clean sand be specified. The clean sand should be non-plastic and have not more than 15 percent of its particles passing the No. 200 sieve.

If excavation spoil is used to backfill the trench, the backfill should be compacted to approximately 95 percent of Standard Proctor density where it will be situated beneath paved areas subject to traffic. Control of backfill density is less critical where the pipe is situated beneath vegetated areas along the median or right-of-way of Navigation Boulevard. However, in these areas, we recommend that topsoil be placed as the upper 6 inches of the trench backfill.

Rigid pipe bedded and backfilled in this manner will have a load factor, $L_f$, of approximately 1.5. If exceptionally heavy loading conditions are identified at any place along this alignment, the load factor may be increased by specifying AWWA Type 4 or Type 5 Bedding.

Vehicular Traffic and Railroad Loads

The conveyance pipeline proposed for Design Segment 5B will pass beneath a number of "turn arounds" and left turn bays in the Navigation Boulevard median and therefore will be subjected to loads at these locations resulting from vehicular traffic. The pipeline will also pass beneath a number of streets and drives which intersect the alignment where situated along the north or south right-of-way of Navigation Boulevard. The alignment for Design Segment 5B also passes beneath a railroad at approximately Sta. 88+00.

Calculated vertical loads resulting from vehicular traffic and trains at different depths below the pavement or cross ties are presented in Tables 2 and 3, respectively. The design vehicle for the traffic loading table is a 120 kip tractor-trailer. The design railroad loading is a 70-ton capacity car with a loaded weight of

212 kips.

The additional load (in addition to soil overburden) to which the pipe will be subjected may be used to check the adequacy of the class of pipe or the bedding criteria to be used at these locations.

Thrust Restraint

Every time that the direction of flow or the velocity of flow changes within a pressure pipeline, an unbalanced hydrostatic force is created at the point of change. If these changes are significant, a reaction must be provided for this unbalanced force or damage to the pipe or separation at the pipe joints may occur. Within a pressure pipeline system, unbalanced hydrostatic forces will be associated with vertical and horizontal bends, wyes, tees, offsets, valves, reducers, etc.

No changes in velocity, i.e., reducers or valves, are anticipated throughout the length of this design segment. The only known major horizontal unbalanced hydrostatic force will be associated with hydrotesting of the new 84-inch I.D. pipeline. If the terminal (west) end of Design Segment 5B is plugged and hydrotested prior to connection to the next design segment, the unbalanced hydrostatic force generated at the west end of the alignment will be approximately 832,000 pounds. This assumes that the pipeline is operated at a design pressure of 150 psi.

As shown in Figure 2c, a sharp horizontal curve occurs in the Segment 5B (Navigation Boulevard) alignment in the vicinity of Engle Street (Sta. 46+00). The radius of this curve is estimated to be approximately 500 feet. Based on review of manufacturer's design information for the 84-inch I.D. prestressed concrete embedded cylinder pipe, the maximum allowable deflection angle per joint of pipe is on the order of $0.65^{\circ}$ (2-1/8 inch offset) assuming a 16 foot laying length for the pipeline. The maximum deflection angle of $0.65^{\circ}$ will produce a maximum allowed curve radius of approximately 1,400 feet when nominal 16 foot pipe lengths are used. Consequently, an approximate $30^{\circ}$ bend will likely be required in the 84-inch I.D. pipeline at this location. The unbalanced hydrostatic force thus generated by the 30° bend in the pipeline will be on the order of 430,500 pounds, assuming a 150 psi design operating pressure. The direction of this thrust will be in a horizontal plane at an angle of 105° to either reach of the alignment. A similar bend will likely be required at the beginning and end of the probable offset in the pipeline alignment in the vicinity of the Navigation Boulevard Bridge (Sta. 97+00 and Sta. 81+00, respectively).

In order to provide reaction for these unbalanced hydrostatic forces, the force has to be distributed to the surrounding soil. Further, the thrust must be distributed over a sufficient area of the soil so that the soil does not fail. Commonly used methods of distributing this thrust force to the soil at a safe level of stress is through the use of thrust blocks or restrained joints.

Thrust blocks are generally massive reinforced or unreinforced concrete blocks which distribute the resultant thrust force over a large vertical surface of the soil. Thrust blocks are sometimes augmented by battered piles which aid in resisting the thrust force. For a thrust block to resist the thrust forces at the previously described locations a large bearing area will be required.

Assuming the thrust block has a height of 10 feet and bears on the soil between a depth of 10 and 20 feet below the ground surface, the required length of the bearing surface to resist the 832,000 pound axial force generated during hydrotesting of the 84-inch pipeline will be on the order of 15 to 20 feet. Therefore, for a thrust block bearing on the soil between these depths the required bearing area is between 150 and 200 square feet. This assumes 8 to 10 feet of cover will be placed over the pipeline. A thrust block with a bearing area on the order of 50 to 100 square feet would be required to resist the 430,500 pound force generated by the 30° bend in the 84-inch pipeline near the midpoint of the alignment (Sta. 46+00). A thrust block of the same size would be required at the beginning and end of the possible offset in the pipeline alignment in the vicinity of the Navigation Boulevard Bridge (Sta. 81+00 and Sta. 97+00) due to similar bends at these locations. If a thrust block is designed to provide any of these reactions, reinforcement may be required because of its size to insure that the block acts as an integral unit.

The second method of providing reaction to thrust forces is through restrained joints. The pipe joints are typically restrained in such a manner that they can rotate slightly, however they cannot separate. The pipe then takes advantage of the passive resistance of the soil on which it bears and the frictional resistance between the soil and the pipe.

If restrained joints are used to provide reaction to thrust forces due to the 30° bends in the 84-inch I.D. pipeline near the midpoint of the alignment and at the possible offset in the alignment in the vicinity of the Navigation Boulevard Bridge, at least 15 feet of the pipe on both sides of each bend would require restrained joints. This length of restraint assumes 8 to 10 feet of cover. For shallower cover the length of pipe with restrained joints would be much longer. Consequently, with 8 to 10 feet of cover at least one joint on both sides of each bend would require restraint, assuming a 16 foot laying length for the pipeline.

Details of the design methodology used in sizing thrust reaction systems were included in the Preliminary Geotechnical Engineering Report submitted to LAN by WCC in October 1984.

Several locations exist along the proposed pipeline alignment at which vertical bends may be required to allow crossing under (or over) existing obstructions. Examples of locations which may require vertical bends are:

- The railroad tracks crossing the proposed alignment at approximate Sta. 88+00; and
- Other locations where existing utilities and/or other unidentified obstructions may intersect the proposed alignment.

Where vertical changes in the pipeline alignment occur an unbalanced hydrostatic force is developed as with horizontal bends in the pipeline. Reaction is necessary to resist the unbalanced hydrostatic force and thus prevent strain and possible distress to the pipeline. The type of reaction provided at these locations is dependent on the magnitude and direction of the unbalanced force.

Since the design depth of the pipeline and the vertical angle of deflection is not known at this time, only general comments can be made with respect to providing reaction at these locations. Both thrust blocks and restrained joints may be employed. The direction of the resultant unbalanced force is very important in selecting and designing the reaction system.

If a vertical bend produces a resultant force which is downward with respect to a horizontzal plane, thrust blocks may be designed with respect to bearing capacity by procedures developed for inclined footings. Parameters required for this design are:

- Properties of the insitu soil at and below the bearing surface of the thrust block;
- Depth to the bearing surface;
- Dimensions of the bearing surface; and,
- Angle and magnitude of the resultant force.

For a downward resultant force at vertical bends and a bearing surface between 15 and 25 feet below the ground surface, thrust blocks, in our opinion, may be designed for an allowable bearing capacity (F.S.=2) of:

- Between 1,000 and 7,000 psf where the thrust block bears on clay.

- Between 640B and 3,200B psf where the thrust block bears on sand or silt. B is the least dimension of the bearing surface.

If a vertical bend produces a resultant force which is upward with respect to a horizontal plane, thrust blocks should be designed to resist the resultant force by dead weight only. The dead weight of the thrust block may include the weight of the concrete and the weight of the soil prism above the block. The reaction capacity of thrust blocks designed to resist resultant forces which are inclined upward from the horizontal may be enhanced by the use of driven friction piling, straight shafts, or drilled-underreamed shafts.

Restrained joints may be used to resist unbalanced resultant forces occurring at vertical bends in the pipeline. Restrained joint reactions may be designed in the same manner as those designed for horizontal bends with respect to soil pipe friction. The passive resistance component of restrained joints used for horizontal bends should, however, be replaced by:

- Methods of design for inclined continuous footings when the resultant force is downward with respect to the horizontal; and,

- The dead weight of the pipe and overlying soil prism when the resultant force is upward with respect to the horizontal.

CREDITS

This report has been prepared for Lockwood, Andrews, and Newnam, Inc. for their use in design of Segment 5B improvements to the distribution system of the expanded East Water Purification Plant. Subsurface data collection (soil borings) and laboratory testing was conducted by Murillo Engineering, Inc. Assistance in report preparation was provided by Woodward-Clyde Consultants. Project management was provided by Murillo Engineering, Inc.

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Vladi H. Vonas, P.E.

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FIGURES


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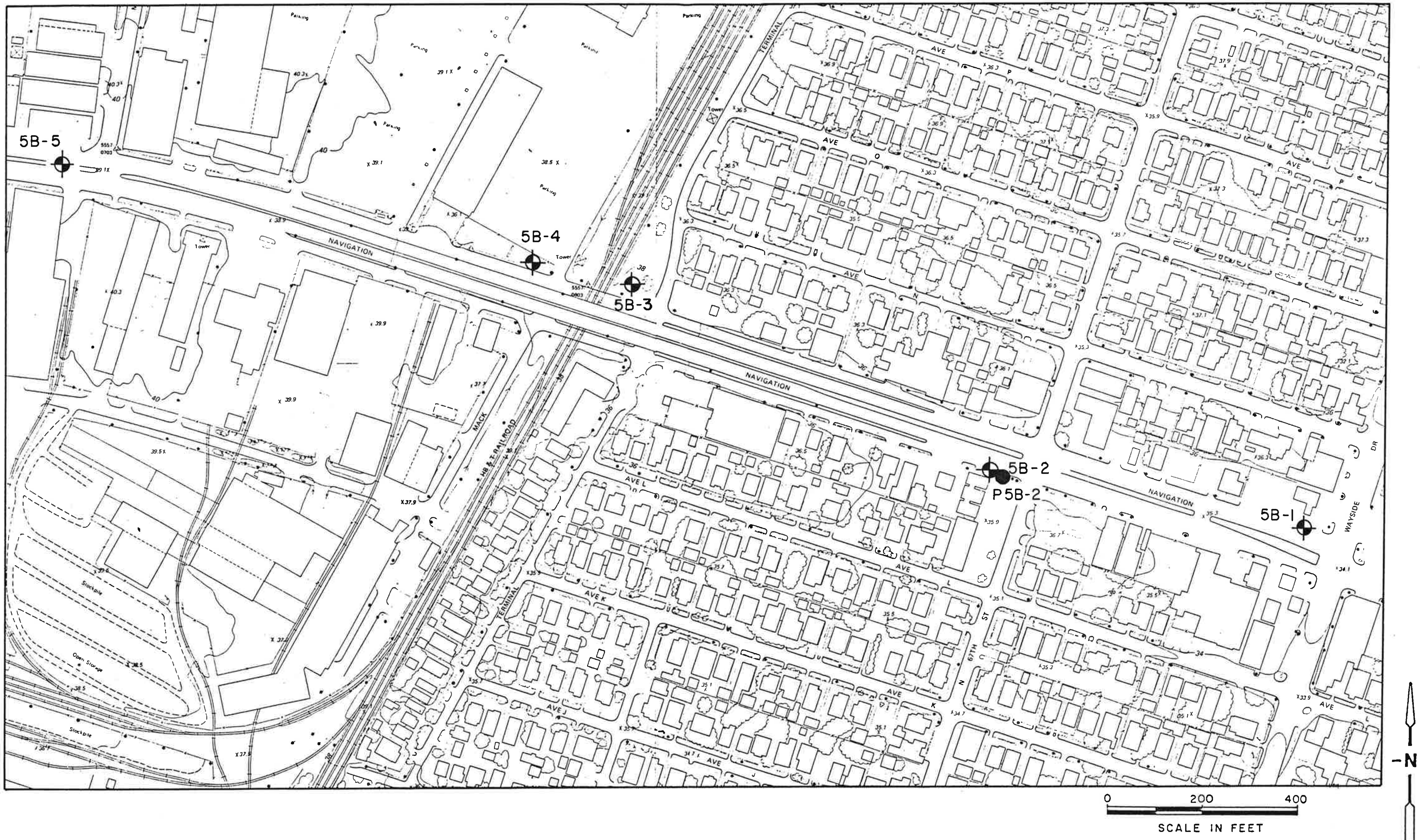
| NAME: City of Houston East Water Program | Murillo Engineering, Inc. | | SITE LOCATION MAP | FILE No. 85C6214 |
| FOR: LOCKWOOD, ANDREWS & NEWNAM | SCALE: NOTED | MADE BY: R.G. DATE: 4-25-86<br>CHECKED BY: CVB. DATE: 5-23-86 | | FIGURE 1 |

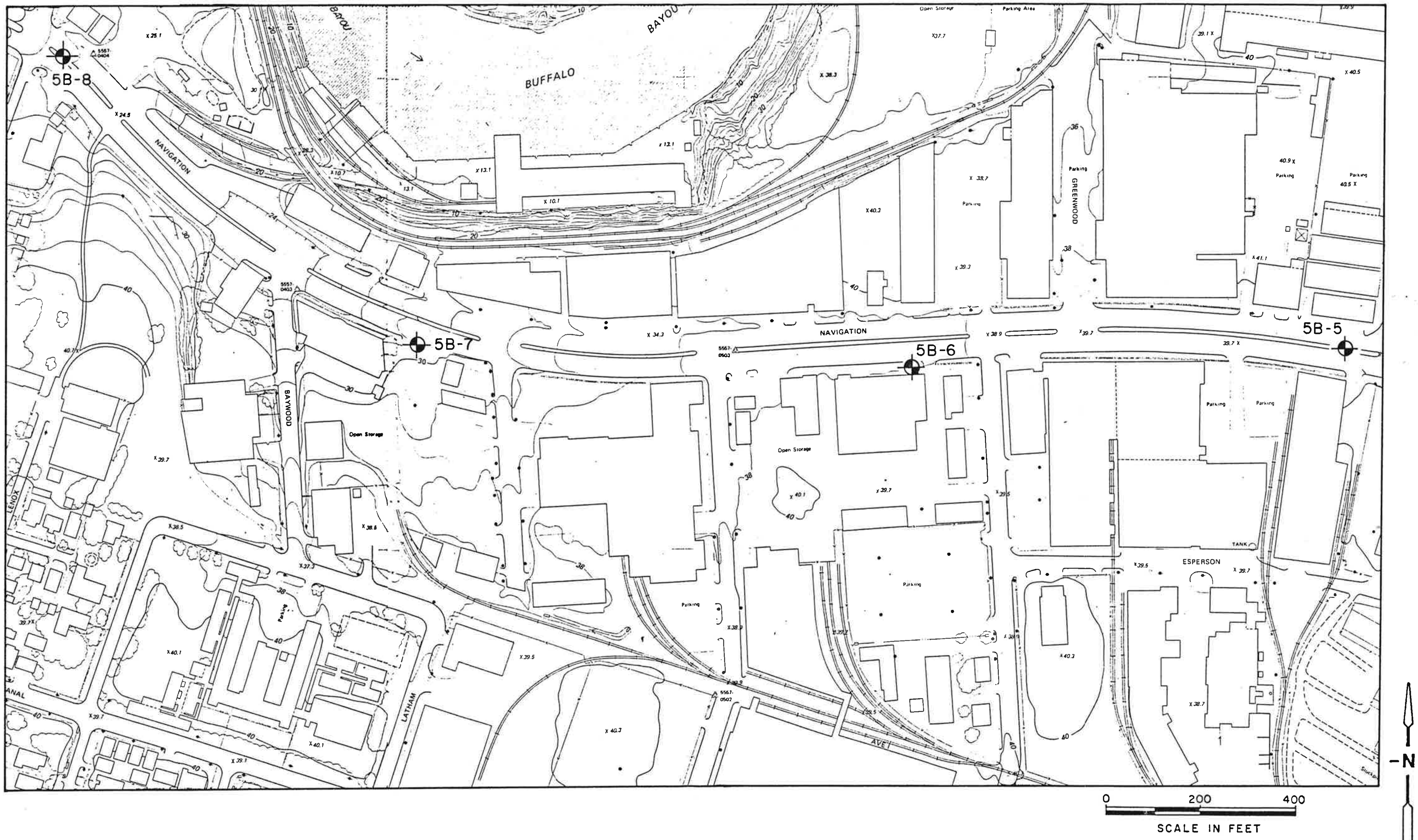
FILE No. 85C6214
FIGURE 1




LEGEND

| | | |
|---|---|---|
| ⊕ | 5B-11 | Borehole Location |
| ● | P5B-11 | Piezometer Location |

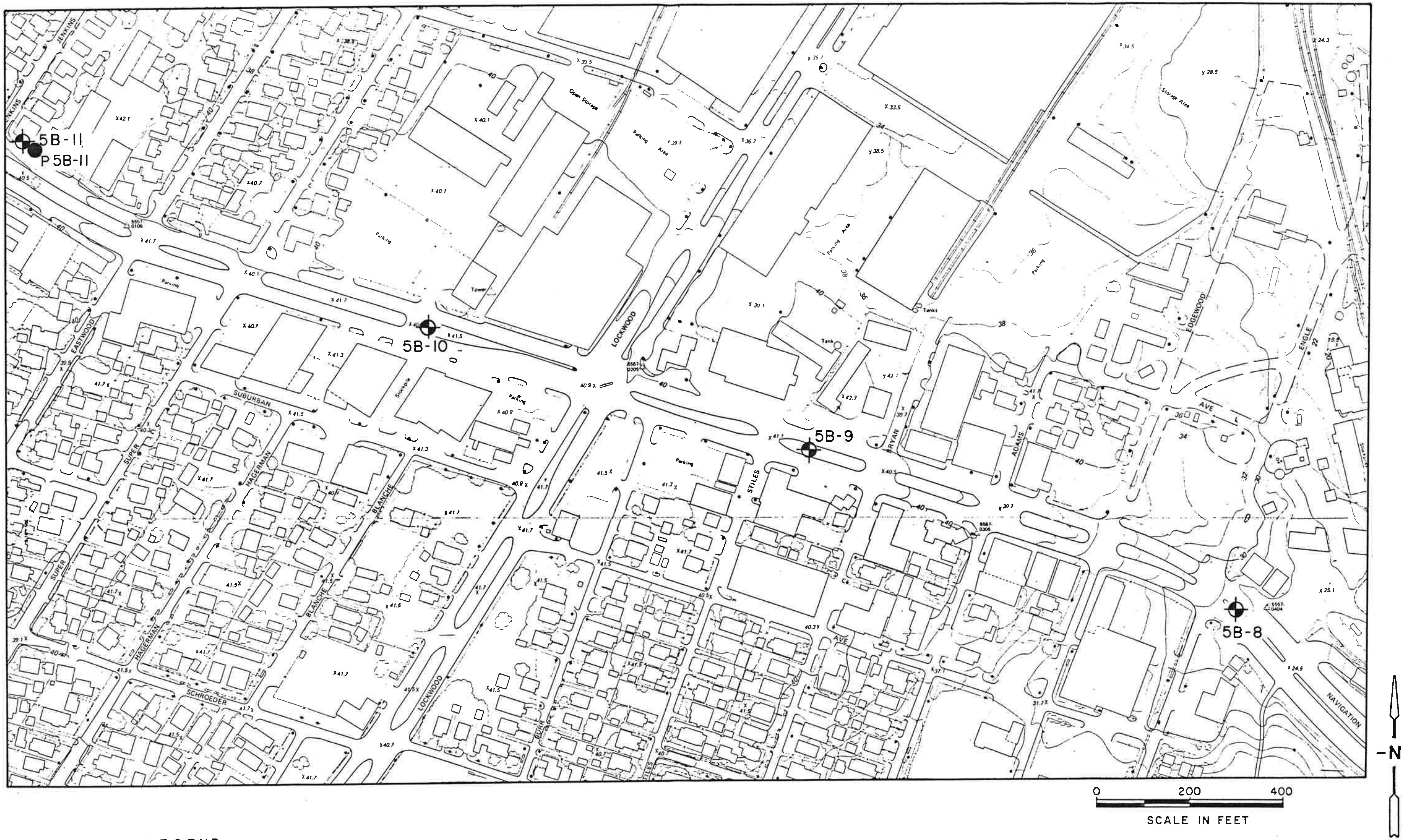
| | | | | |
|---|---|---|---|---|
| NAME: City of Houston East Water Program | Murillo Engineering, Inc. | | SEGMENT 5B (East-West) BORING LOCATION MAP | FILE No. 85C6214 |
| FOR: LOCKWOOD, ANDREWS & NEWNAM | SCALE: NOTED | MADE BY: R.G. DATE: 5-15-86<br>CHECKED BY: CVB. DATE: 5-23-86 | | FIGURE 2a |




LEGEND

| | |
|---|---|
| 5B-11 | Borehole Location |
| P5B-11 | Piezometer Location |

| | | | |
|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | SEGMENT 5B (East-West) BORING LOCATION MAP | FILE No. 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NOTED — MADE BY: R.G DATE: 5-15-86 — CHECKED BY: CVB. DATE: 5-23-86 | | FIGURE 2b |

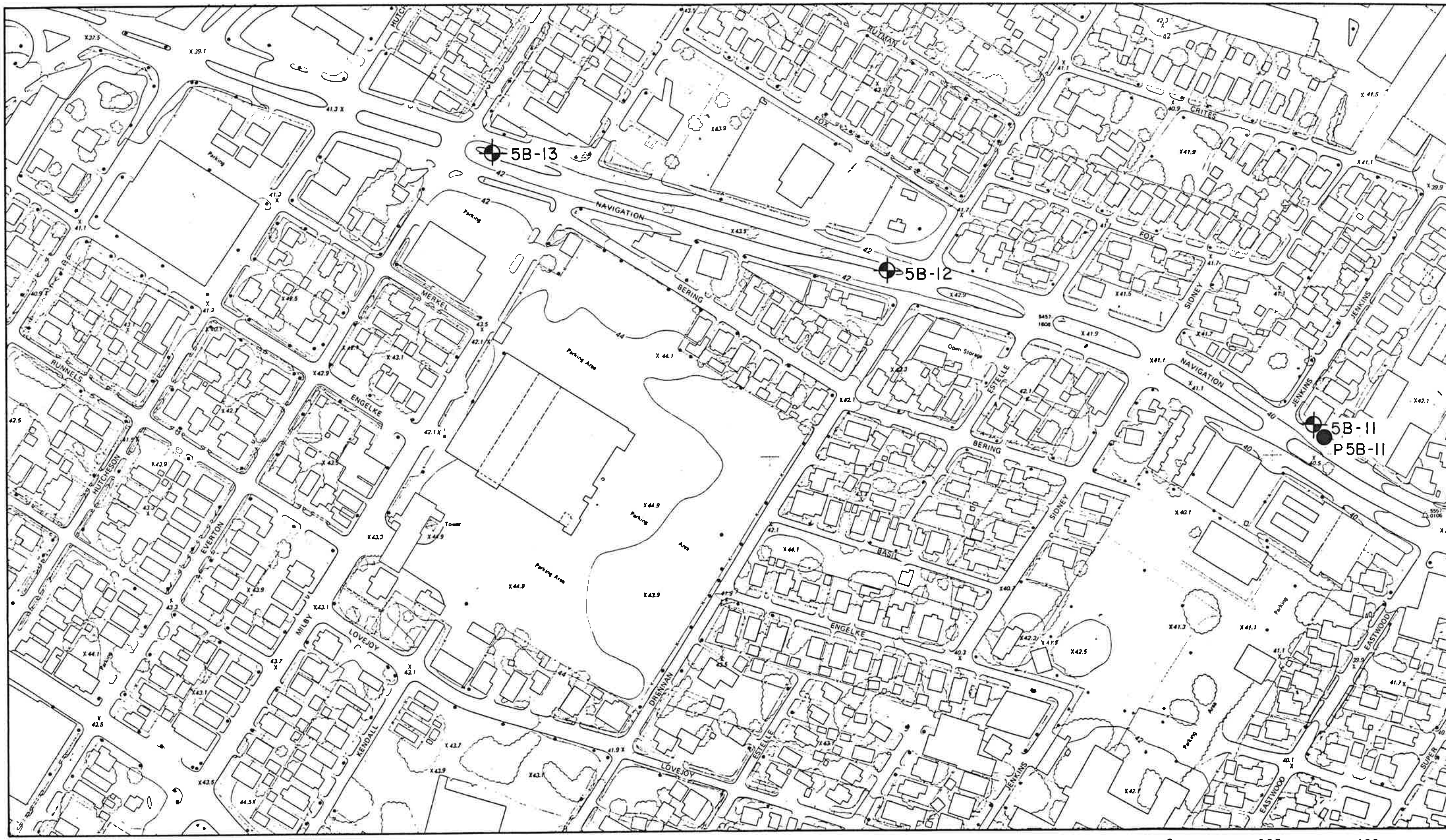



LEGEND

5B-11 Borehole Location

P5B-11 Piezometer Location

| | | | |
|---|---|---|---|
| NAME: City of Houston East Water Program | Murillo Engineering, Inc. | SEGMENT 5B (East-West) BORING LOCATION MAP | FILE No. 85C6214 |
| FOR: LOCKWOOD, ANDREWS & NEWNAM | SCALE: NOTED; MADE BY: R.G. DATE: 5-15-86; CHECKED BY: CVB. DATE: 5-23-86 | | FIGURE 2c |



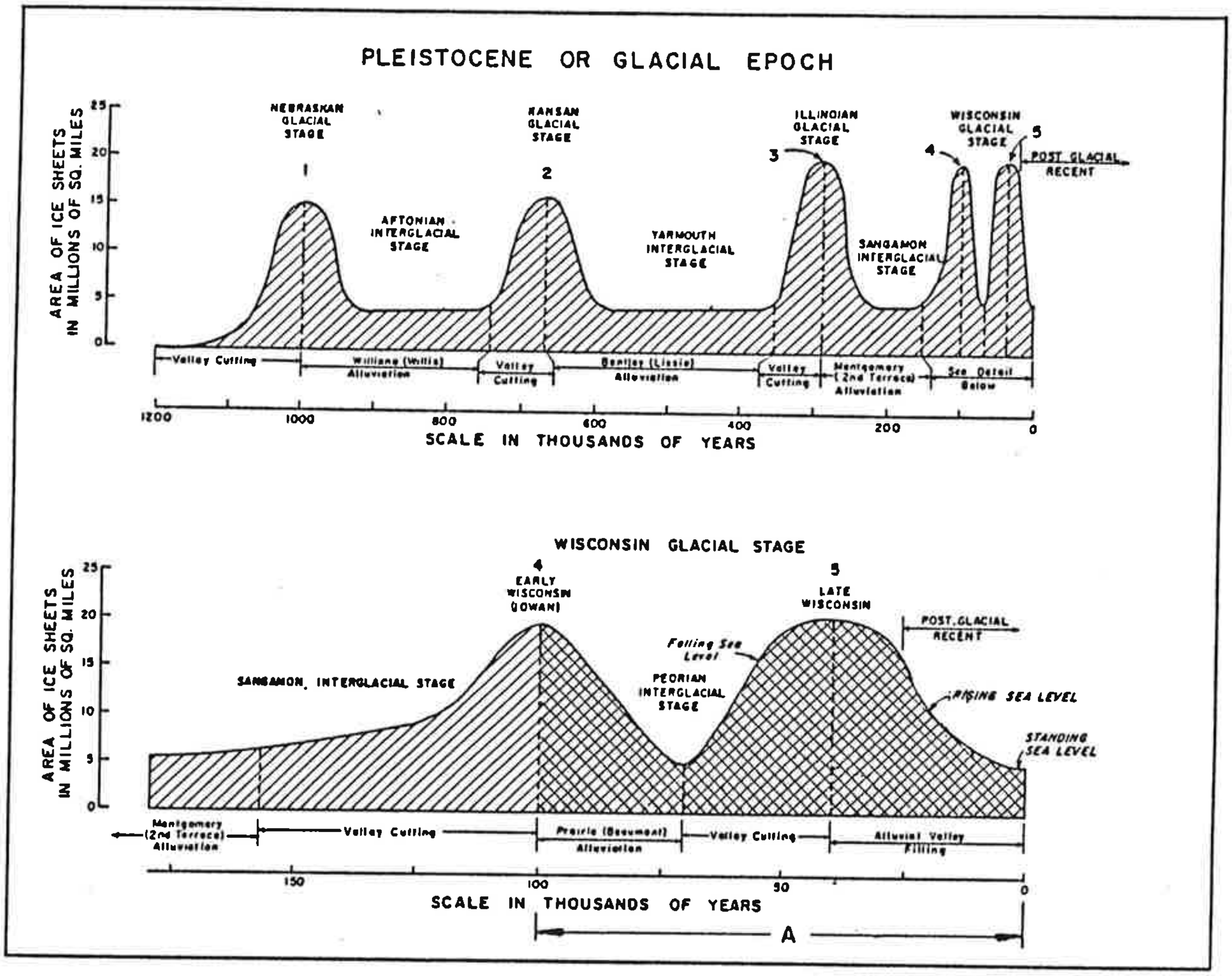

-N-

0 200 400
SCALE IN FEET

LEGEND

| | |
|---|---|
| 5B-11 | Borehole Location |
| P5B-11 | Piezometer Location |

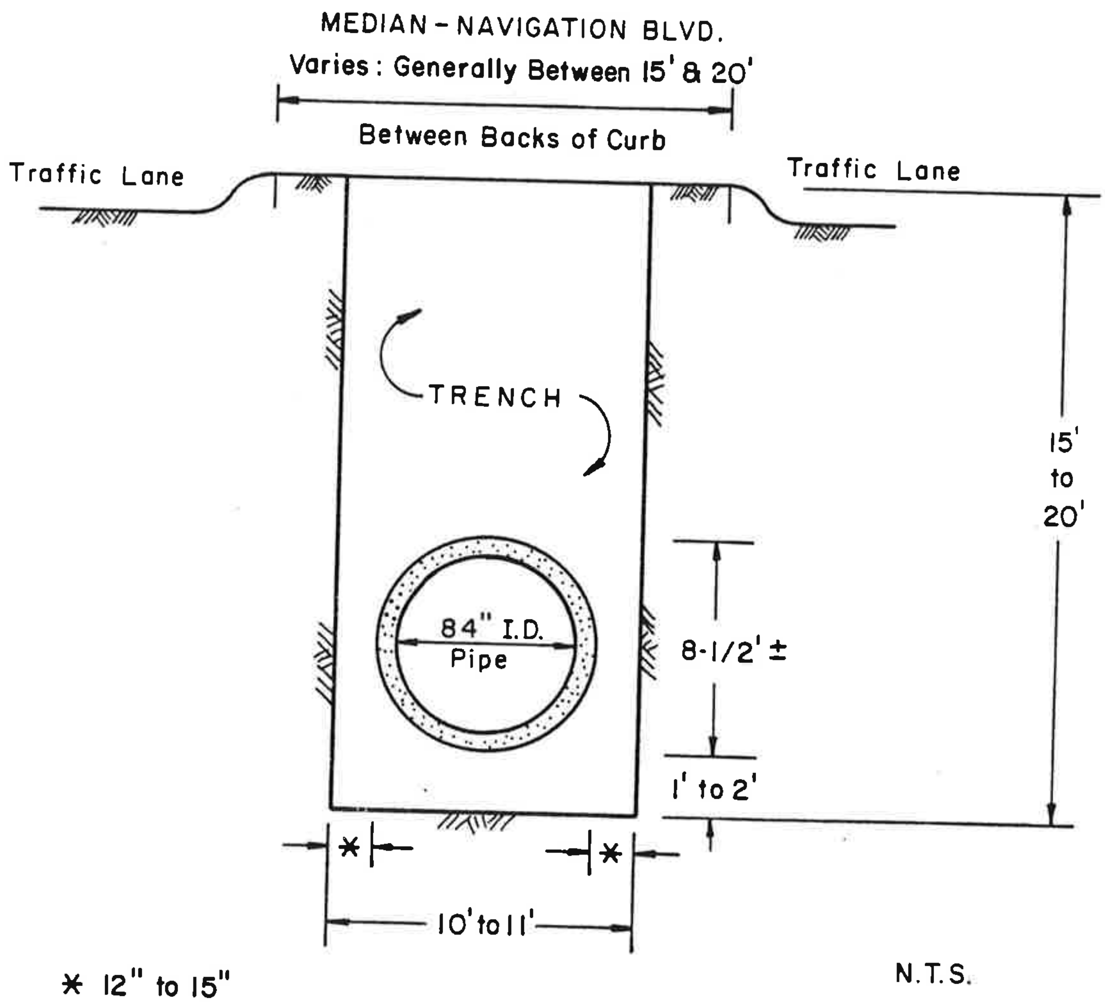
| | | | |
|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | SEGMENT 5B (East-West) BORING LOCATION MAP | FILE No. 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NOTED; MADE BY: R.G. DATE: 5-15-86; CHECKED BY: CVB. DATE: 5-23-86 | | FIGURE 2d |




A-Period of deposition of soils along Design Segment 4A alignment

Correlation of Pleistocene formations of Texas and Pleistocene events (modified after Bernard, et al, 1970).

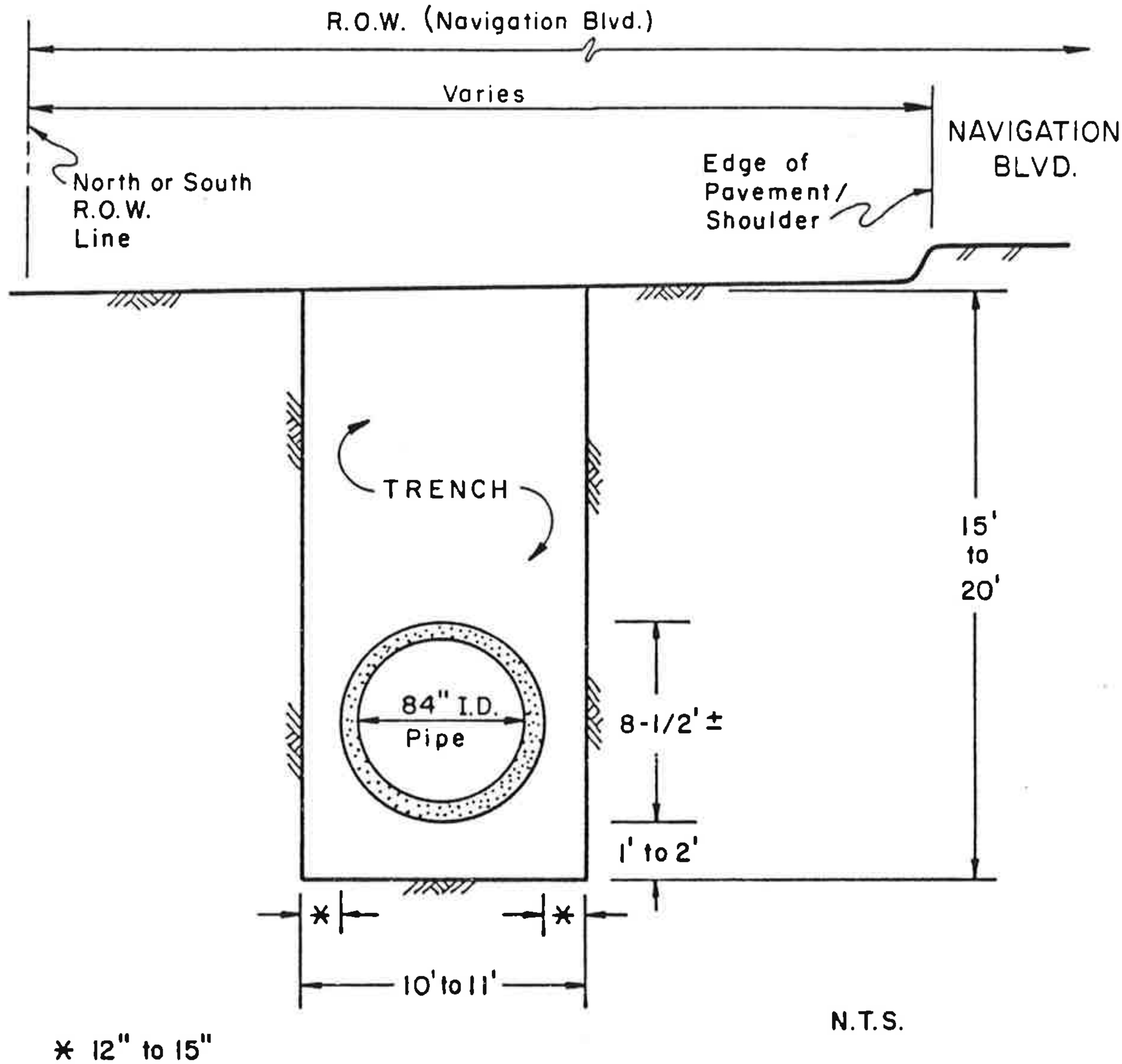
| | | | | |
|---|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | | PLEISTOCENE FORMATIONS AND EVENTS OF TEXAS | FILE No. 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NTS | MADE BY: R. G. DATE: 4-25-86<br>CHECKED BY: CVB. DATE: 5-12-86 | | FIGURE 3 |




ASSUMED TYPICAL DIMENSIONS FOR PIPELINE EXCAVATION AND PLACEMENT BENEATH MEDIAN

DESIGN SEGMENT 5B

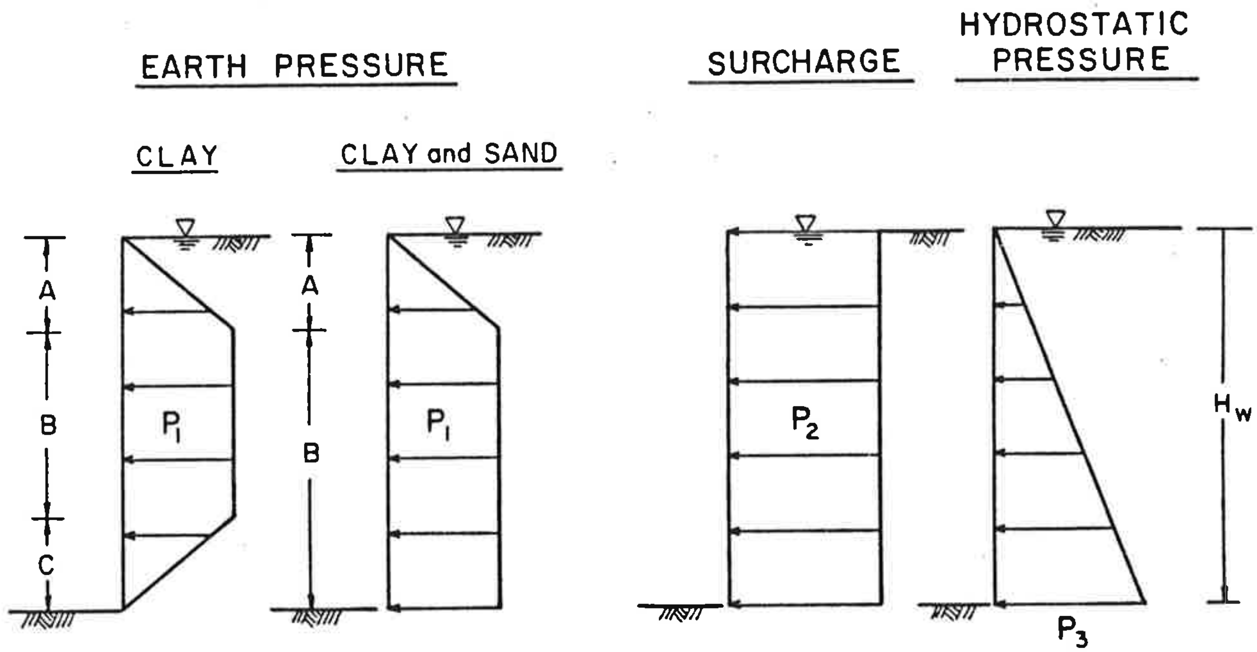
| | | | |
|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | TYPICAL DIMENSIONS FOR PIPELINE EXCAVATION AND PLACEMENT | FILE № 85C621 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NTS; MADE BY R.G. DATE: 4-8-86; CHECKED BY: CVB DATE: 4-15-86 | | FIGURE 4 |




ASSUMED TYPICAL DIMENSIONS FOR PIPELINE EXCAVATION AND PLACEMENT ALONG RIGHT-OF-WAY

DESIGN SEGMENT 5B

| NAME City of Houston East Water Program | Murillo Engineering, Inc. | | TYPICAL DIMENSIONS FOR PIPELINE EXCAVATION AND PLACEMENT | FILE No. 85C6214 |
|---|---|---|---|---|
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NTS | MADE BY R. G. DATE: 4-8-86<br>CHECKED BY: CYB. DATE: 4-15-86 | | FIGURE 4 b |




$\gamma$ (Submerged) = 62.6 pcf

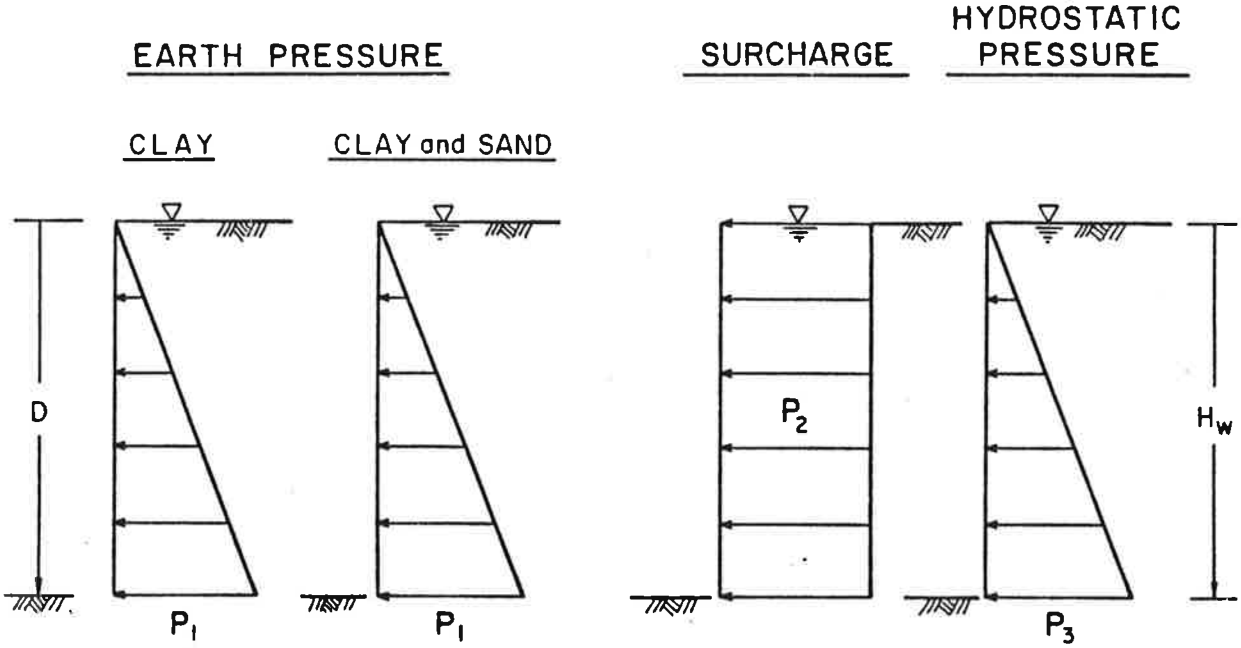
| LETTER | DIMENSION, FT | | PRESSURE | MAGNITUDE, PSF | |
|---|---|---|---|---|---|
| | CLAY | SAND & CLAY | | CLAY | CLAY & SAND |
| A | 5 | 5 | $P_1$ | 375 | 325 |
| B | 10 | 15 | $P_2$* | 500 | 500 |
| C | 5 | - | $P_3$** | $62.4\,H_w$ (1,250) | $62.4\,H_w$ (1,250) |

LOAD ON BRACED WALL = $P_1 + P_2 + P_3$

* Assumed 500 psf construction surcharge
** Value in parenthesis assumes $H_w$ = 20 ft.

## SHORT-TERM LOADS ON BRACED EXCAVATION WALLS
## NAVIGATION BLVD. (DESIGN SEGMENT 5B)

| | | | | |
|---|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | | LOADS ON BRACED EXCAVATION WALLS | FILE No. 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NTS | MADE BY: R.G. DATE: 9-8-86<br>CHECKED BY: CVB. DATE: 4-15-86 | | FIGURE 5 |




$\gamma$ (Submerged)= 62.6 pcf

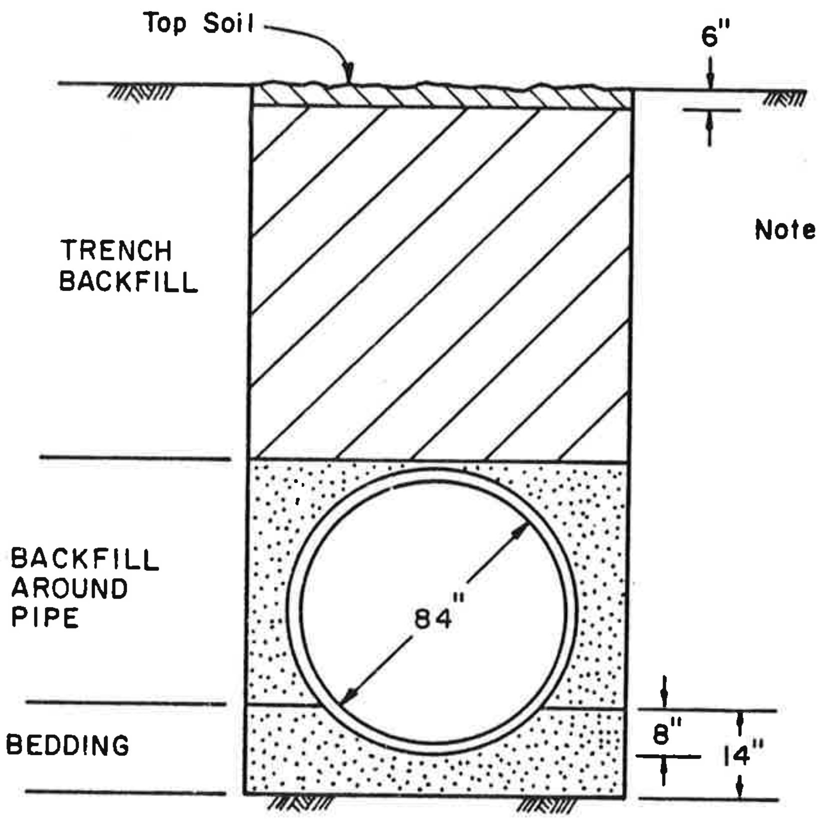
| DIMENSION, FT | PRESSURE | MAGNITUDE, PSF | |
|---|---|---|---|
| | | CLAY | CLAY & SAND |
| $0 \leq D \leq 20$ (20) | $P_1$ | 31.3 D (625) | 26.1 D (525) |
| $0 \leq D \leq 20$ | $P_2$* | 500 | 500 |
| $0 \leq H_w \leq 20$ (20) | $P_3$ | $62.4 H_w$ (1,250) | $62.4 H_w$ (1,250) |

LOAD ON CANTILEVERED WALL = $P_1 + P_2 + P_3$

*Assumed 500 psf construction surcharge

## SHORT-TERM LOADS ON CANTILEVERED EXCAVATION WALLS
## NAVIGATION BLVD. (DESIGN SEGMENT 5B)

| | | | |
|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | LOADS ON CANTILEVERED EXCAVATION WALLS | FILE No 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: N.T.S. — MADE BY: R.G. DATE: 4-15-86 — CHECKED BY: CVB. DATE: 4-15-86 | | FIGURE 6 |




Note: Compacted trench backfill should extend to subbase elevation below surface paving. Within the limits of the vegetated median, the top 6 inches of trench backfill should be top soil.

(AWWA TYPE 3 BEDDING)

RECOMMENDED BEDDING AND BACKFILL FOR
DESIGN SEGMENT 5B

| | | | | |
|---|---|---|---|---|
| NAME City of Houston East Water Program | Murillo Engineering, Inc. | | RECOMMENDED BEDDING AND BACKFILL | FILE No. 85C6214 |
| FOR LOCKWOOD, ANDREWS & NEWNAM | SCALE: NTS | MADE BY: R. G. DATE: 4-25-86<br>CHECKED BY: CVB. DATE: 5-22-86 | | FIGURE 7 |

TABLES

## TABLE I

## DESIGN SOIL PARAMETERS

| SOIL PARAMETER | COHESIVE SOILS | | | GRANULAR SOILS |
|---|---|---|---|---|
| | Clay (Fill) | Clay (Insitu) | Sandy Clay | Sands/ Silts |
| Total Unit Weight (pcf.) | 120 | 120 | 130 | 130 |
| Undrained Shear Strength: | | | | |
| $S_u$ (psf.) | 1,250 | 1,750 | 1,500 | 0 |
| $\phi$ (degree) | 0 | 0 | 0 | 30 |
| Drained Shear Strength: | | | | |
| $c'$ (psf.) | 0 | 0 | 0 | 0 |
| $\phi'$ (degree) | 18 | 20 | 20 | 30 |

Note: Parameters at a specific location may vary somewhat from the values reported in this table.

## TABLE 2

## PIPE LOADS DUE TO TRAFFIC

| Depth to Top of Pipe, Ft | Vertical Pressure, ksf |
|---|---|
| 1 | 1.32 |
| 3 | 0.76 |
| 5 | 0.50 |
| 7 | 0.35 |
| 9 | 0.26 |
| 11 | 0.20 |
| 13 | 0.16 |
| 15 | 0.13 |

## TABLE 3

## PIPE LOADS DUE TO SINGLE RAILROAD

| Depth to Top of Pipe, Ft | Vertical Pressure, ksf |
|---|---|
| 1 | 1.36 |
| 3 | 0.99 |
| 5 | 0.75 |
| 7 | 0.59 |
| 9 | 0.48 |
| 11 | 0.34 |
| 13 | 0.29 |
| 15 | 0.25 |

# APPENDIX A

# BORING LOGS

**murillo engineering, incorporated**
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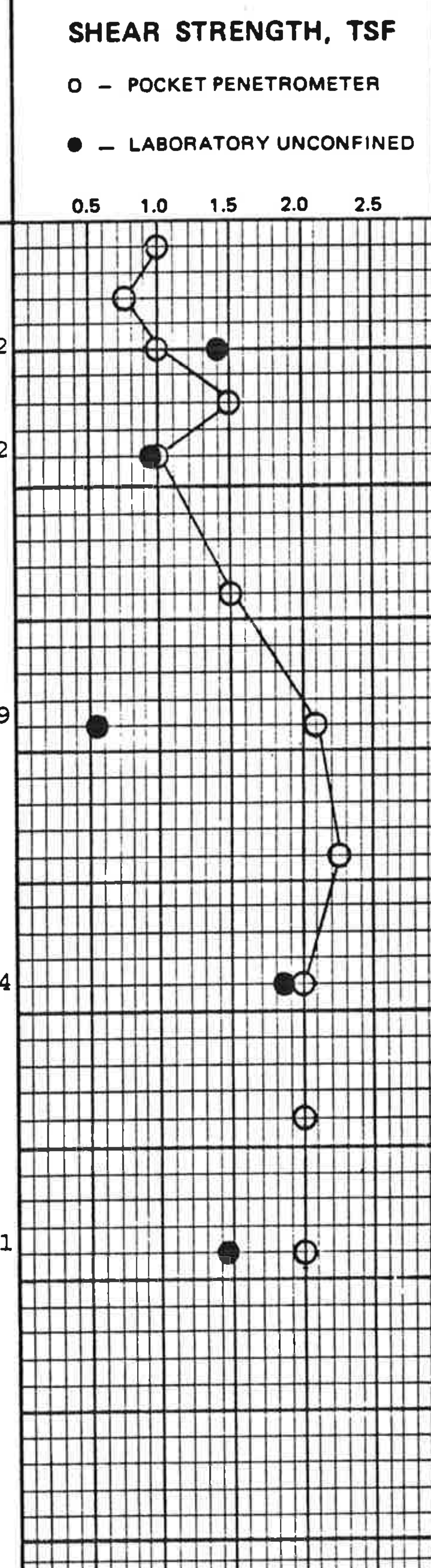
**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-1

**DATE** 4-23-86 **TYPE** 3" Core **LOCATION** Sta.103+76; 9' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | 15" Asphalt and Concrete Pavement | | | | | |
| | | | Dark gray clay | | | | | |
| 5 | | | | | 20 | 112 | 50 | 19 |
| | | | - red and gray with calcareous nodules | | 25 | 102 | 69 | 24 |
| 10 | | | | | | | | |
| 15 | | | | | | | | |
| | | | Gray and tan sandy clay with sand seams | | 15 | 119 | 25 | 14 |
| 20 | | | | | | | | |
| | | | Red and gray clay with sand seams and slickensides | | | | | |
| 25 | | | | | | | | |
| | | | - gray and tan | | 17 | 114 | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| | | | | | 19 | 111 | 61 | 21 |
| 40 | | | BOTTOM AT 40 FEET | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |




BOTTOM AT 40 FEET

1. Bailed boring to 15'-11" upon completion
2. Water level at 6'-11" after 48 hours

**murillo engineering, incorporated**
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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-2

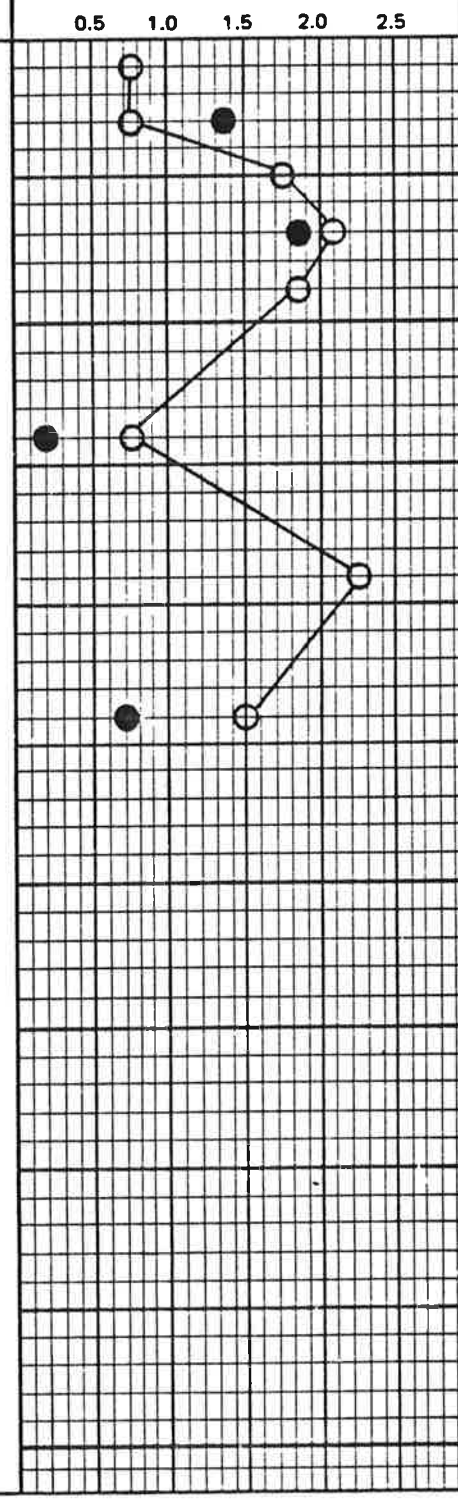
**DATE** 4-17-86 **TYPE** 3" Core **LOCATION** Sta.96+07;104' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray sandy clay with shell; FILL | | | | | |
| | | | Gray and tan clay with calcareous nodules | | 23 | 104 | 67 | 24 |
| 5 | | | | | 24 | 104 | | |
| 10 | | | Gray and tan sandy clay with sand seams | | 18 | 113 | 19 | 16 |
| 15 | | | | | | | | |
| 20 | | | Red and gray clay with ferrous nodules | | 27 | 96 | 79 | 26 |
| 25 | | | BOTTOM AT 25 FEET | | | | | |

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED




BOTTOM AT 25 FEET

1. Bailed boring to 17'-8" upon completion
2. Water level at 12'-9" after 7 days

**murillo engineering, incorporated**

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-3

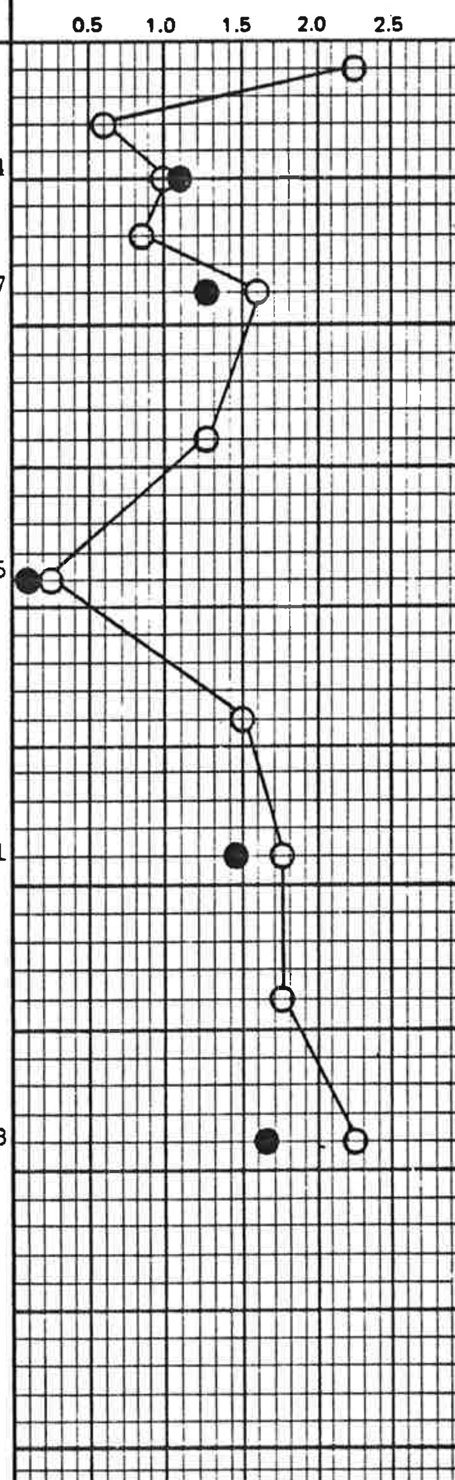
**DATE** 4-16-86 **TYPE** 3" Core **LOCATION** Sta.88+60; 65' L of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray silty clay with shell; FILL | | | | | |
| | | | Gray and tan clay with calcareous nodules | | | | | |
| 5 | | | | | 19 | 114 | 50 | 21 |
| | | | | | 21 | 107 | | |
| 10 | | | | | | | | |
| | | | - gray and brown | | | | | |
| 15 | | | | | | | | |
| | | | Gray and tan clayey silt with sand seams | | 18 | 116 | 20 | 16 |
| 20 | | | | | | | | |
| 25 | | | Red and gray clay with calcareous nodules and sand seams | | | | | |
| | | | - gray and tan | | 22 | 101 | 65 | 23 |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| | | | Gray and red silty clay with calcareous nodules | | 18 | 113 | | |
| 40 | | | | | | | | |
| | | | BOTTOM AT 40 FEET | | | | | |
| 45 | | | 1. Bailed boring to 17'-10" upon completion<br>2. Water level at 11'-1" after 24 hours | | | | | |
| 50 | | | | | | | | |

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



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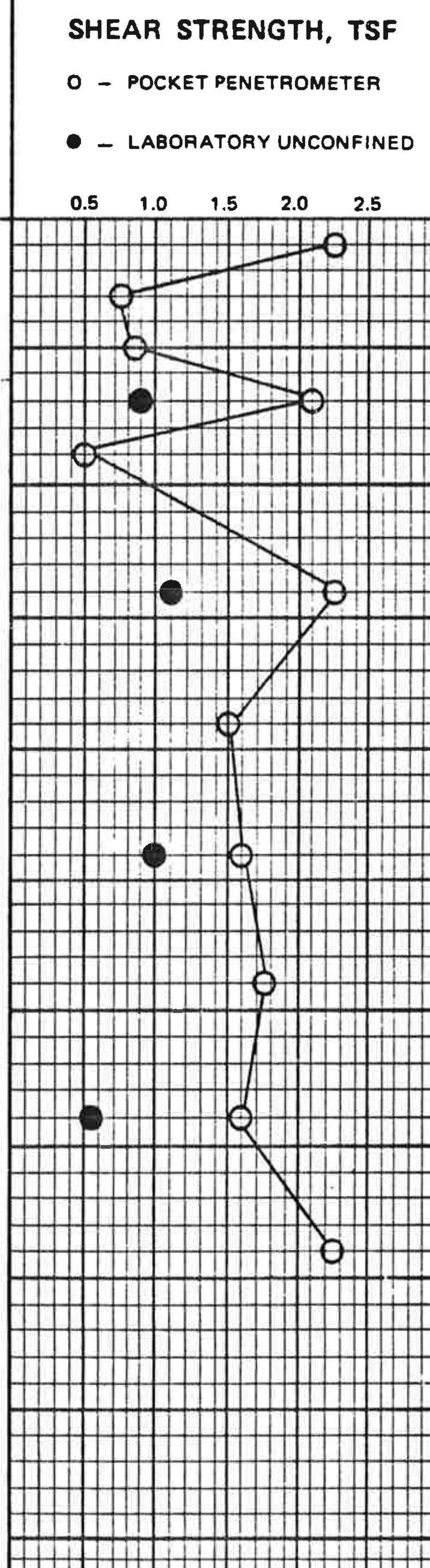
**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-4

**DATE** 4-17-86 **TYPE** 3" Core **LOCATION** Sta.86+13;38' L of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | SHEAR STRENGTH, TSF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray sandy clay; FILL | | | | | | |
| | | | Gray and tan clay with calcareous nodules and ferrous oxide | | | | | | |
| 5 | | | - gray and tan with silt seams | | 20 | 107 | | 53 | 19 |
| 10 | | | | | | | | | |
| 15 | | | - red and gray with slickensides | | 19 | 110 | | 62 | 21 |
| | | | Gray and tan sandy clay with calcareous nodules | | | | | | |
| 20 | | | | | | | | | |
| | | | Red and gray clay with slickensides | | 34 | 90 | | | |
| 25 | | | | | | | | | |
| | | | - gray and tan | | | | | | |
| 30 | | | | | | | | | |
| | | | | | 26 | 100 | | 81 | 26 |
| 35 | | | | | | | | | |
| | | | - gray and brown with slickensides | | | | | | |
| 40 | | | BOTTOM AT 40 FEET | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |




BOTTOM AT 40 FEET

1. Bailed boring to 16'-6" upon completion
2. Water level at 8'-5" after 8 days

*murillo engineering, incorporated*

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-5

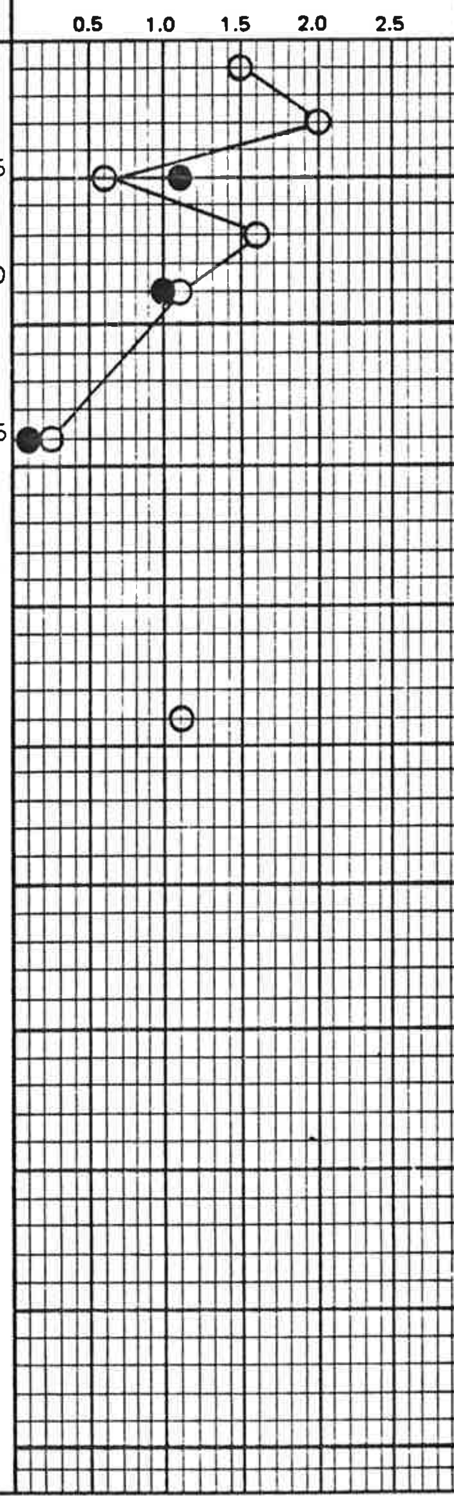
**DATE** 4-16-86 **TYPE** 3" Core **LOCATION** Sta.75+87;58' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | SHEAR STRENGTH, TSF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray silty clay | | | | | | |
| 5 | | | - gray and tan with ferrous oxide | | 21 | 106 | | | |
| | | | | | 22 | 110 | | 47 | 18 |
| 10 | | | - with calcareous nodules | | | | | | |
| | | | Gray and tan clayey sand with sand seams | | 21 | 106 | | 21 | 16 |
| 15 | | | | | | | | | |
| 20 | | | -fine sand (19% Pass #200) | 42 | | | | | |
| | | | Brown and gray silty clay | | | | | | |
| 25 | | | | | | | | | |
| | | | BOTTOM AT 25 FEET | | | | | | |
| 30 | | | 1. Bailed boring to 10'-10" upon completion | | | | | | |
| | | | 2. Water level at 10'-7" after 24 hours | | | | | | |
| 35 | | | | | | | | | |
| 40 | | | | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



*murillo engineering, incorporated*

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-6

**DATE** 4-16-86 **TYPE** 3" Core **LOCATION** Sta.64+92; 121' R of BL

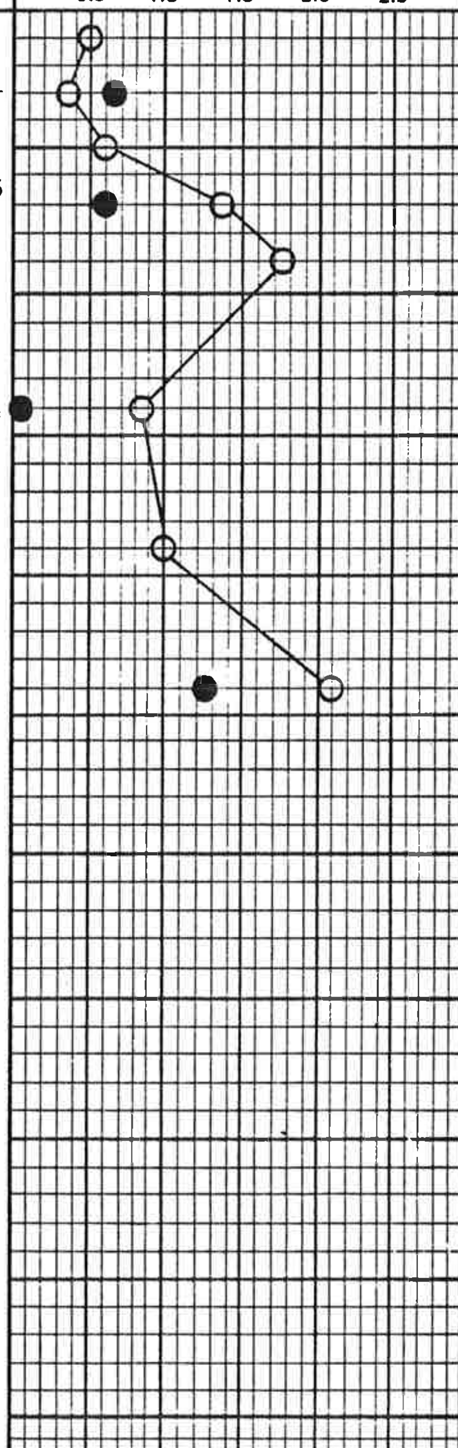
| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray clay | | | | | |
| | | | | | 23 | 101 | 60 | 23 |
| 5 | | | - gray and tan with calcareous nodules | | 27 | 95 | | |
| 10 | | | | | | | | |
| | | | Gray and tan clayey sand | | 17 | 114 | 20 | 16 |
| 15 | | | | | | | | |
| | | | Gray and brown sandy clay | | | | | |
| 20 | | | | | | | | |
| | | | Red and gray clay with calcareous nodules | | 26 | 102 | 58 | 23 |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED

0.5 1.0 1.5 2.0 2.5



BOTTOM AT 25 FEET

1. Bailed boring to 16'-10" upon completion
2. Water level at 12'-4" after 24 hours

**murillo engineering, incorporated**

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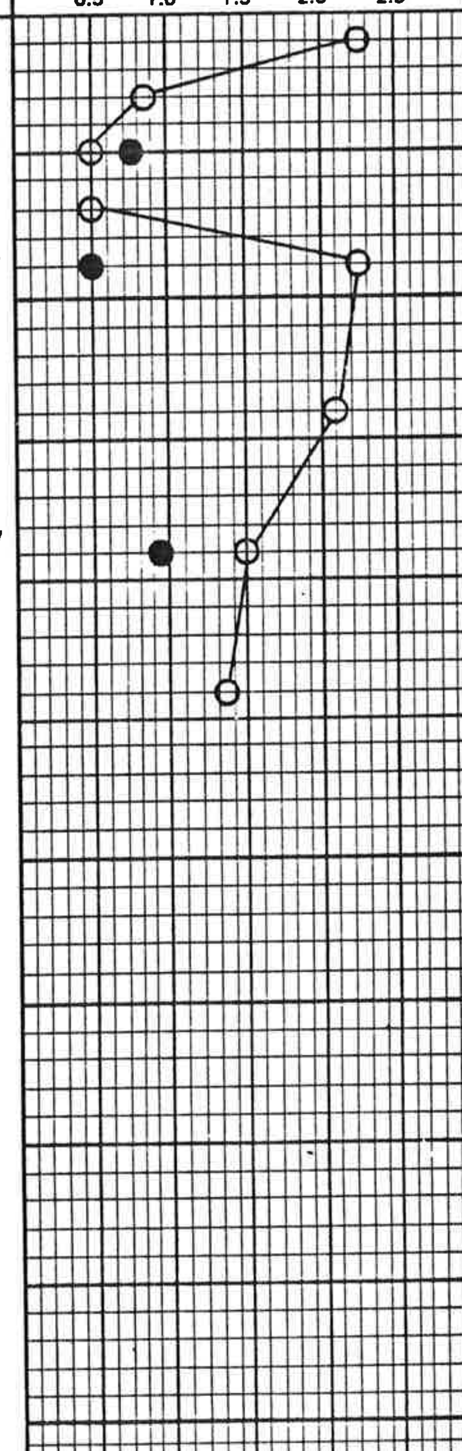
MURILLO

PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B　　BORING 5B-7

DATE 4-16-86　　TYPE 3" Core　　LOCATION Sta.55+43; 95' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | SHEAR STRENGTH, TSF (O – POCKET PENETROMETER; ● – LABORATORY UNCONFINED) 0.5 1.0 1.5 2.0 2.5 | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray and tan clay with organic: FILL | | | | | | |
| | | | Brown and gray clay with - ferrous oxide | | | | | | |
| 5 | | | | | 19 | 108 | | 60 | 24 |
| | | | | | 22 | 105 | | | |
| 10 | | | | | | | | | |
| 15 | | | - red and gray with slicken- sides | | | | | | |
| | | | | | 27 | 97 | | 78 | 24 |
| 20 | | | | | | | | | |
| 25 | | | - tan and gray | | | | | | |
| | | | BOTTOM AT 25 FEET | | | | | | |
| 30 | | | 1. Bailed boring to 18'-7" upon completion | | | | | | |
| 35 | | | 2. Water level at 6'-3" after 24 hours | | | | | | |
| 40 | | | | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |



*murillo engineering, incorporated*

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-8

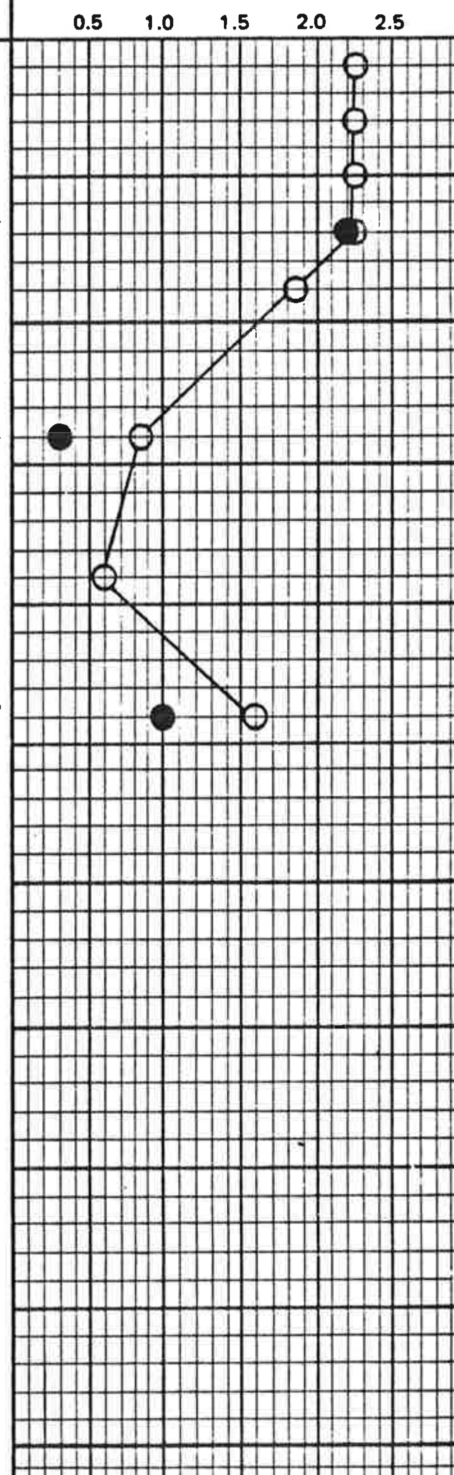
**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.47+39; 24' R of BL

| DEPTH IN FEET | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|
| 0 | Gray and tan silty clay with calcareous nodules; FILL | | | | | |
| | Gray and tan clay with calcareous nodules | | | | | |
| 5 | | | 16 | 111 | 75 | 26 |
| 10 | Gray silty clay with ferrous oxide | | 20 | 111 | | |
| 15 | | | | | | |
| | Gray and tan clay | | | | | |
| 20 | | | | | | |
| | - red and gray | | 21 | 107 | 52 | 22 |
| 25 | | | | | | |

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED




BOTTOM AT 25 FEET

1. Bailed boring to 10'-1" upon completion
2. Water level at 15'6" after 24 hours

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-9

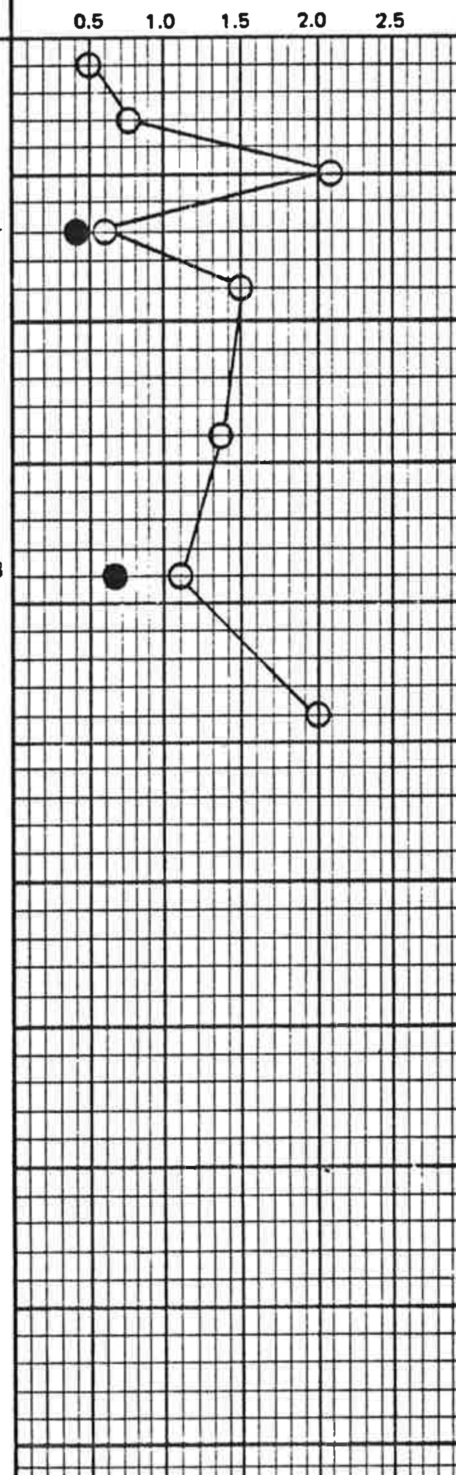
**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.37+50;47' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray clay; FILL | | | | | |
| | | | Gray clay | | | | | |
| 5 | | | - with silt partings | | | | | |
| | | | -red and gray with calcareous nodules | | 31 | 91 | 81 | 27 |
| 10 | | | | | | | | |
| | | | Gray and tan sandy clay with ferrous oxide | | | | | |
| 15 | | | | | | | | |
| | | | Red and gray clay with slickensides | | 28 | 98 | 82 | 29 |
| 20 | | | | | | | | |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED




BOTTOM AT 25 FEET

1. Bailed boring to 14' upon completion
2. Water level at 8'-1" after 24 hours

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-10

**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.27+74; 46' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray silty clay; FILL | | | | | |
| | | | Dark gray clay with sand seams | | | | | |
| 5 | | | | | 23 | 101 | 54 | 20 |
| | | | - gray and tan with calcareous nodules | | | | | |
| 10 | | | | | | | | |
| | | | Gray and tan sandy clay with sand seams | | 15 | 119 | 38 | 16 |
| 15 | | | | | | | | |
| | | | Red and gray clay with calcareous nodules and slickensides | | | | | |
| 20 | | | | | | | | |
| | | | | | 26 | 100 | 69 | 22 |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |

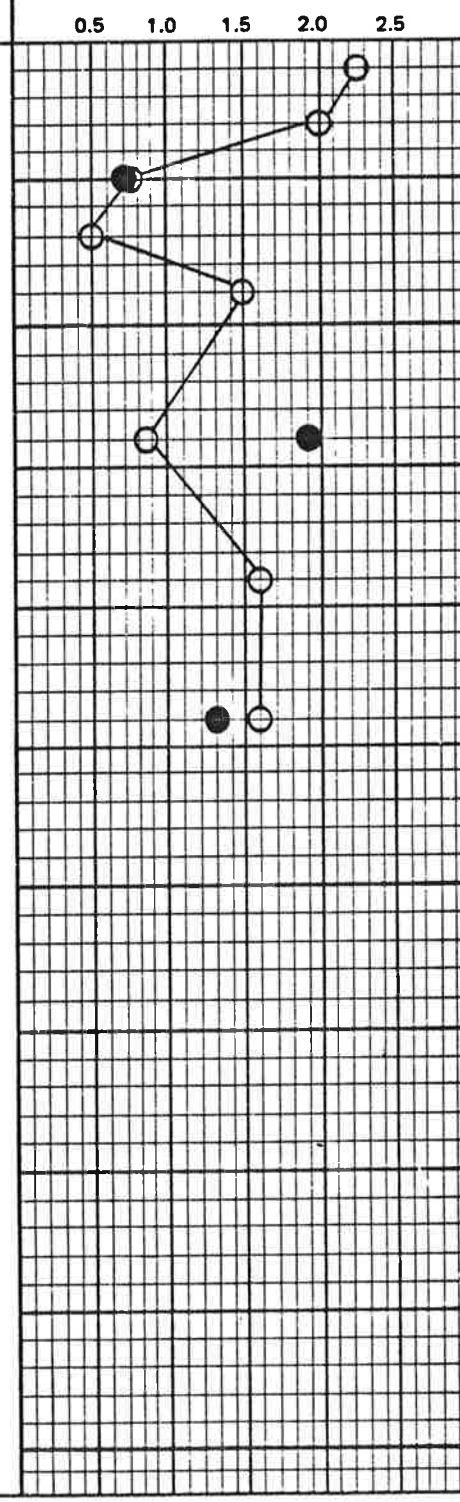
BOTTOM AT 25 FEET

1. Bailed boring to 20'-11" upon completion
2. Water level at 7'-10" after 24 hours

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-11

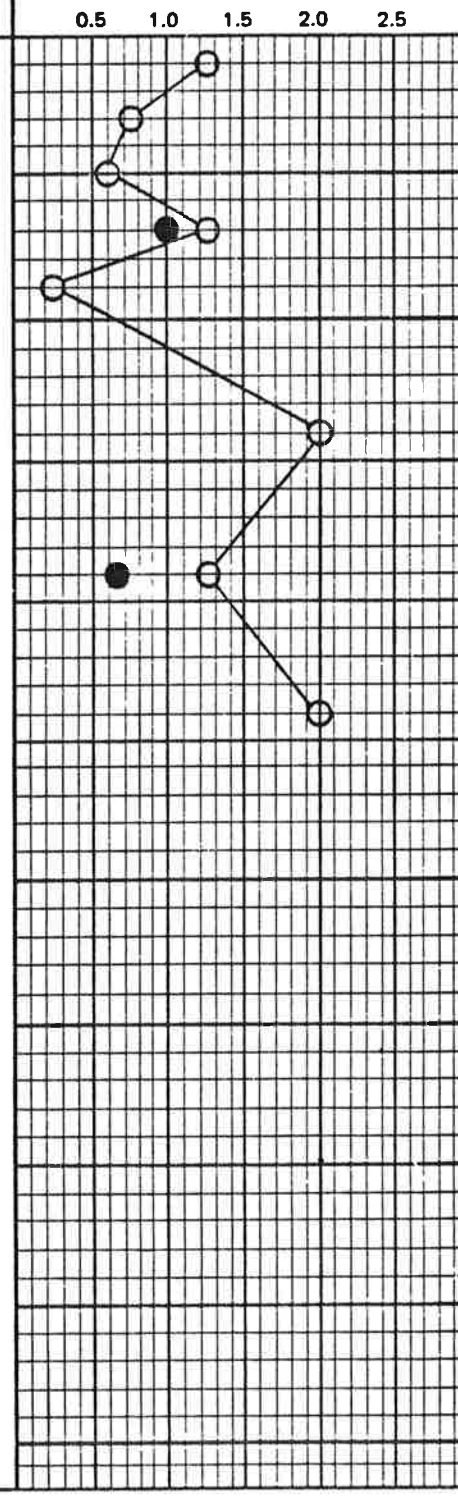
**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.17+70; 13' L of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray silty clay with sand seams; FILL | | | | | |
| | | | Dark gray clay | | | | | |
| 5 | | | | | | | | |
| | | | - gray and tan with calcareous nodules | | 19 | 109 | 52 | 20 |
| 10 | | | | | | | | |
| | | | Gray sandy clay | | | | | |
| 15 | | | | | | | | |
| | | | Red and gray clay with calcareous nodules and slickensides | | 27 | 102 | 66 | 22 |
| 20 | | | | | | | | |
| 25 | | | BOTTOM AT 25 FEET | | | | | |
| 30 | | | 1. Bailed boring to 22'-6" upon completion | | | | | |
| 35 | | | 2. Water level at 9'-0" after 24 hours | | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



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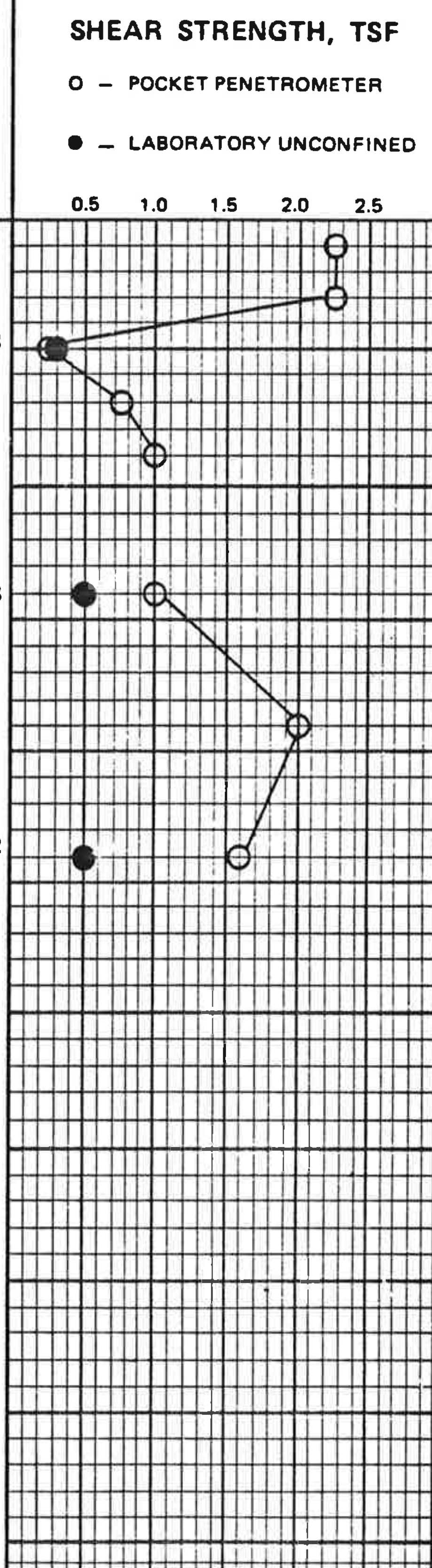
**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-12

**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.9+70;40' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray and tan clay;FILL | | | | | |
| 5 | | | Gray and tan silty clay | | 24 | 98 | 40 | 16 |
| 10 | | | - with calcareous nodules | | | | | |
| | | | Gray and tan sandy clay with sand seams | | 18 | 113 | 26 | 16 |
| 25 | | | Red and gray clay with calcareous nodules and slickensides | | 26 | 102 | 72 | 27 |




BOTTOM AT 25 FEET

1. Bailed boring to 12'-9" upon completion
2. Water level at 8'-0" after 24 hours

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**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-13

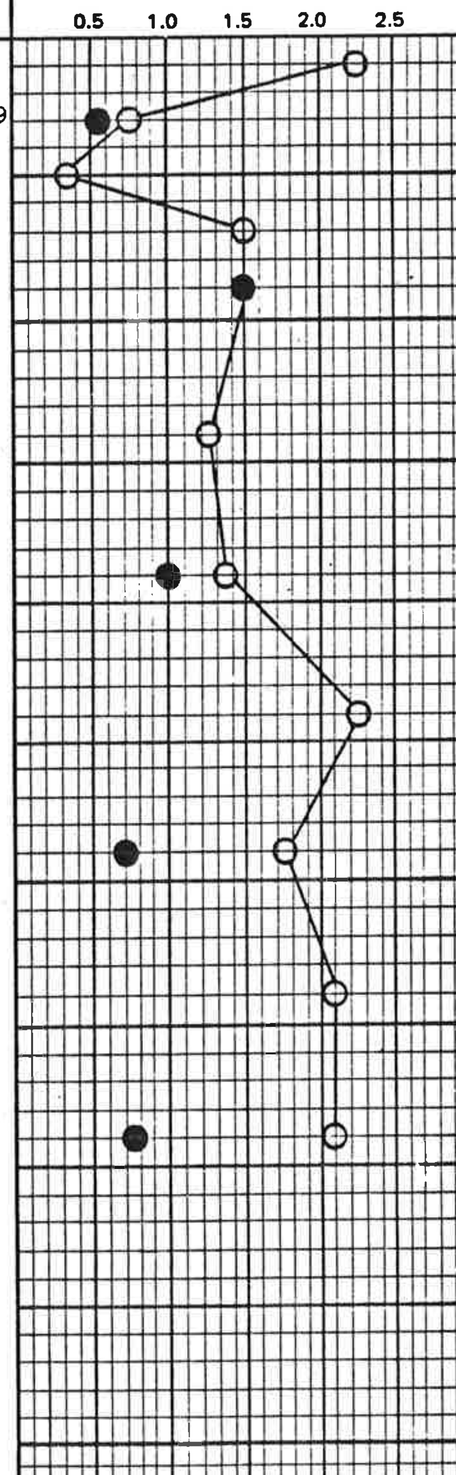
**DATE** 4-15-86 **TYPE** 3" Core **LOCATION** Sta.1+31;34' R of BL

| DEPTH IN FEET | SYMBOL | CORES | DESCRIPTION | BLOWS / FOOT | M.C., % | U.D.W., PCF | LIQUID LIMIT | PLASTIC LIMIT |
|---|---|---|---|---|---|---|---|---|
| 0 | | | Dark gray sandy clay; FILL | | | | | |
| | | | Dark gray clay | | 26 | 99 | 60 | 22 |
| 5 | | | - brown and gray with calcareous nodules | | 23 | 106 | | |
| 10 | | | Gray and tan sandy clay with clay pockets | | | | | |
| 15 | | | Red and gray clay with slickensides | | 27 | 99 | 67 | 22 |
| 20 | | | | | | | | |
| 25 | | | Red and gray silty clay with clay partings | | 22 | 106 | 25 | 16 |
| 30 | | | Tan and gray clay | | | | | |
| 35 | | | - red and gray with calcareous nodules and slickensides | | 19 | 110 | 53 | 22 |
| 40 | | | BOTTOM AT 40 FEET | | | | | |
| 45 | | | 1. Bailed boring to 26'-2" upon completion | | | | | |
| 50 | | | 2. Water level at 12'-11" after 24 hours | | | | | |

**SHEAR STRENGTH, TSF**

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



PIEZOMETER INSTALLATION REPORT

Project Name: CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B

Project No.: 275-85E  Piezometer No. 5B-2

Instrument Location Sta. 96+35; 104' R of BL

Date Installed 4-24-86  By MEI

Time Installed -  Inspector Y. Rahmani

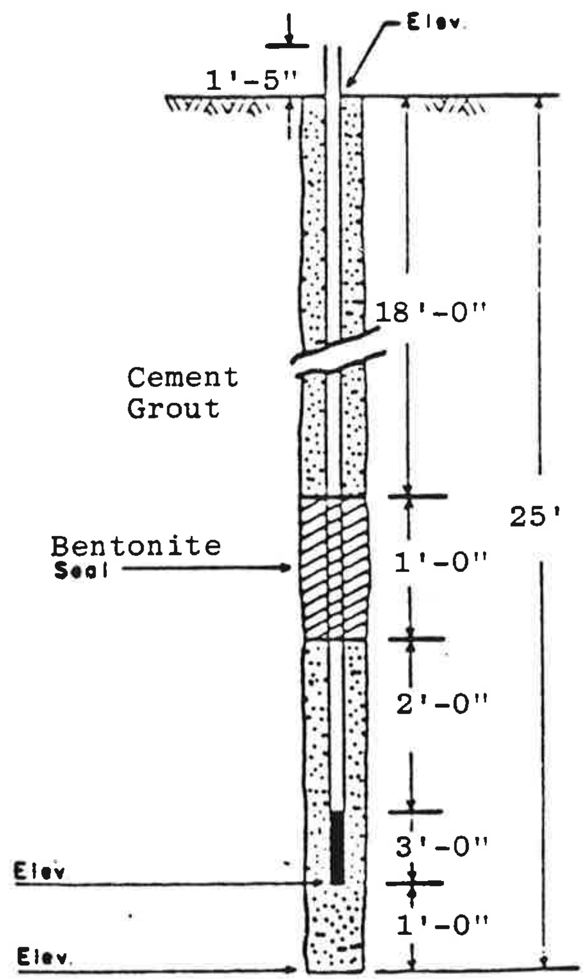
Piezometer Tip $1\frac{1}{2}$" Slotted PVC Pipe

Security: ___ Yes _x_ No  Type $1\frac{1}{2}$" PVC Cap

Lock No. ______

Comments:

- Water Level (W.L.) readings are below ground surface
- Piezometer number is the same as nearest soil boring




Subsoil at Tip Elev.

| Reading Dates | Depth to W.L. |
|---|---|
| 4-25-86 | 14'-0" |
| 4-28-86 | 13'-3" |
| 4-30-86 | 13'-3" |
| 5-6-86 | 13'-0" |

FIG.

PIEZOMETER INSTALLATION REPORT

Project Name: CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B

Project No.: 275-85E  Piezometer No. 5B-11

Instrument Location Sta. 17+70; 13' L of BL

Date Installed 4-25-86  By MEI

Time Installed -  Inspector Y. Rahmani

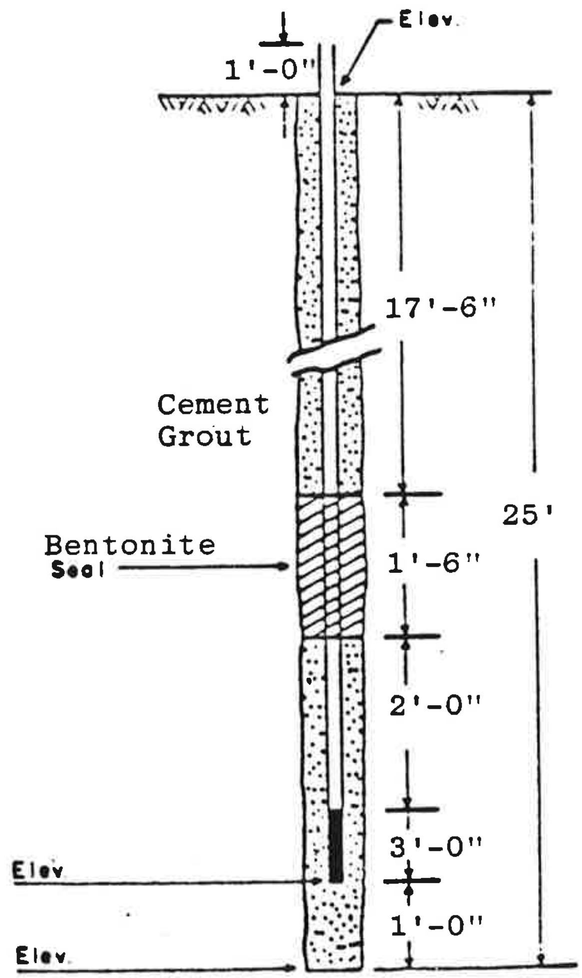
Piezometer Tip 1½" Slotted PVC Pipe

Security: ___ Yes _X_ No  Type 1½" PVC Cap

Lock No. ____

Comments:

- Water Level (W.L.) readings are below ground surface
- Piezometer number is the same as nearest soil boring




Subsoil at Tip Elev.

| Reading Dates | Depth to W.L. |
|---|---|
| 4-28-86 | 7'-9" |
| 4-30-86 | 7'-8" |
| 5-6-86 | 7'-6" |

FIG.

# APPENDIX B

# LABORATORY TEST RESULTS

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TABLE B-1

# SUMMARY OF LABORATORY TEST DATA

**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B

| BORING NUMBER | DEPTH IN FEET | MOISTURE, % | DRY DENSITY, PCF | COMPRESSION, TSF | STRAIN, % | TYPE FAILURE | LAT. PRESSURE | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | SIEVE (-No.200) | CONSOLIDATION | SWELL, % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5B-1 | 4-6 | 20 | 112 | 2.8 | | | | 50 | 19 | 31 | | | |
| | 8-10 | 25 | 102 | 1.9 | | | | 69 | 24 | 45 | | | |
| | 18-20 | 15 | 119 | 1.1 | | | | 25 | 14 | 9 | | | |
| | 28-30 | 17 | 114 | 3.7 | | | | | | | | | |
| | 38-40 | 19 | 111 | 2.9 | | | | 61 | 21 | 40 | | | |
| 5B-2 | 2-4 | 23 | 104 | 2.7 | | | | 67 | 24 | 43 | | | |
| | 6-8 | 24 | 104 | 3.7 | | | | | | | | | |
| | 13-15 | 18 | 113 | 0.4 | | | | 19 | 16 | 3 | | | |
| | 23-25 | 27 | 96 | 1.4 | | | | 79 | 26 | 53 | | | |
| 5B-3 | 4-6 | 19 | 114 | 2.2 | | | | 50 | 21 | 29 | | | |
| | 8-10 | 21 | 107 | 2.5 | | | | | | | | | |
| | 18-20 | 18 | 116 | 0.2 | | | | 20 | 16 | 4 | | | |
| | 28-30 | 22 | 101 | 2.9 | | | | 65 | 23 | 42 | | | |
| | 38-40 | 18 | 113 | 3.3 | | | | | | | | | |
| 5B-4 | 6-8 | 20 | 107 | 1.8 | | | | 53 | 19 | 34 | | | |
| | 13-15 | 19 | 110 | 2.2 | | | | 62 | 21 | 41 | | | |
| | 23-25 | 34 | 90 | 2.0 | | | | | | | | | |
| | 33-35 | 26 | 100 | 1.1 | | | | 81 | 26 | 55 | | | |

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TABLE B-2

# SUMMARY OF LABORATORY TEST DATA

**PROJECT** CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B

| BORING NUMBER | DEPTH IN FEET | MOISTURE, % | DRY DENSITY, PCF | COMPRESSION, TSF | STRAIN, % | TYPE FAILURE | LAT. PRESSURE | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | SIEVE (-No.200) | CONSOLIDATION | SWELL, % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5B-5 | 4-6 | 21 | 106 | 2.2 | | | | | | | | | |
| | 8-10 | 22 | 110 | 2.0 | | | | 47 | 18 | 29 | | | |
| | 13-15 | 21 | 106 | .2 | | | | 21 | 16 | 5 | | | |
| | 18-20 | | | | | | | | | | 19 | | |
| 5B-6 | 2-4 | 23 | 101 | 1.3 | | | | 60 | 23 | 37 | | | |
| | 6-8 | 27 | 95 | 1.2 | | | | | | | | | |
| | 13-15 | 17 | 114 | 0.08 | | | | 20 | 16 | 4 | | | |
| | 23-25 | 26 | 102 | 2.5 | | | | 58 | 23 | 35 | | | |
| 5B-7 | 4-6 | 19 | 108 | 1.5 | | | | 60 | 24 | 36 | | | |
| | 8-10 | 22 | 105 | 1.0 | | | | | | | | | |
| | 18-20 | 27 | 97 | 1.9 | | | | 78 | 24 | 54 | | | |
| 5B-8 | 6-8 | 16 | 111 | 4.4 | | | | 75 | 26 | 49 | | | |
| | 13-15 | 20 | 111 | 0.6 | | | | | | | | | |
| | 23-25 | 21 | 107 | 2.0 | | | | 62 | 22 | 40 | | | |
| 5B-9 | 6-8 | 31 | 91 | 0.8 | | | | 81 | 27 | 54 | | | |
| | 18-20 | 28 | 98 | 1.3 | | | | 82 | 29 | 53 | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |

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TABLE B-3

# SUMMARY OF LABORATORY TEST DATA

PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B

| BORING NUMBER | DEPTH IN FEET | MOISTURE, % | DRY DENSITY, PCF | COMPRESSION, TSF | STRAIN, % | TYPE FAILURE | LAT. PRESSURE | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | SIEVE (-No.200) | CONSOLIDATION | SWELL, % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5B-10 | 4-6 | 23 | 101 | 1.4 | | | | 54 | 20 | 34 | | | |
| | 13-15 | 13 | 119 | 3.8 | | | | 38 | 16 | 22 | | | |
| | 23-25 | 26 | 100 | 2.6 | | | | 69 | 22 | 47 | | | |
| 5B-11 | 6-8 | 19 | 109 | 2.0 | | | | 52 | 20 | 32 | | | |
| | 18-20 | 27 | 102 | 1.3 | | | | 66 | 22 | 44 | | | |
| 5B-12 | 4-6 | 24 | 98 | 0.6 | | | | 40 | 16 | 24 | | | |
| | 13-15 | 18 | 113 | 1.0 | | | | 26 | 16 | 10 | | | |
| | 23-25 | 26 | 102 | 1.0 | | | | 72 | 27 | 45 | | | |
| 5B-13 | 2-4 | 26 | 99 | 1.1 | | | | 60 | 22 | 38 | | | |
| | 8-10 | 23 | 106 | 3.0 | | | | | | | | | |
| | 18-20 | 27 | 99 | 2.0 | | | | 67 | 22 | 45 | | | |
| | 28-30 | 22 | 106 | 1.4 | | | | 25 | 16 | 9 | | | |
| | 38-40 | 19 | 110 | 1.5 | | | | 53 | 22 | 31 | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |

APPENDIX C

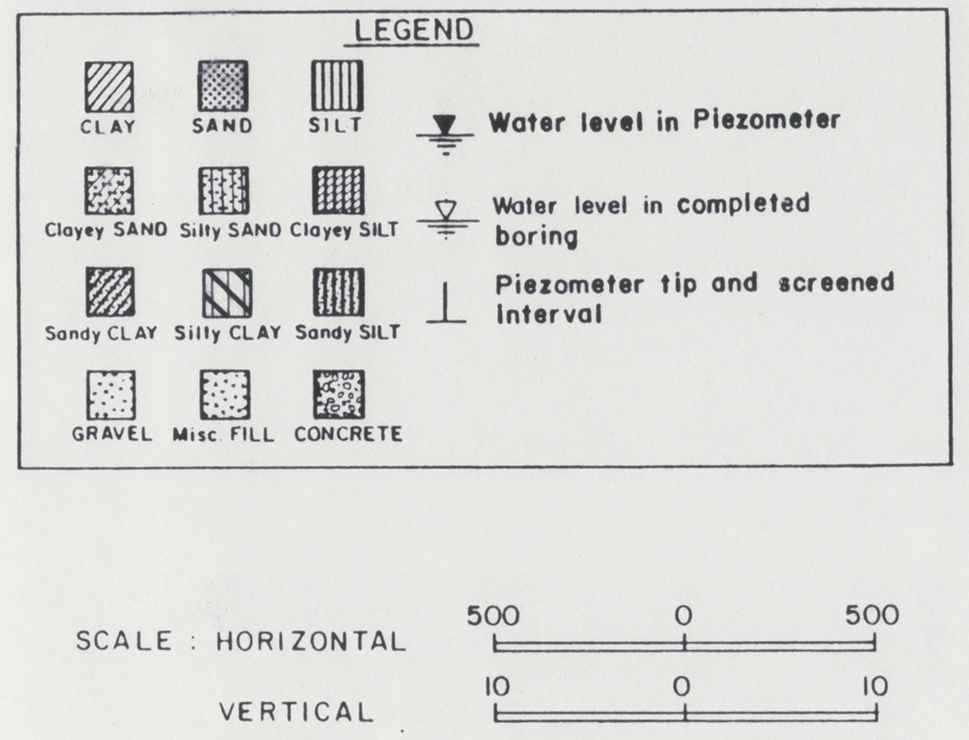
SUBSURFACE PROFILE


GENERAL NOTES

1. The interpretation of stratigraphic conditions along this alignment is based on widely spaced borings, consequently local variation in the stratigraphy should be expected.

2. The water table in this general area is known to vary with climatic conditions. The water table elevations shown on this profile are those measured shortly after each individual boring was completed. Consequently the actual water table at the time of construction should be expected to vary from that shown on this profile.

3. Boring locations are offset from the baseline at various distances. When the horizontal alignment of the pipeline is established the relationship of these boring locations to the pipeline should be determined in order to determine the degree of reliability which may be placed on each boring.




| | | | |
|---|---|---|---|
| NAME CITY OF HOUSTON EAST WATER PROGRAM | Murillo Engineering, Inc. | GENERALIZED SOIL PROFILE | FILE No. 85C6214 |
| FOR LOCKWOOW, ANDREWS & NEWNAM | SCALE: NOTED / MADE BY: J.P.W. DATE: 5-16-86 / CHECKED BY: CVB. DATE: 5-23-86 | SEGMENT 5B | FIGURE C-1 |

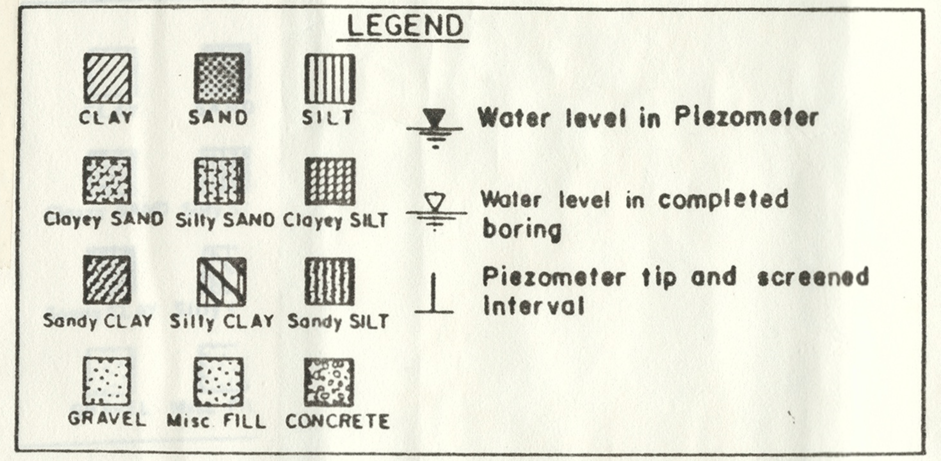

GENERAL NOTES

1. The interpretation of stratigraphic conditions along this alignment is based on widely spaced borings, consequently local variation in the stratigraphy should be expected.

2. The water table in this general area is known to vary with climatic conditions. The water table elevations shown on this profile are those measured shortly after each individual boring was completed. Consequently the actual water table at the time of construction should be expected to vary from that shown on this profile.

3. Boring locations are offset from the baseline at various distances. When the horizontal alignment of the pipeline is established the relative locations of these boring locations to the pipeline should be determined in order to determine the degree of reliability which may be placed on each boring.

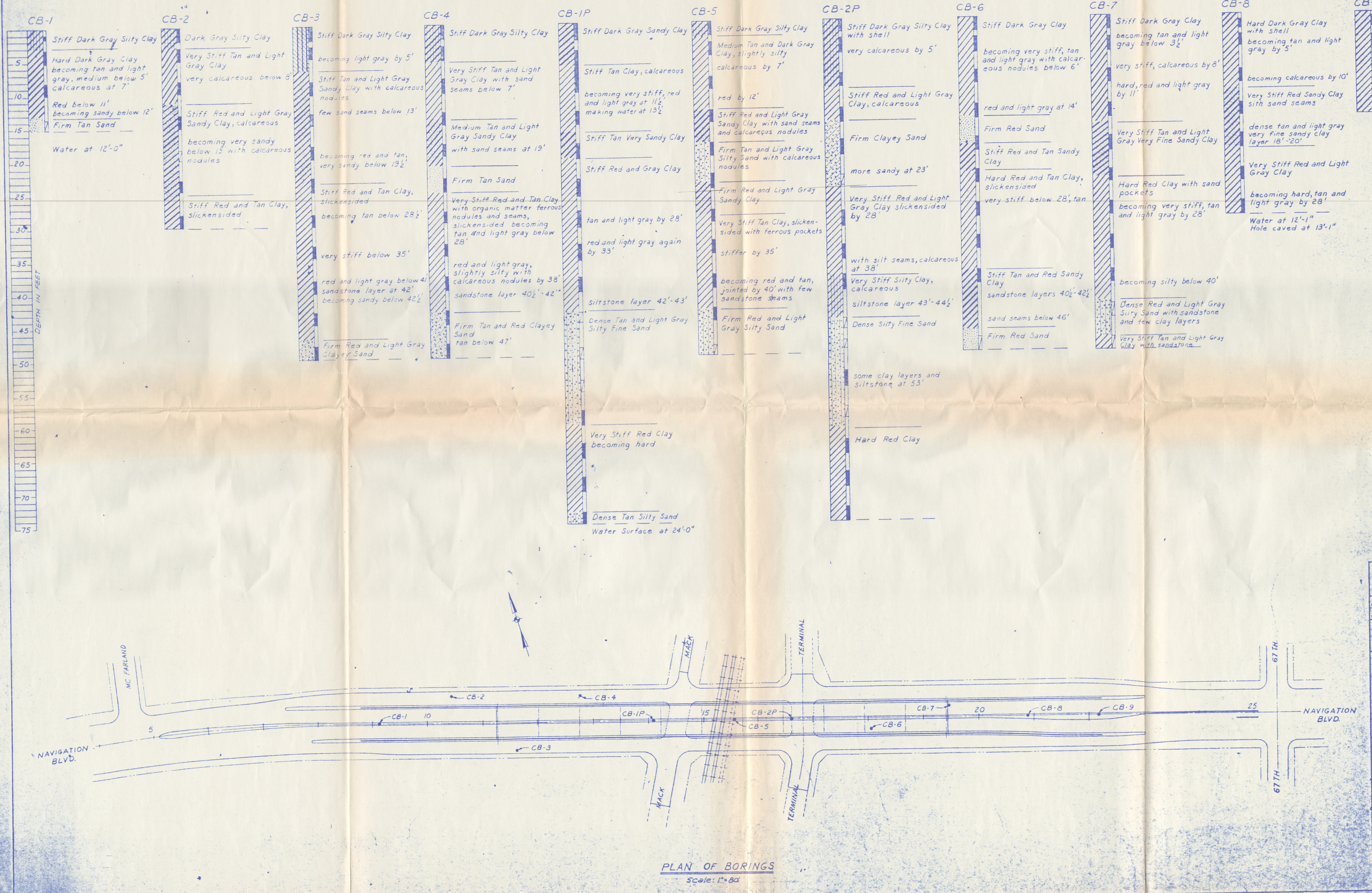
4. Project stationing was arbitrarily chosen to facilitate discussion of borehole locations. Locations and elevations of boreholes were surveyed by Cadastral Surveying.




SCALE : HORIZONTAL 500 0 500

VERTICAL 10 0 10

| NAME CITY OF HOUSTON EAST WATER PROGRAM | Murillo Engineering , Inc. | | GENERALIZED SOIL PROFILE | FILE No. 85C6214 |
|---|---|---|---|---|
| FOR LOCKWOOW, ANDREWS & NEWNAM | SCALE NOTED | MADE BY: J.P.W. DATE: 5-16-86 CHECKED BY: CVB DATE: 5-23-86 | PACKAGE 5B | FIGURE C-1 |




TURNER, COLLIE & BRADEN, INC.
CONSULTING ENGINEERS
HOUSTON PORT ARTHUR

APPROVED [signature]
DATE June, 1963 JOB NO. 7-61

NAVIGATION - HB & T GRADE SEPARATION

SOIL BORINGS

CITY OF HOUSTON
DEPARTMENT OF PUBLIC WORKS

APPROVALS
[signatures]

SUBMITTED G.C.C.
SCALE As Noted