murillo engineering, incorporated
10910 STANCLIFF ROAD • (713) 933-9702 • HOUSTON, TEXAS 77099

GEOTECHNICAL ENGINEERING REPORT

CITY OF HOUSTON - EAST WATER PROGRAM

CONTRACT 5D

REPORT NUMBER

275-85E

REPORTED TO

LOCKWOOD, ANDREWS, & NEWNAM INC.
HOUSTON, TEXAS

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TABLE OF CONTENTS

TABLE OF CONTENTS
(CONTIUNED)

GEOTECHNICAL ENGINEERING REPORT
WATER DISTRIBUTION SYSTEM IMPROVEMENTS
EAST WATER PROGRAM
CONTRACT 5D
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 coneyance 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 Contract 5D improvements. The results of the geologic/ geotechnical characterization and geotechnical analyses for the Contract 5D improvements are presented herein.

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 of the proposed waterline in Contract 5D extends from the intersection of Harrisburg and Dowling Streets, south along Dowling to Bell Street, a distance of approximately 3000 linear feet. A trench of suitable width to accommodate a new 72-inch I.D. open cut water distribution line will likely be excavated along the eastern one-half of Dowling Street, because of the existance of other utility lines along the western half of the street right-of-way.

The line may consist of either prestressed concrete segments, or of welded steel pipe. The operating pressure of the line is presently set at 96 psi, the test pressure 175 psi, and the maximum surge pressure 210 psi. Calculations for joint restraints etc. are based on 210 psi.

The scope of work for the Contract 5D alignment included 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 anchorages, dewatering, bedding and backfill requirements, and other engineering considerations which may impact the proposed construction. The entire line segment is expected to be constructed using open-cut (trenching) techniques, with the possible exception of the single and double railroad crossings at Walker and Rusk Streets respectively which intersect the alignment.

open cut

FIELD INVESTIGATION

An exploratory subsurface investigation along Dowling Street was conducted by MEI in August 1986. The subsurface investigation consisted of 5 borings (5D-1 through 5D-5) drilled to depths ranging from 25 to 40 feet. The spacing of individual borings along the alignment generally varied from approximately 430 to 960 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. One piezometer was also installed near Boring 5D-3 for long-term monitoring of groundwater. Approximate boring and piezometer locations are shown on the Boring Location Map in Figure 1. The locations of the borings related to baseline stationing are provided on the logs of borings. Logs of borings and piezometer data are given in Appendix A. A subsurface profile along the alignment is given in Appendix C. The location and elevation of borings, and the ground surface profile in Appendix C were based on field surveys performed by Geogram in this area prior to our field work and made available to us during this investigation.

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. The results of the laboratory tests are provided in Appendix B and are submitted 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 and B-2. A discussion of Index and Engineering properties (see next paragraph) will be found under Soil Charaterization on page 10.

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 and B-2.

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 surface 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. 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 andgenesis is of importance. Of primary interest is the understanding of glacial and interglacial stages.

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 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, Contract 5D

The stratigraphic unit which outcrops and is present in the subsurface along the route of Contract 5D is the Pleistocene

Beaumont Formation. Soils encountered in the five borings are dominated by high plasticity clays near the surface, underlain by lower plasticity sandy clays. These soils most likely represent overbank flood plain deposits of the deltaic system. No sand strata were found within the depth explored, except a few sand seams at random locations indicated in the boring logs. The Beaumont clays along this alignment are generally stiff to very stiff.

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 Coastal Geosyncline and of greater compaction rates in the offshore area where finer grained sediments have been deposited.

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 trench 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. No growth or salt dome faults were identified which would intersect or otherwise impact Contract 5D.

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 have little bearing on the project. Man's activities, however, have served to accelerate ground movements that may impact the project.

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. Piezometer 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 and is not subject to reversing of flow, as is the case with a system designed for gravity flow.

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 Charactierization

The following paragraphs describe our interpretation of the subsurface soil and associated laboratory data along the alignment of Contract 5D. This interpretation of subsurface conditions was made in order to provide LAN with general geotechnical parameters pertinent to this alignment for use in the 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. Pleistocene soils (the Beaumont Formation) were encountered throughout the entire alignment from the ground surface to the termination depth of borings. The soils within the Beaumont Formation in this area generally consist of interbedded layers of stiff to very stiff clay, silty clay, and sandy clay. Clay and silty clay soils encountered in the upper 10 to 15 feet along the alignment had measured undrained shear strengths which ranged from 1000 to 2600 psf and plasticity indices which ranged from 32 to 55. Below a depth of 10 to 15 feet the sandy clay soils had measured undrained shear strengths on the order of 700 to 2500 psf and plasticity indices which ranged from 6 to 31.

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

Groundwater Conditions. The depth to the groundwater table measured in the borings 24 hours after completion of drilling and in the piezometer 17 days following installation, ranged from 14 to 15 ft, as shown in the borings logs and Piezometer Report (Appendix A),and the interpreted profile (Appendix C). It should be expected to vary with changes in environmental conditions, such as frequency and magnitude of precipitation, and the time of year that construction takes place.

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 each end of the line segment;
- Thrust reactions and vertical anchorage at railroad tracks (Sta. 21+00 and Sta. 23+75) and at other locations where existing utilities or other obstructions may intersect the alignment.

Excavation for Pipeline Placement

It is assumed that the excavation for the 72-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 the pipes, 1 to 2 feet of over excavation for bedding placement, and 6 to 10 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.

The existing segment of Dowling Street in Contract 5D is a four-lane undivided street with curbs and gutters, and storm drain inlets to provide surface drainage. The street has a concrete pavement section within the 80 foot right-of-way.

The two-way traffic volume is fairly high in this important thoroughfare.

The location of the propsed 72-inch pipeline within the street right-of-way and the method of installation are not known at this time. Review of available utility drawings indicates that the east side of the street is less conjested with underground obstructions. However, unless traffic is re-routed through adjacent streets, a shored or braced excavation with vertical walls is in our opinion the most practical method of maintaining an open excavation for placement of the pipeline. Either a cantilevered sheet piling or internally braced shoring system may be used. 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 is provided on Figure 3 . A similar earth pressure diagram for a cantilevered piling system is provided in Figure 4 . The pressures indicated on these figures have assumed a typical 500 psf construction surcharge and no dewatering. The actual amount of surcharge will depend on the contractors operations, i.e., if the contractor plans to store more than 2 or 3 feet of excavation soil 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.

Dewatering

The ground water table at the time of our field exploration was found consistantly at a depth of approximately 14 to 15 feet. The soil profile as indicated elsewhere in this report and in the attached boring logs, consists of mostly stiff clays in the upper 15 feet, underlain by stiff sandy clays to the maximum depth of exploration of 25 or 40 feet. If the proposed pipe is installed at 15 to 20 feet depth, little ground water should be encountered, and such that the contractor will be able to control by means of a sump pump installed within each excavated trench segment. If a dewatering system is found necessary to maintain to ground water level at least 2 feet below the bottom of excavation, common available methods for temporary dewatering in clayey soil strata utilize either vacuum well points or eductor wells. The former can lower the water table to a depth of 15 to 18 feet, while the latter are used for depths greater than 18 feet.

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 large size (72 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. The same bedding may also be used under steel or other types of pipes. A typical section showing this recommended bedding and backfill is provided on Figure 5. 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 around the pipe should be placed in lifts not exceeding 6 inches and be well tamped.

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, provided that no sluicing or flooding is used for backfill densification.

Excavation spoil or other select material used to backfill the trench above the pipe should be placed in lifts not exceeding 12 inches loose thickness and be compacted to approximately 95 percent of Standard Proctor density, whether the trench is located under paved areas or not. Control of backfill density where a trench is located in vegetated areas

Can not excavation spoil

is less critical and a lesser density is normally specified. However, in the case of this line segment, because of the frequent presence of street intersections, access lanes to commercial properties, other surface improvements, and the uncertainty with regard to the type and limits of future pavement along Dowling Street, we believe that the same type of backfill density control should be specified throughout this contract.

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 5D may partially be located beneath street pavements and therefore will be subjected to loads at these locations resulting from vehicular traffic. The alignment for Contract 5D also passes beneath a single and a double railroad track at approximately sta. 21+00 and 23+75 respectively.

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. 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 at this time throughout the length of this design segment. The only known major horizontal unbalanced hydrostatic force will be associated with hydrotesting of the new 72-inch I.D. pipeline. If either the north or south end of this line segment is plugged and hydrotested prior to connection to the next segment, the unbalanced hydrostatic force generated at the end of the alignment will be approximately 855,000 pounds. This assumes that the pipeline is tested at a maximum surge pressure of 210 psi. The basis of calculations of thrust forces, thrust blocks, and length of restraint joints is R.J. Carlsen's paper entitled "Thrust Restraint for Underground Piping Systems" attached herewith as Appendix D.

In order to provide a reaction to this unbalanced axial force, the force must be distributed to the surrounding soil in a manner that does not overstress the soil. Commonly used methods of distributing this thrust force to the soil are through the use of concrete thrust blocks, restrained joints (in the case of jointed concrete pipe), or piles. If a concrete thrust block is utilized, it is estimated that it will have to provide a bearing area of 175 square feet to resist the 855,000 pound axial force generated in the pipeline.

This assumes 10 feet of cover will be placed over the pipeline. With a minimum recommended 6 ft cover the required bearing area increases to 207 ft^2. After this line segment is connected to the next segment the thrust block would no longer be required.

Restrained joints are sometimes used in place of thrust blocks to resist thrust forces generated at bends and elsewhere where the passive resistance of the soil in contact with the pipe is mobilized as well as frictional resistance between the soil and the outside surface of the pipe. The passive resistance of the soil is actually the major force resisting the unbalanced thrust force at the bend. In the case of axial loads however, as is the case discussed here, where passive resistance cannot be developed, the total length of restrained joints required to mobilize sufficient frictional resistance against the axial thrust force may be considerable, depending on roughness of the exterior surface of the pipe. Continuous welded steel pipes on the other hand, except for relatively short lengths, can provide sufficient length in contact with the adjacent soil to develop the required frictional resistance to the thrust force. For example, assuming 10 feet of well compacted backfill over the pipe, sand around the pipe and friction angle of 28 degrees between the pipe and the sand, the estimated length of pipe required to mobilize sufficient frictional resistance is 114 ft. This length is inversely proportional to the amount of cover over the pipe in the case of dead-end calculations. Thus if the cover is reduced to 5 ft from 10 ft, the length becomes 228 ft or twice as large etc. (See Appendix D for method of estimating size of thrust blocks).

There are at least two locations known along the proposed pipeline alignment at which vertical bends may be required to allow crossing under existing obstructions. They are:

- The railroad tracks crossing the proposed alignment at approximate Sta. 21+00 (Walker Street) and at Sta. 23+75 (Rusk Street);

Where vertical changes in the pipeline alignment occur, an unbalanced hydrostatic force is developed as with horizontal bends in a 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 depth and angle of deflection is not known 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. 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:

- Properties of the in situ soil at and below the bearing surface of the thrust block;
- Depth to the bearing surface; and
- 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 from the horizontal may be enhanced by the use of driven friction piling, straight shafts, or drilled underreamed shafts.

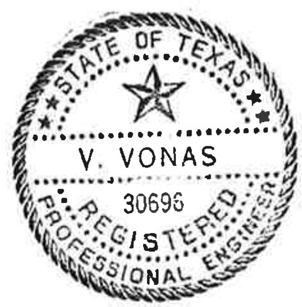
For a downward resultant force at vertical bends and a bearing surface between 15 and 25 feet below the ground surface, thrust blocks, in our opinion, may be designed for an allowable bearing capacity (F.S.=2) between 1000 and 7000 psf where the thrust blocks bear on clay.

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 for horizontal bends with respect to soil pipe friction presented in the Carlsen paper. 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 Contract 5D improvements to the distribution system of the expanded East Water Purification Plant. Subsurface data collection (soil borings), laboratory testing and report preparation were conducted by Murillo Engineering, Inc.



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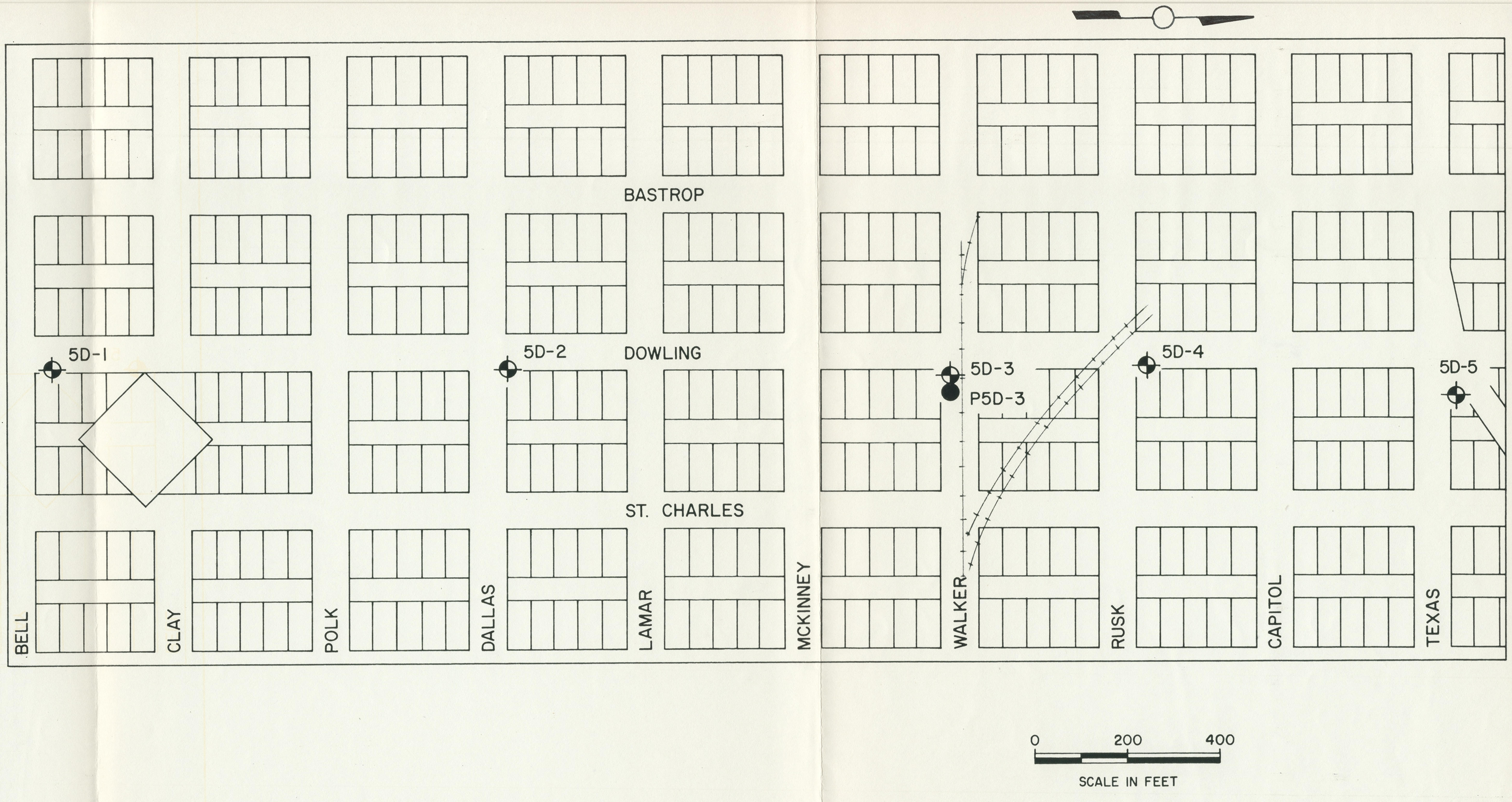
FIGURES

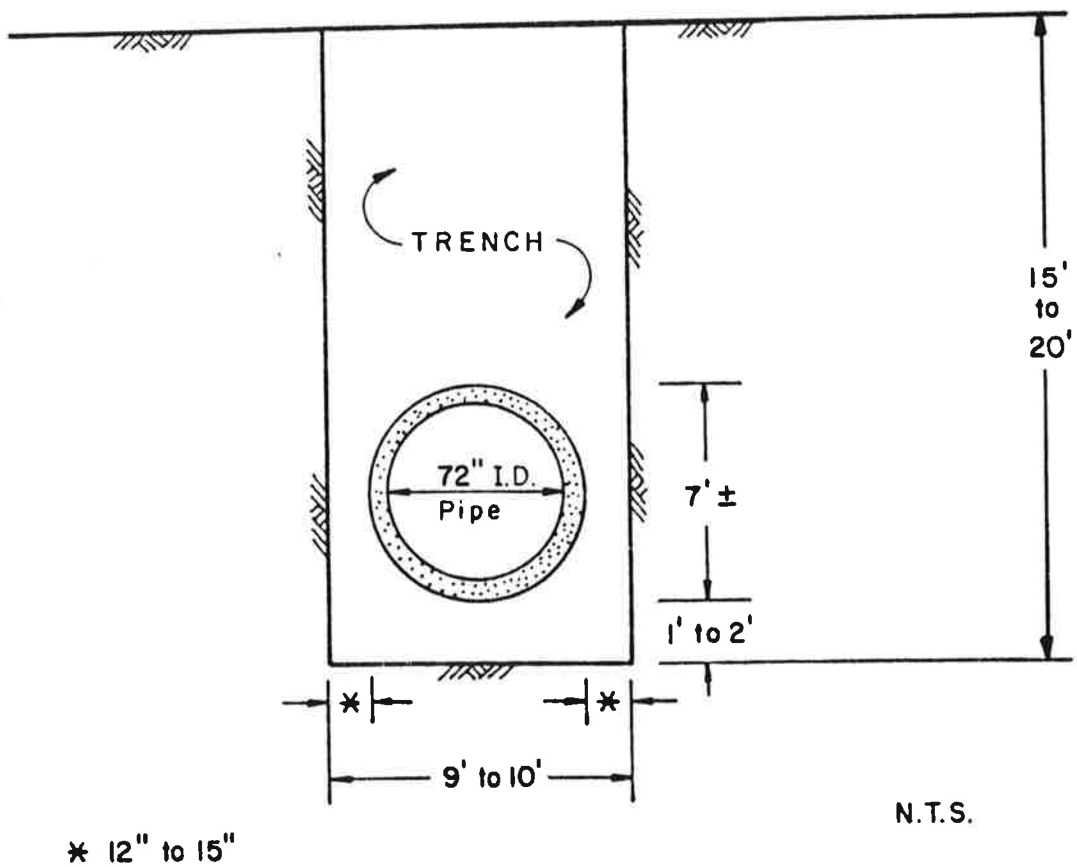


LEGEND

5D-3 BOREHOLE LOCATION

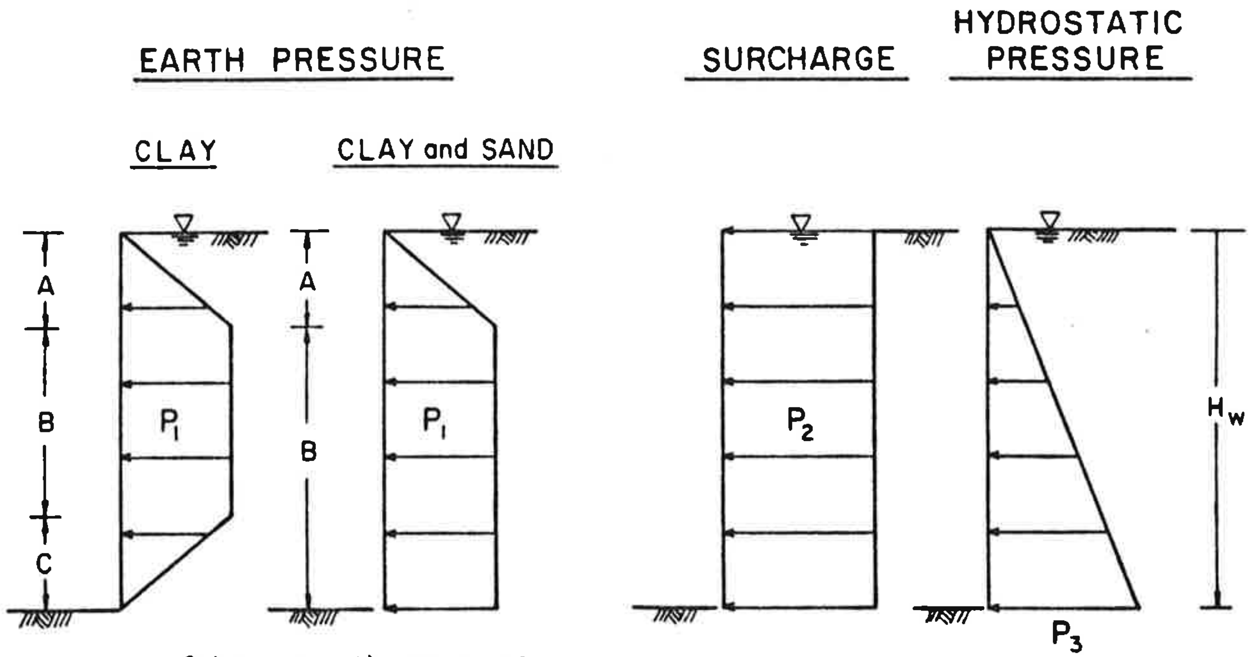
P5D-3 PIEZOMETER LOCATION

NAME City of Houston East Water Program	Murillo Engineering, Inc.	PACKAGE 5D BORING LOCATION MAP	FILE Nº 275-85E
FOR LOCKWOOD, ANDREWS & NEWNAM	SCALE NOTED / MADE BY: J.H. DATE: 8-29-86 / CHECKED BY: V.H.V. DATE: 9-2-86		FIGURE 1



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

FIGURE 2



γ (Submerged) = 62.6 pcf

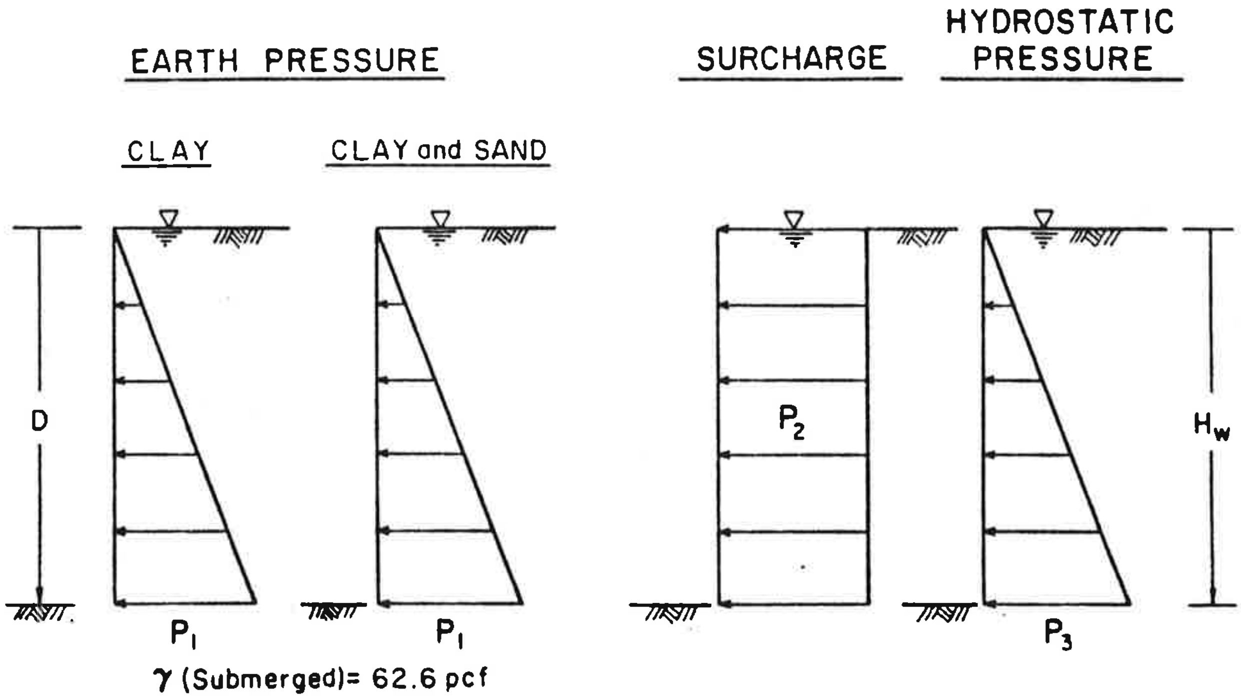
LETTER	DIMENSION, FT		PRESSURE	MAGNITUDE, PSF	
	CLAY	SAND & CLAY		CLAY	CLAY & SAND
A	5	5	P_1	375	325
B	10	15	P_2*	500	500
C	5	-	P_3**	62.4 H_w (1,250)	62.4 H_w (1,250)

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

FIGURE 3



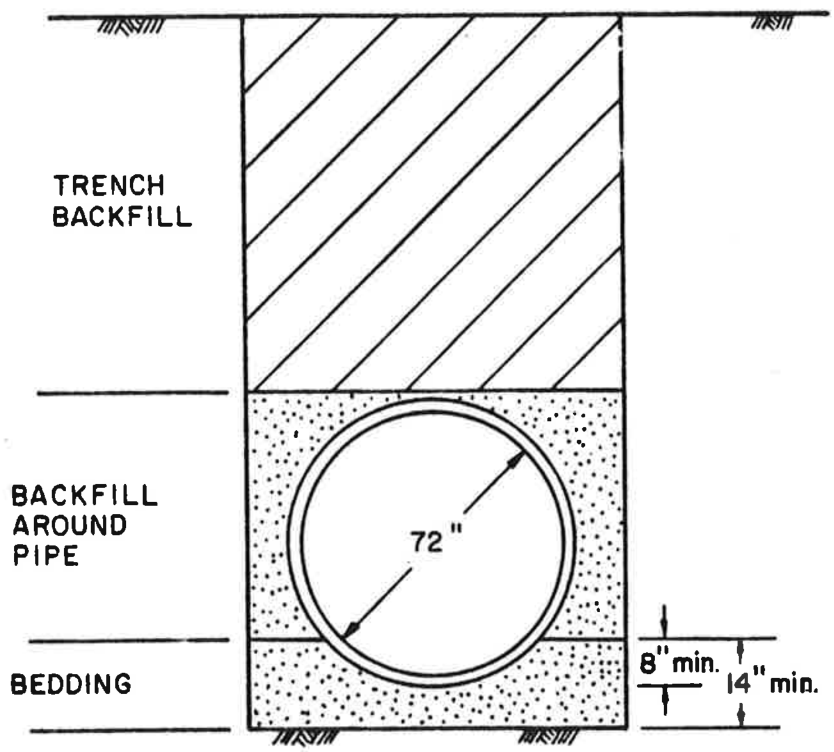
DIMENSION, FT	PRESSURE	MAGNITUDE, PSF	
		CLAY	CLAY & SAND
$0 \leq D \leq 20$ (20)	P_1	31.3 D (625)	26.1 D (525)
$0 \leq D \leq 20$	P_2*	500	500
$0 \leq H_w \leq 20$ (20)	P_3	$62.4 H_w$ (1,250)	$62.4 H_w$ (1,250)

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

*Assumed 500 psf construction surcharge

SHORT-TERM LOADS ON CANTILEVERED EXCAVATION WALLS

FIGURE 4



(AWWA TYPE 3 BEDDING)

RECOMMENDED BEDDING AND BACKFILL

FIGURE 5

TABLES

TABLE I

DESIGN SOIL PARAMETERS
(FOR DESIGN OF BRACED EXCAVATIONS)

SOIL PARAMETER	TYPE OF SOIL: Clay (Insitu)	Sandy Clay
Total Unit Weight (pcf)	120	130
Undrained Shear Strength:		
S_u (psf.)	1500	1200
ϕ (degree)	0	0
Drained Shear Strength:		
c' (psf.)	0	0
ϕ' (degree)	20	20

Notes:
- Parameters at a specific location may vary somewhat from values reported in this table.
- S_u and c' are cohesive shear strengths of clay
- ϕ and ϕ' represent the angle of internal friction of soil particles
- Undrained shear strengths are short term values obtained in the laboratory
- Drained shear strengths are long-term parameters which were assumed based on experience with similar soils.

TABLE 2

PIPE LOADS DUE TO TRAFFIC

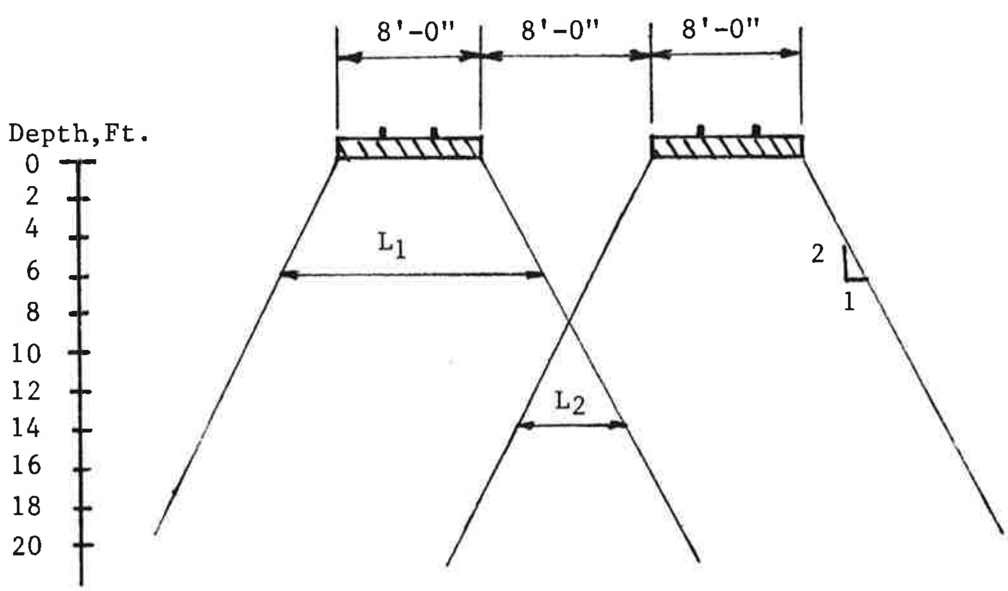
Depth to Top of Pipe, Ft.	Vertical Pressure, ksf
1	1.32
3	0.76
5	0.50
7	0.35
9	0.26
11	0.20
13	0.16
15	0.13

TABLE 3

PIPE LOADS DUE TO RAILROADS

Depth to Top of Pipe, Ft.	Vertical Pressure, ksf		Length of Tunnel Affected, Ft.	
	Single Track (a)	Double Track (b)	Single Track, L_1	Double Track, L_2
1	1.36	(c)	8	(c)
3	.99	(c)	10	(c)
5	.75	(c)	12	(c)
7	.59	(c)	14	(c)
9	.48	(c)	16	(c)
11	.40	.60	18	3
13	.34	.64	20	5
15	.29	.58	22	7
17	.25	.50	24	9
19	.22	.44	26	11

VERTICAL PRESSURE DISTRIBUTION DIAMGRAM



NOTES

(a) Based on one railcar with loaded weight 212 kips
(b) Based on one railcar on each track weighing 212 kips each
(c) At this depth, design to be based on single track pressures

APPENDIX A

BORING LOGS

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PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D **BORING** 5D-1

DATE 8-6-86 **TYPE** 3" Core **LOCATION** Sta.1+70;6' 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 with ferrous oxide		18	106	63	20
5			- tan and gray with calcareous nodules		20	106		
10–15			- brown and gray with sand seams		27	99	72	26
20–25			Gray and tan sandy clay		15	118		

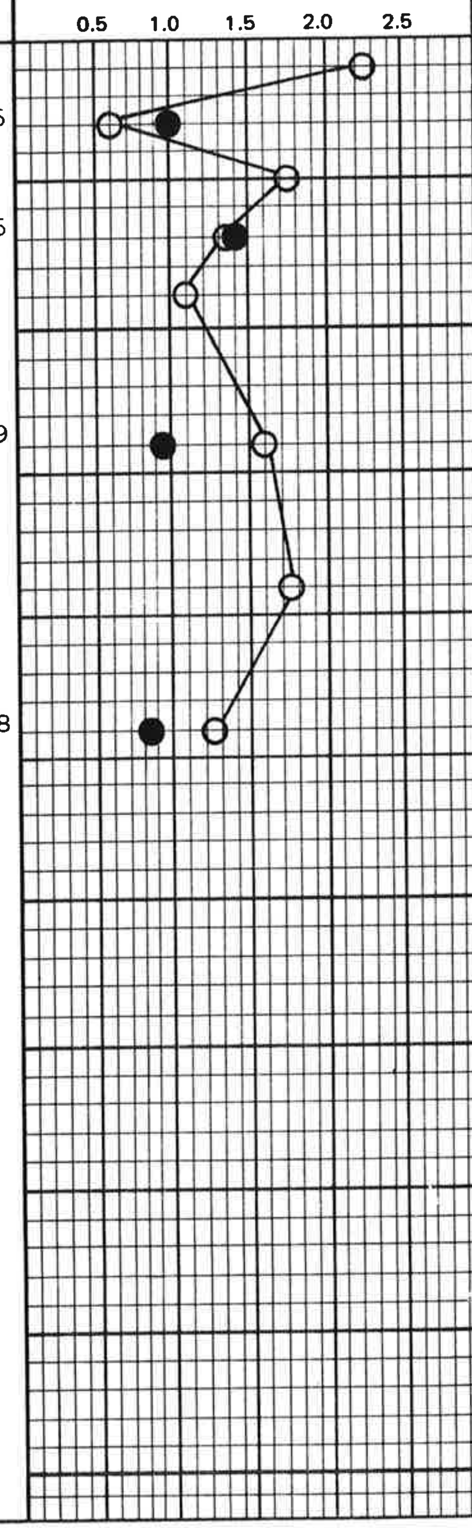
BOTTOM AT 25 FEET

1. Bailed boring to 17'-11" upon completion
2. Water level at 16'-3" after 24 hours

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



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PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D **BORING** 5D-2

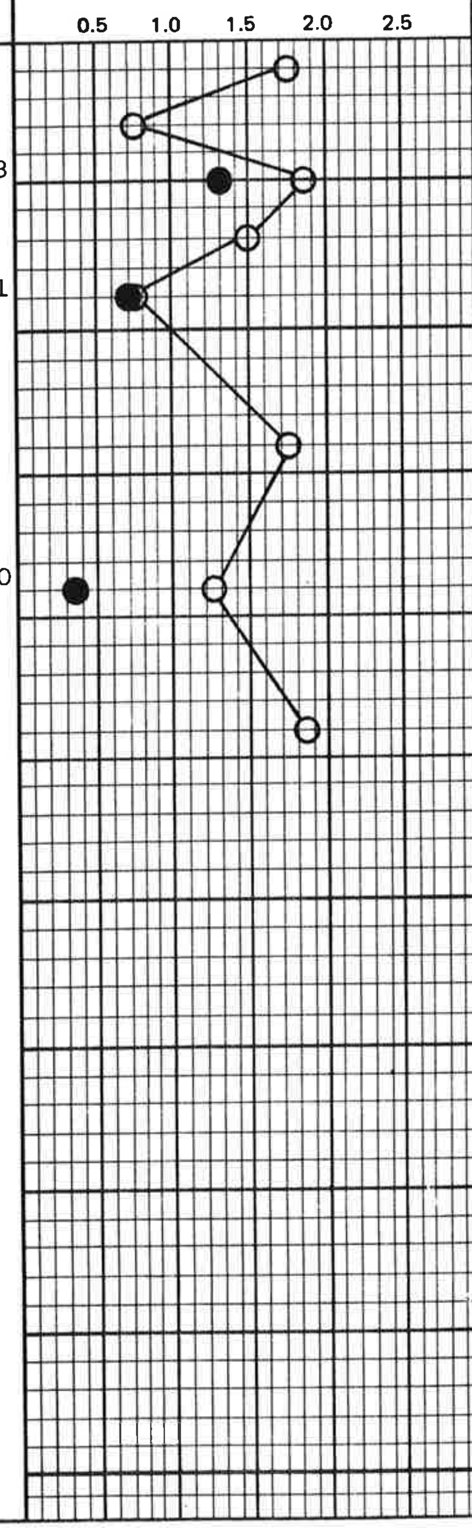
DATE 8-6-86 **TYPE** 3" Core **LOCATION** Sta.11+34;15' R of BL

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS / FOOT	M.C., %	U.D.W., PCF	LIQUID LIMIT	PLASTIC LIMIT
0			Dark gray sandy clay with clay seams					
			Dark gray clay with ferrous oxide		18	108	52	20
5			- tan and gray with calcareous nodules					
					25	101		
			- brown and gray					
10								
15								
			Brown and gray sandy clay		19	110	30	15
20								
25			BOTTOM AT 25 FEET					
			1. Bailed boring to 17'-4" upon completion					
			2. Water level at 14'-7" after 24 hours					
30								
35								
40								
45								
50								

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



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PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D **BORING** 5D-3

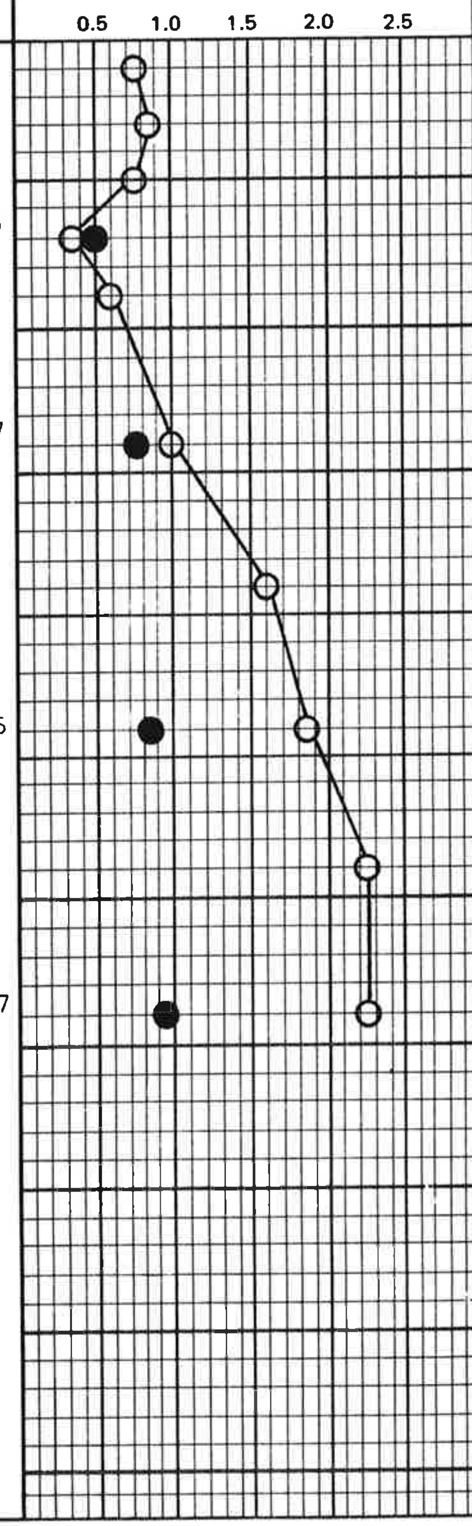
DATE 8-7-86 **TYPE** 3" Core **LOCATION** Sta.20+55;34' R of BL

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS / FOOT	M.C., %	U.D.W., PCF	LIQUID LIMIT	PLASTIC LIMIT
0			Dark gray clay					
5			- gray and tan with calcareous nodules and ferrous oxide		27	96	69	22
10			Tan and gray sandy clay with sand seams		21	107		
15			Red and gray clay with calcareous nodules					
20–25					25	106		
25			Red and gray sandy clay					
30–35			- with clay layers and calcareous nodules		19	107	50	19
35–40				50				

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



BOTTOM AT 40 FEET

1. Bailed boring to 26'3" upon completion
2. Water level at 15'-6" after 24 hours

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50 *murillo engineering, incorporated*

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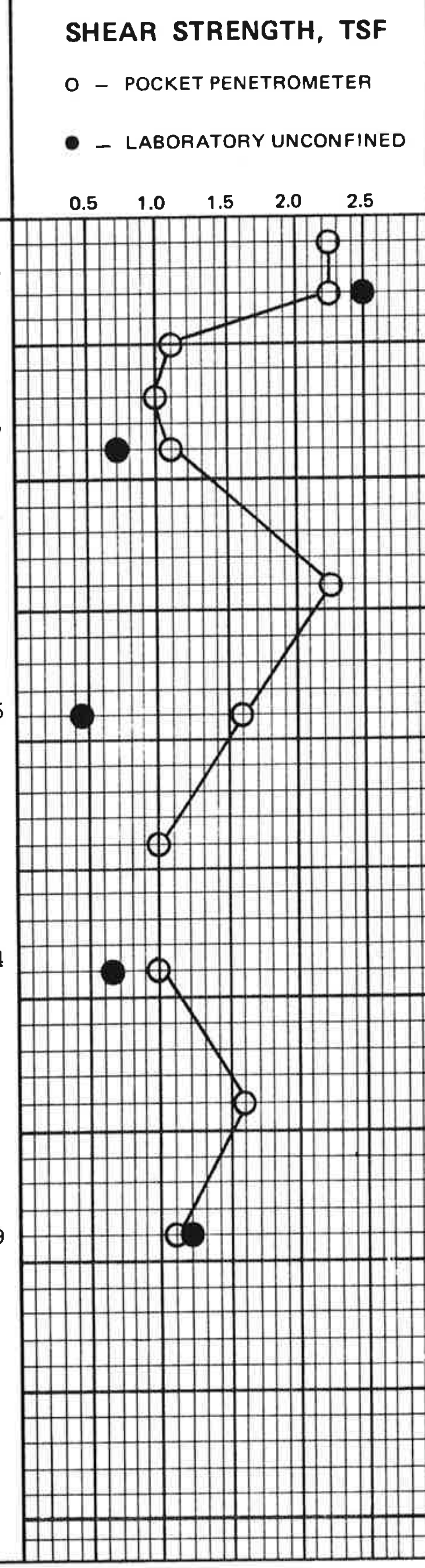
PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D **BORING** 5D-4

DATE 8-7-86 **TYPE** 3" Core **LOCATION** Sta.24+55; 6' 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 sand seams					
			Dark gray clay with ferrous oxide		18	107	56	20
5			- gray and tan with calcareous nodules		24	97		
10								
			Tan and gray sandy clay with sand pockets					
15								
					20	105	24	18
20								
			- with sand layers					
25								
					17	114		
30								
35								
			- with clay layers and calcareous nodules		19	109	49	18
40			BOTTOM AT 40 FEET					
45								
50								

1. Bailed boring to 19'-6" upon completion
2. Water level at 14'-8" after 24 hours



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PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D **BORING** 5D-5

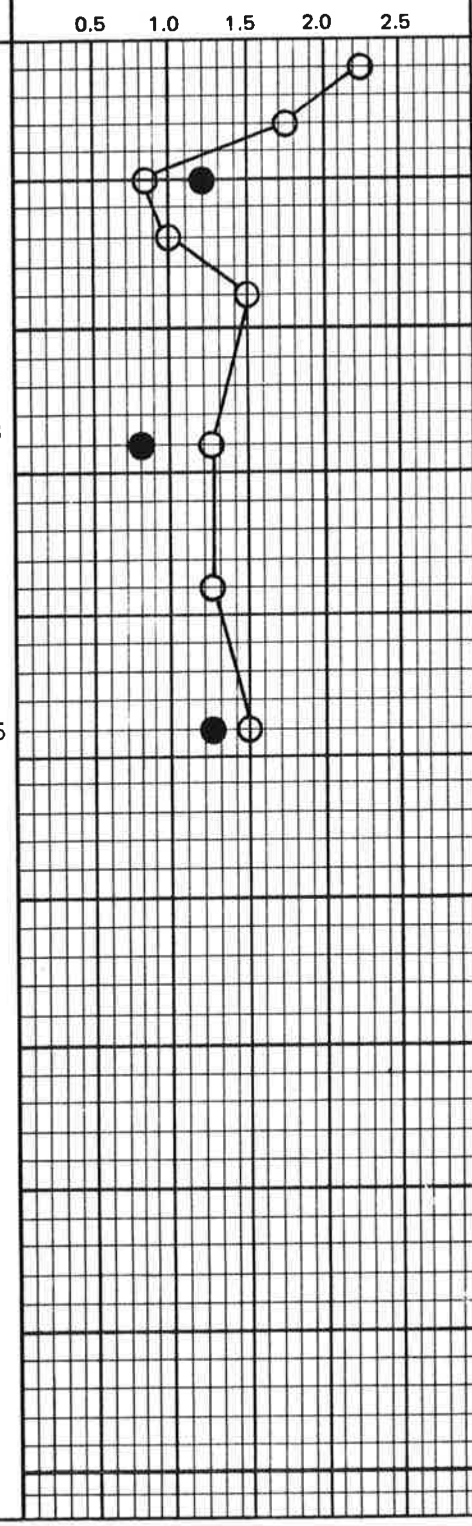
DATE 8-6-86 **TYPE** 3" Core **LOCATION** Sta.31+33; 66' R of BL

DEPTH IN FEET	SYMBOL	CORES	DESCRIPTION	BLOWS / FOOT	M.C., %	U.D.W., PCF	LIQUID LIMIT	PLASTIC LIMIT
0			Dark gray sandy clay					
			Dark gray clay with calcareous nodules and ferrous oxide		22	102	77	22
5								
			- gray and tan					
10								
			Tan and gray sandy clay with calcareous nodules		18	113		
15								
			- with sand seams					
			Brown and gray clay with calcareous nodules					
20					22	105	46	20
25								
30								
35								
40								
45								
50								

SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



BOTTOM AT 25 FEET

1. Bailed boring to 17'-1" upon completion

2. Water level at 14'-2" after 24 hours

PIEZOMETER INSTALLATION REPORT

Project Name: CITY OF HOUSTON-EAST WATER PROGRAM-CONTRACT 5D

Project No.: 275-85E **Piezometer No.** 5D-3

Instrument Location Sta. 20+55; 49' R of BL

Date Installed 8-8-86 **By** MEI

Time Installed - **Inspector** Y. Rahmani

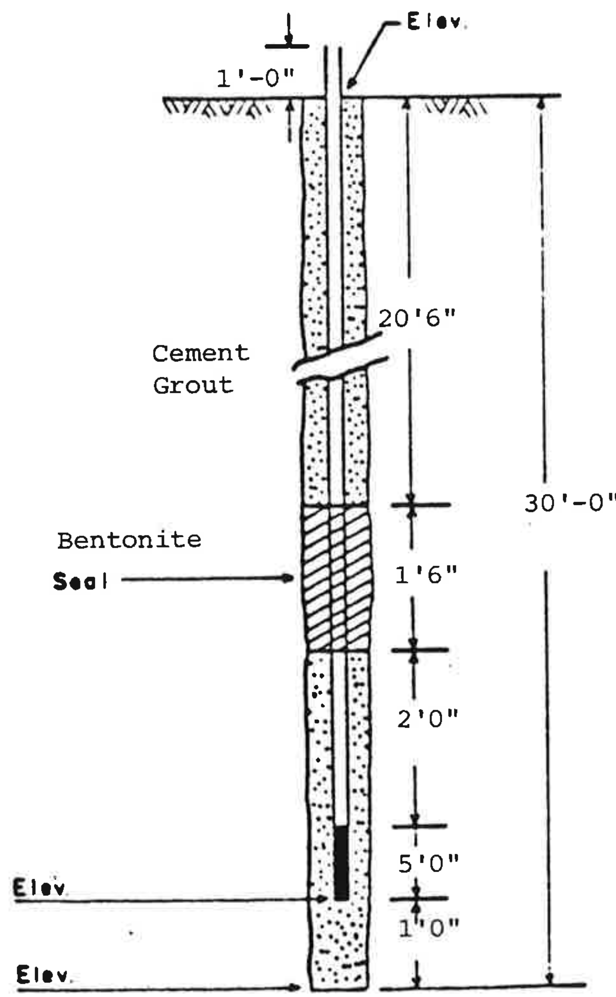
Piezometer Tip 1½" Slotted PVC Pipe

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.
8-11-86	14'-7"
8-14-86	14'-7"
8-18-86	14'-7"
8-22-86	14'-8"
8-25-86	14'-8"

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-CONTRACT 5D

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, %
5D-1	2-4	18	106	2.0				63	20	43			
	6-8	20	106	2.8									
	13-15	27	99	1.9				72	26	46			
	23-25	15	118	1.7									
5D-2	4-6	18	108	2.6				52	20	32			
	8-10	25	101	1.4									
	18-20	19	110	0.7				30	15	15			
5D-3	6-8	27	96	1.0				69	22	47			
	13-15	21	107	1.5									
	23-25	25	106	1.7									
	33-35	19	107	1.9				50	19	31			

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TABLE B-2

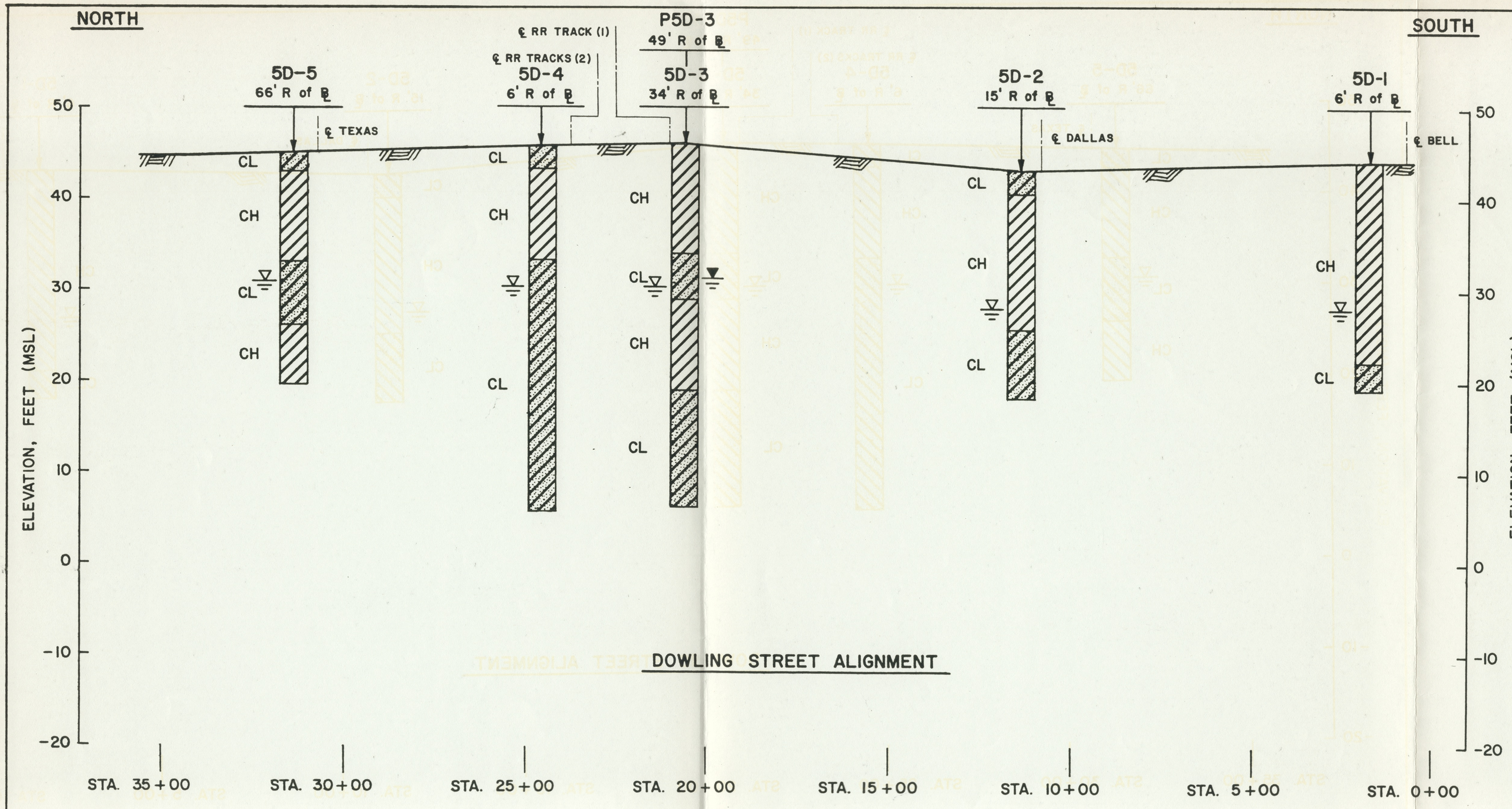
SUMMARY OF LABORATORY TEST DATA

PROJECT CITY OF HOUSTON, EAST WATER PROGRAM-CONTRACT 5D

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, %
5D-4	2-4	18	107	6.0				56	20	36			
	8-10	24	97	1.4									
	18-20	20	105	0.9				24	18	6			
	28-30	17	114	1.3									
	38-40	19	109	2.4				49	18	31			
5D-5	4-6	22	102	2.4				77	22	55			
	13-15	18	113	1.6									
	23-25	22	105	2.5				46	20	26			

APPENDIX C

SUBSURFACE PROFILE



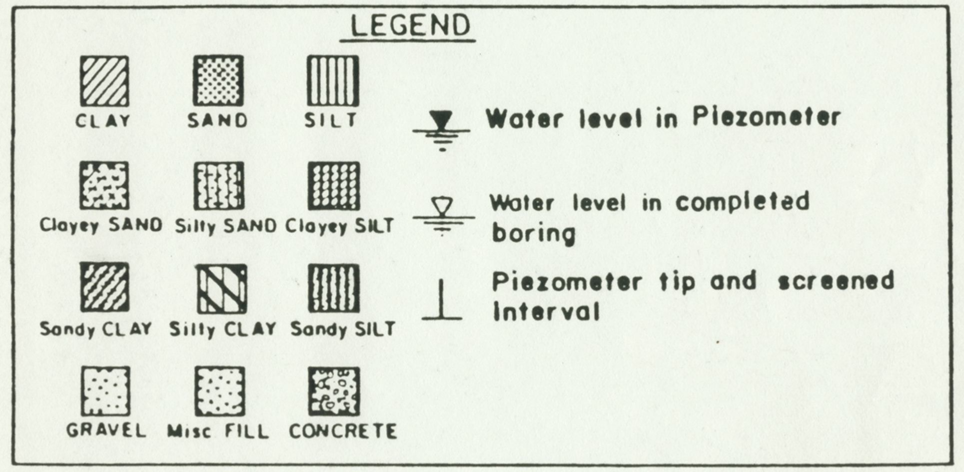
GENERAL NOTES

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.

4. Typical spacing of soil borings for this investigation was 1000 feet.



SCALE: HORIZONTAL 250 0 250; VERTICAL 10 0 10

NAME: City of Houston East Water Program	Murillo Engineering, Inc.	PACKAGE 5D GENERALIZED SOIL PROFILE	FILE Nº 275-85E
FOR: LOCKWOOD, ANDREWS & NEWNAM	SCALE: NOTED; MADE BY: J.H. DATE: 8-28-86; CHECKED BY: V.H.V. DATE: 9-2-86		FIGURE C-1

APPENDIX D

J.R.Carlsen's Paper on Thrust Restraint

THRUST RESTRAINT for Underground Piping Systems

by:
Roger J. Carlsen, P.E.
Senior Engineer

Introduction

Fundamental design principles of fluid mechanics recognize the presence of unbalanced thrust forces in pressure piping systems. These forces, resulting from static and dynamic fluid action on the pipe, require physical restraint for system stabilization.

Locations where unbalanced thrust forces commonly occur are:

Bends	Wyes
Reducers	Offsets
Tees	Dead-ends
Valves	Hydrants

In addition, installations on steep slopes, in swamps, marshes, muck or peat bogs frequently require special restraining techniques for efficient anchorage.

Adequate restraint is generally achieved for ductile or cast iron piping systems by employing one or more of the following methods:

- Restrained Joints
- Thrust Blocks
- Tie Rods
- Combined Systems and Structural Connections

A discussion of these restraining methods, including a new design approach for restrained joint systems, is presented in this paper.

Soil characteristics are of prime importance in the design of thrust restraining systems. Accepted principles of soil mechanics have been applied in the derivation and formulation of the design procedures discussed herein.

DEFINITION OF TERMS

A = Pipe cross-sectional area (in²) ($36 \pi D^2$) with "D" in (ft)
A_b = Minimum bearing area of block base (ft²)
A_p = Conduit surface area (ft²/lf)
A_r = Cross-sectional area of rod (in²)
b = Width of thrust block (ft)
B = Gravity block base dimension (ft)
C = Pipe cohesion (psf)
C_s = Soil cohesion (psf)
D = Conduit outside diameter (ft)
D_b = Outside diameter of bell (ft)
F = Force developed per rod (lbs)
F_b = Bell resistance (lbs)
f_c = Ratio of pipe cohesion/soil cohesion
F_h = Acting horizontal force (plf)
F_H = Horizontal force acting on conduit at maximum passive pressure (plf)
F_N = Lateral soil resistance (lbs)
F_p = Resisting force developed by P_p (lbs)
F_s = Conduit frictional resistance neglecting bell resistance (plf)
F_s' = Conduit frictional resistance including bell resistance (plf)
F_z = Residual thrust force at partial restraint (lbs)

f_ϕ = Ratio of pipe friction angle/soil friction angle
h = Height of thrust block (ft)
H = Cover above conduit (ft)
H_c = Depth of cover to conduit centerline (ft)
H_T = Depth to bottom of block (ft)
K = Bend coefficient
L = Restrained pipe length each side (ft)
L_p = Nominal pipe length adj. to fitting (ft)
L_T = Length of tee (ft)
L_j = Distance from bend to joint (ft)
L_x = $L_T + 2L_p$ (ft)
N = Number of rods
N_ϕ = $\tan^2 (45° + \phi/2)$
P = Max. sustained pressure (psi) (test pressure or sustained surge pressure)
P_p = Passive soil pressure (psf)
P_v = Vertical soil pressure (psf)
R = Reduction factor
S = Tensile stress of rod material (psi)
S_f = Safety factor (usually 1.25)
T = Resultant thrust force (lbs)
T_j = Thrust force at joint (lbs)
T_x = x thrust force component (lbs)
T_y = y thrust force component (lbs)
V = Fluid velocity (fps)
V_G = Volume of gravity block (ft³)
w = Soil unit weight (pcf)
W = Normal force on pipe (plf)
W_e = Prism earthload (plf)
W_c = $W_e + W_p + W_w$ (plf)
W_g = Weight of gravity block (lbs)
W_m = Density of block material (pcf)
W_p = Weight of pipe (plf)
W_w = Contained liquid weight (plf)
W_y = Effective weight of soil, pipe and water (lbs)
Θ = Bend deflection angle (degrees)
δ = Pipe friction angle (degrees)
ϕ = Soil internal friction angle (degrees)
$\triangle$ = Angle between T and x-axis (degrees)

Basic Design Criteria

Buried thrust systems rely almost entirely upon supporting soil strength. As backfilling commences, intermediate soil pressure within the active to passive range, depending upon compaction, is exerted upon the conduit. As pressurization occurs, unbalanced thrust forces cause slight movement of the system, producing sufficient soil deformation to develop passive soil resistance. Since movement must be limited for system integrity, compaction of the backfill soils is necessary to reduce excessive soil deformation. Generally, sands and silts compacted to critical void ratio will satisfy this criteria (about 80% Standard AASHO).

Important soil properties influencing design are passive soil pressure (P_p), cohesion (C_s), friction (ϕ) and density (w).

Design criteria for restrained joint systems consider both lateral soil resistance (F_N) and sliding frictional resistance (F_s) acting simultaneously on the conduit. Approximately one half of the passive soil pressure was used in the determination of lateral soil resistance represented by the triangular force diagram in Figure 3 on page 11. The reduction of passive soil pressure was considered practical since compacted backfill will encompass the pipe rather than undisturbed soil.

Sliding frictional resistance is a function of conduit cohesion (C), friction angle (δ), surface area (A_p) and normal (radial) force (W) exerted upon the conduit by the surrounding soil. Ratios, f_c and f_ϕ respectively, relating pipe cohesion and friction to soil cohesion and friction have been developed for various soil types (see Table 1). Laboratory evaluation of soil shearing resistance thus enables determination of pipe friction and cohesion through application of these factors.

Normal force is a function of vertical soil pressure (P_v) and conduit surface area.

Dynamic fluid action has been neglected since the effect is insignificant for lower fluid velocities prevailing in the water works industry.

Gasket friction, lateral sliding friction, active soil pressure and resistance from connecting service laterals represent other factors neglected in design.

Pipe bells and external retainer rings provide considerable resistance to sliding. Passive soil shear strength, developed on the bell or ring as slight movement occurs, may be considered to achieve design efficiency for conditions such as shallow cover, buoyancy, dead-ends and tees. For thrust systems inundated by water, buoyant soil properties are used for design.

Restrained Joint Thrust Systems

Horizontal Bends: A restrained joint must be capable of transmitting thrust and shear forces through the joint to the pipe wall, whereby these forces are counteracted by combined frictional resistance and lateral soil pressure.

Manufacturers of cast and ductile iron pressure pipe have developed and marketed restrained joints (see Figure 1) satisfying these criteria. Most of these joints demonstrate flexibility, which is advantageous for underground service.

Restrained joints are predominantly used where thrust blocks are not economical or practical due to limited space, access, unstable soils, or possible disturbance by future excavation.

The thrust force acting on a bend is

(1) $T = 2\ PA\ \text{Sin}\ \dfrac{\Theta}{2}$ (see Figure 2)

Introducing the stabilizing forces of lateral soil resistance and frictional resistance into the free-body diagram of Figure 2 yields the force system shown in Figure 3. An analytical solution of this force system for restrained length (L) gives,

(2) $L = \dfrac{S_f KPA}{KF_s + DP_p}$ where $K = 4\ \text{Tan}\ \dfrac{\Theta}{2}$

Frictional resistance is expressed by a general form of the Coulomb equation.

(3) $F_s = A_p C + W \tan \delta$ where $C = f_c C_s$ and $\delta = f_\phi \phi$

(see Table 1)

Table 1

Soil Friction and Cohesion Factors

Soil Description	Friction Angle ϕ (Degrees)	Cohesion C_s (psf)	f_ϕ	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

A comparative study of three separate loading conditions, shown in Figures 4 and 5, revealed that (W) can be determined by applying a reduction factor (R) (see Table 2) to the product of ($P_v A_p$).

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

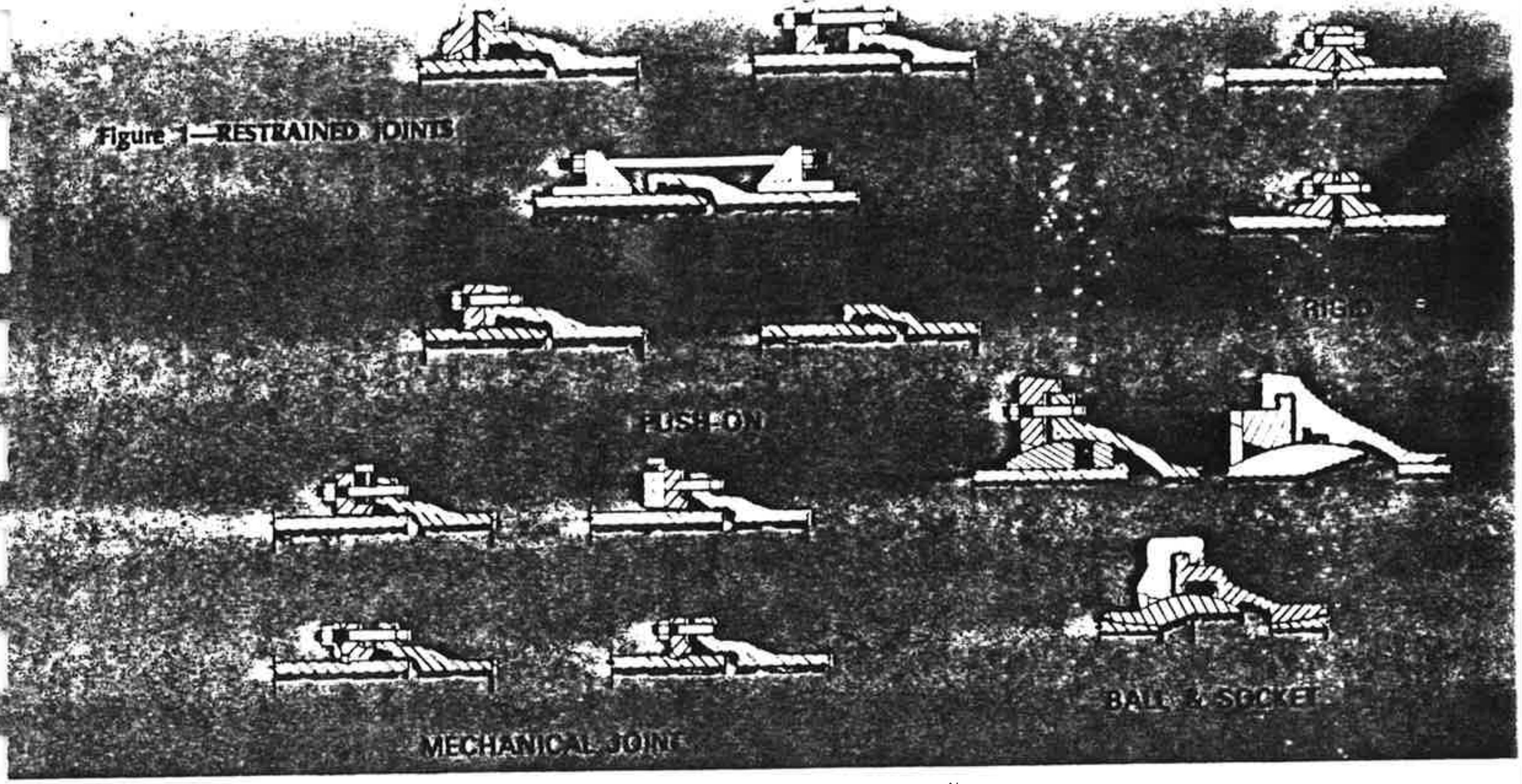


Figure 1—RESTRAINED JOINTS

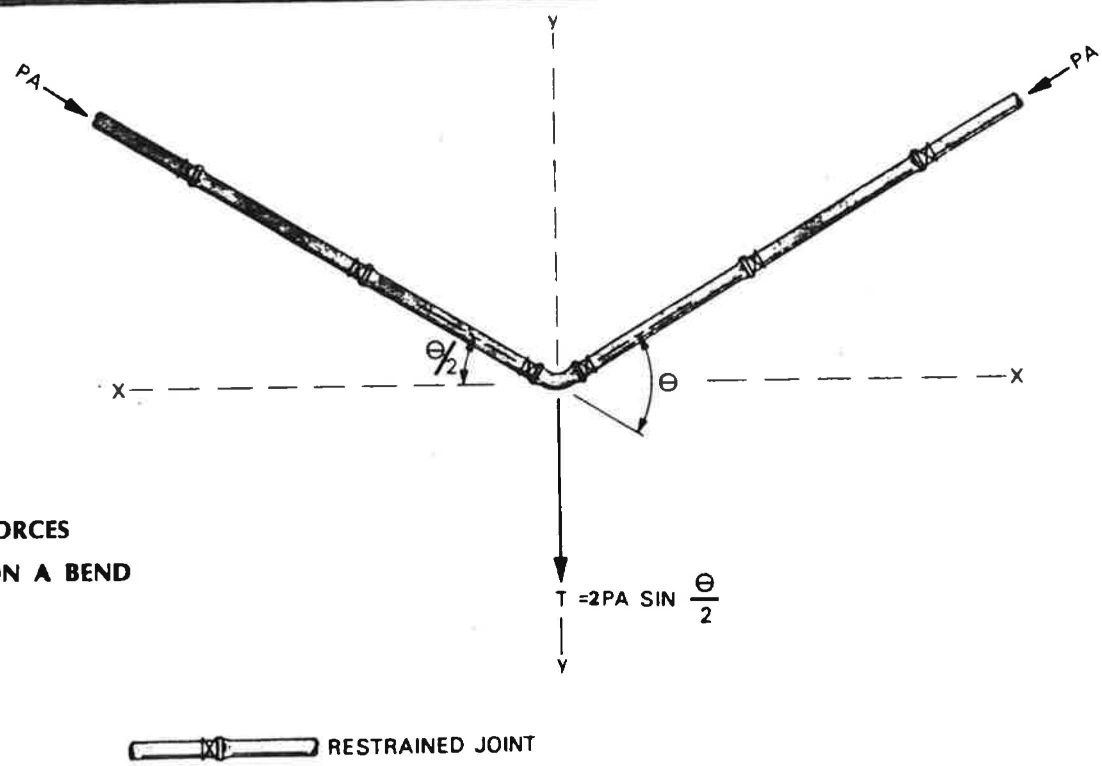


Figure 2
THRUST FORCES
ACTING ON A BEND

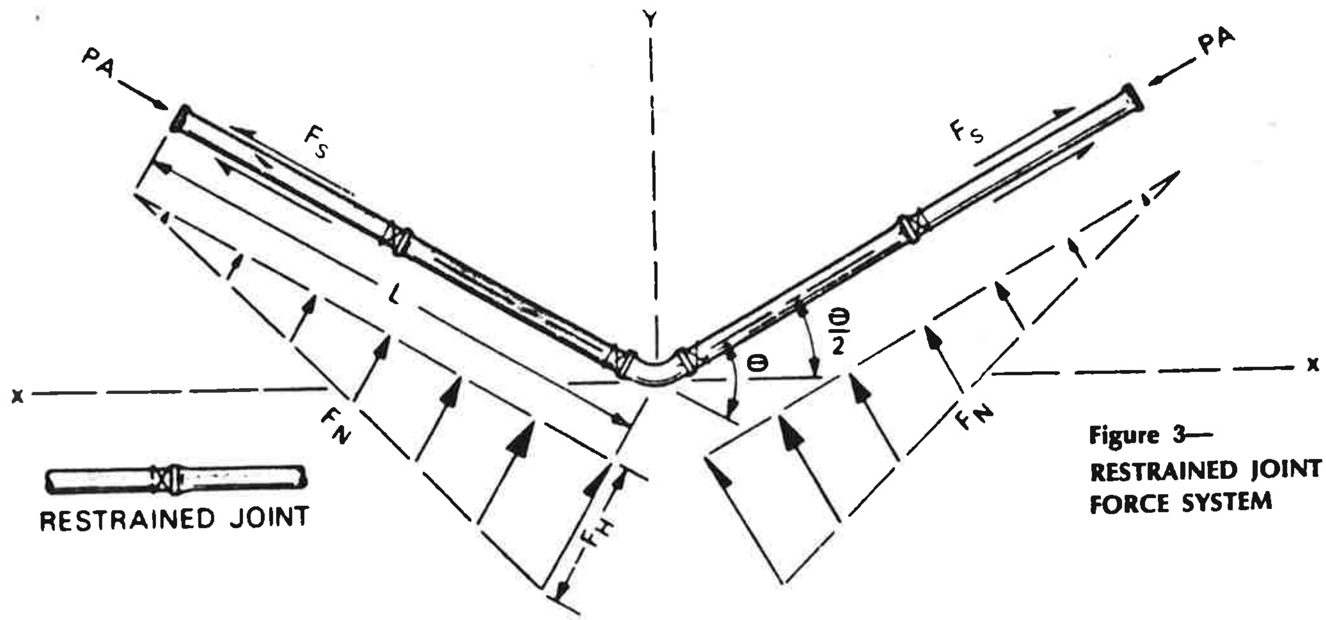
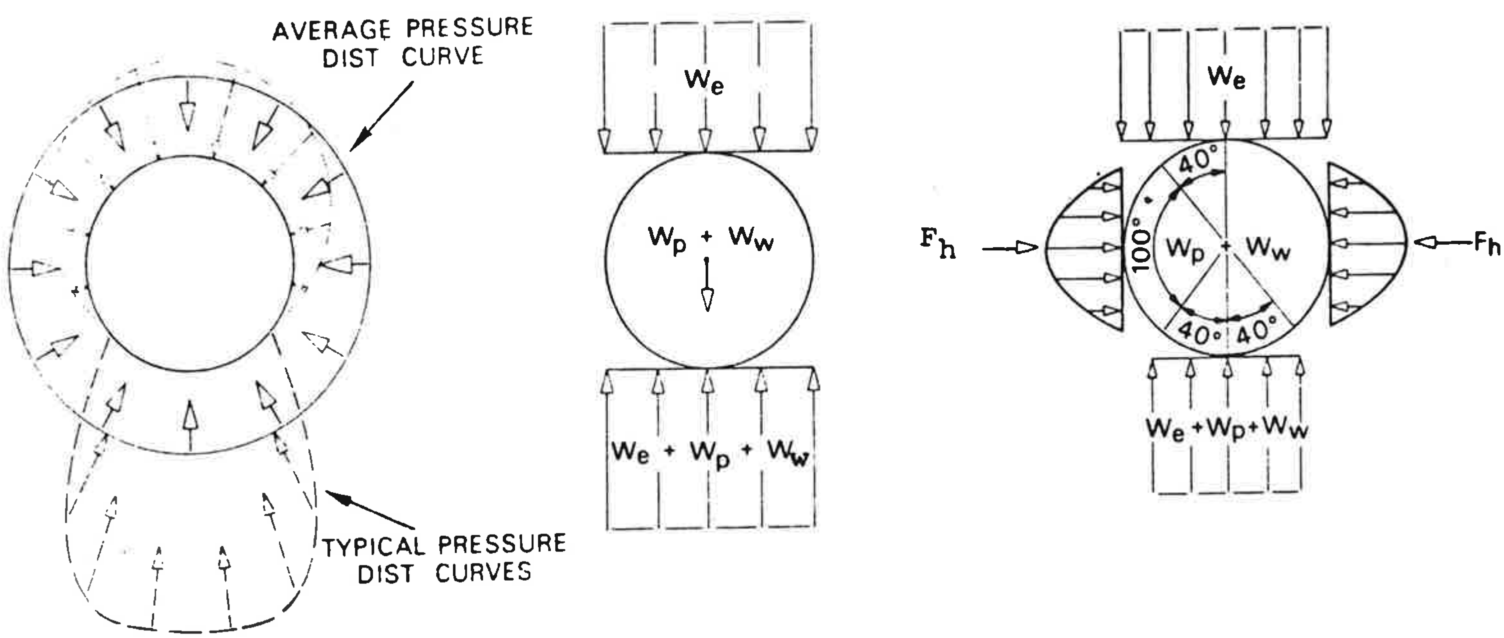


Figure 3—
RESTRAINED JOINT
FORCE SYSTEM



CONDITION 1

$W_1 = \pi wHD$

CONDITION 2

$W_2 = 2W_e + W_p + W_w$

CONDITION 3

$W_3 = 2W_e + W_p + W_w + 2F_h$

Figure 4—SOIL PRESSURE DIAGRAMS

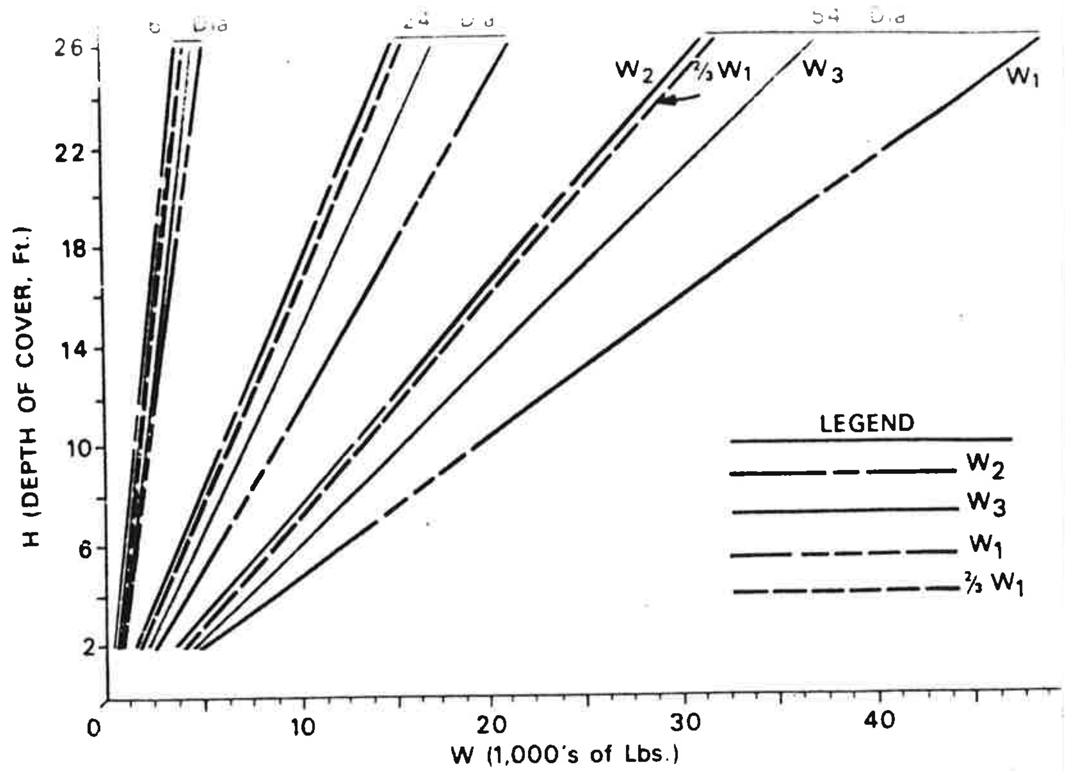


Figure 5—COMPARISON OF THEORETICAL NORMAL FORCES

(4) $W = \pi wRHD$

Passive soil pressure according to the Rankine theory is,

(5) $P_p = wH_cN_\phi + 2C_s\sqrt{N_\phi}$
where $N_\phi = \text{Tan}^2\left(45^\circ + \frac{\phi}{2}\right)$

In Figure 3, (F_H) was determined by assuming a parabolic soil pressure distribution, similar to that shown in Condition 3 of Figure 4, acting horizontally on the conduit, in which (P_p) represents the maximum soil pressure.

Bell resistance (F_b) is determined as follows:

(6) $F_b = \pi \dfrac{P_p(D_b^2 - D^2)}{4}$

For design, (F_b) is expressed as an equivalent force per linear foot (F_b/L_p) where (L_p) is the nominal pipe length. Therefore,

(7) $F_s' = A_pC + W \text{ Tan } \delta + F_b/L_p$

CIPRA field tests have indicated that (F_s) and (F_s') should be reduced from 25% to 30% when polyethylene encasement is provided on the conduit.

Vertical Bends: Upward thrusts may be stabilized with restrained joints by considering frictional resistance and weight acting at the bend. An analysis similar to horizontal bends results in the following equation:

(8) $L = \dfrac{S_fKPA}{KF_s + 2W_c}$

The force resisting vertical thrust (W_c) consists of the weight of the soil prism above the pipe ($W_s = wHD$), weight of pipe (W_p) and contained liquid (W_w).

(9) $W_c = W_s + W_p + W_w$

W_c was assumed to vary from the maximum value at the bend to zero at restrained length (L).

The design depth of cover (H) is selected as the minimum cover acting at any location throughout length (L). It is important to note that Equation (8) is dependent upon the soil prism above the pipe remaining in place and relatively free from saturation.

Dead-ends: The required length (L) for restraint of dead-ends is expressed as

$$(10)\quad L = \frac{S_f PA}{F_s'}$$

Passive soil resistance developed at the dead-end may be included in the design, providing the supporting soil remains undisturbed.

Tees: Significant restraint is provided by passive soil pressure supporting main line tees. Since iron pipe joints are capable of transmitting shear force, lateral soil resistance acting on the tee and adjoining lengths of pipe resists thrust forces from the outlet lateral.

Restraint is required when lateral soil resistance, supporting the tee and adjacent piping, is exceeded. Restrained length (L) along the outlet lateral becomes

$$(11)\quad L = \frac{S_f\ (4PA - DP_pL_x)}{4F_s'}$$

where PA is the outlet thrust and $L_x = (L_T + 2L_p)$

A negative value in the numerator indicates that lateral soil resistance alone provides adequate restraint.

Thrust Blocking

Concrete thrust blocks are the most common method of restraint now in use, providing stable soil conditions prevail and space requirements permit placement. Successful blocking is dependent upon factors such as location, availability and placement of concrete, and possible disturbance through future excavation. Concrete blocks are readily utilized in combination with tie rods, structural anchoring, thrust collars and restrained joints.

Thrust blocks are generally categorized into two groups: gravity and bearing blocks.

Gravity Blocks (Figure 6): Important factors considered in design are:

—Horizontal and vertical thrust components
—Allowable bearing value of soil
—Combined weight of pipe, water and soil prism
—Density of block material
—Block dimensions and volume
—A thrust force analysis is conducted similar to Figure 7.

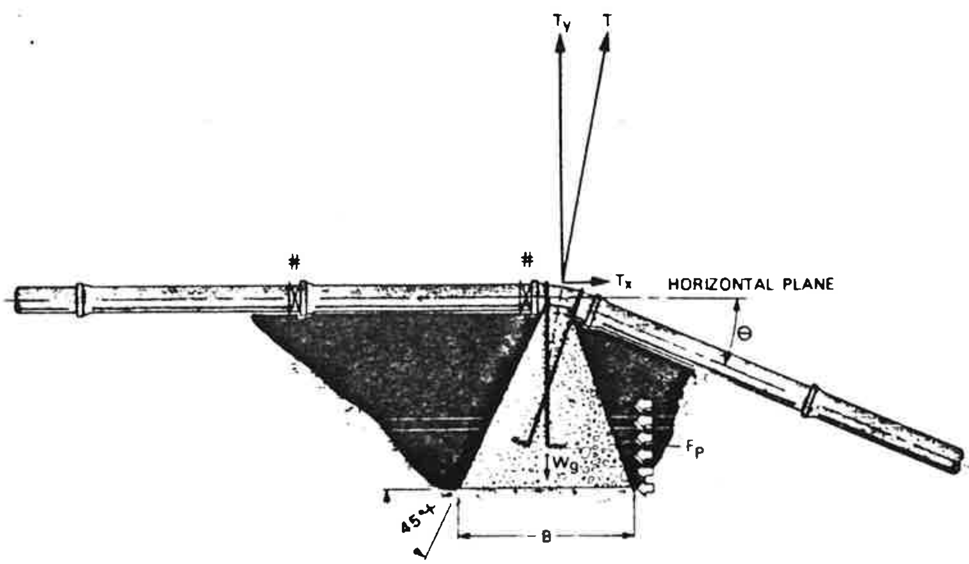
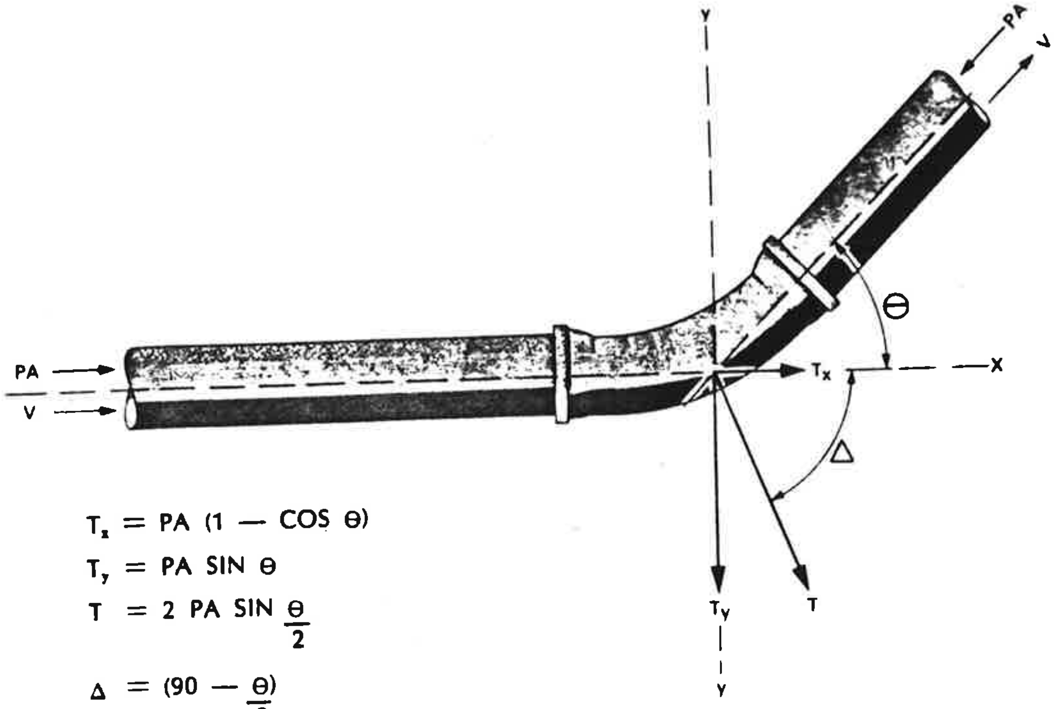


Figure 6—GRAVITY THRUST BLOCK

Restrained Joints may be used when $T_x > F_p$



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

$T_y = PA\ SIN\ \Theta$

$T = 2\ PA\ SIN\ \frac{\Theta}{2}$

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

Figure 7—THRUST FORCES ACTING ON A BEND

Physical characteristics of the block are determined from the following formulas:

(12) $V_G = \frac{PA\ Sin\ \Theta}{W_m}$ (neglecting W_y)

$V_G = \frac{T_y - W_y}{W_m}$ (including W_y)

where $W_y = 1/2\ W_e L_x$

Earth cover (W_e) is neglected, when determining (W_c), if unstable conditions are anticipated. The horizontal thrust component (T_x) is counteracted by soil pressure on the vertical face of the block (F_p) or by joint restraint.

Allowable soil bearing pressure determines the minimum size of the block base.

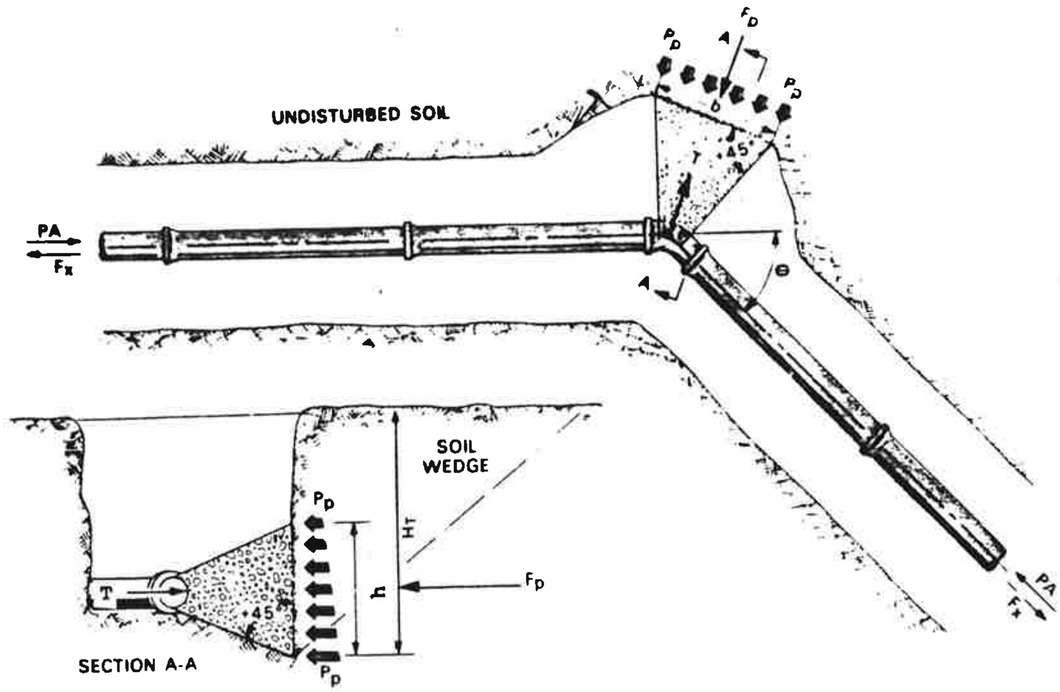


Figure 8—BEARING THRUST BLOCK

Bearing Blocks (Figure 8):

Significant design criteria for bearing blocks include the following factors:

- —Passive soil pressure
- —Placement of bearing surface against undisturbed soil
- —Block height (h) should be equal to or less than one half the total depth to the block base (H_T) except (h) should not be less than (D). Thus $h \leqq 1/2\ H_T$ or $h \geqq D$, whichever is greater.
- —Block width (b) usually varies from one to two times the height (h).
- —Concrete should not be poured on joints, limiting flexibility.

The required block bearing area, based on passive soil pressure, is expressed as follows:

$$(14)\quad A_b = hb = \frac{T}{P_p}$$

For the case where $h = 1/2\ H_T$,

$$(15)\quad b = \frac{2PA \sin\frac{\Theta}{2}}{3/8\ w\ H_T^2\ N_\phi + C_s\ H_T \sqrt{N_\phi}}$$

(H_T) is estimated, permitting calculation of (b). Dimensions are selected by trial and error.

Pipelines under shallow cover are frequently deepened at the bends, increasing the depth of cover, to achieve more efficient block design. Colinear positioning of (T) and (F_p) is required to eliminate overturning moment on the block.

When partial restraint is provided through blocking, the residual thrust force (F_x) may be stabilized with restrained joints.

$$(16)\quad F_x = PA\left(1-\frac{F_p}{T}\right)$$

The restrained length (L) is calculated from Equation 2 by substituting (F_x) for (PA).

Tie Rods

Restraint with tie rods is versatile and relatively easy. Locations where tie rods are readily used include:

- —Anchorage to structure, thrust collars, "deadman" anchors, and superstructures

—Joint restraint by utilizing clamps, pipe flange
—holes, or lugs cast on fittings.
—Restraint for field-cut, make-up sections

Tie rods on exposed piping systems must counteract total resultant thrust forces. However, on buried systems employing soil friction and lateral soil resistance, the effective thrust force at a joint (T_j) is proportional to its distance from the bend (L_j) and the restrained length (L).

(17) $T_j = F_s(L-L_j)$

The required number of rods (N) is

(18) $N = \frac{S_f T_j}{F}$ where $F = SA_r$ and $S_f = 1.5$

Coating or wrapping is recommended for buried tie rods to prevent corrosion attack from corrosive soils.

Combined Systems and Structural Connections

Several restraining techniques are frequently required for thrust stabilization. Typical combinations include concrete blocks and tie rods, restrained joints and tie rods, or restrained joints, tie rods and thrust anchors.

Low head, in-plant piping is conveniently restrained and supported by attachments to nearby structures. Typical anchoring devices include the use of wall brackets, U-bolts and clamps, base elbows and tees, wall sleeves, structural steel frames, concrete supports and anchor bolts or straps.

Selection becomes a matter of preference depending upon convenience.

Summary

The proper restraint of unbalanced thrust forces is an important consideration in pressure piping design. Functional methods employed for cast iron piping systems are restrained joints, thrust blocks, tie rods or any combination thereof.

Due to the increased use of restrained joints, a new design procedure, which considers lateral soil resistance in combination with frictional resistance, is presented. The ability of gray and ductile iron pipe restrained joints to transmit lateral shear and axial thrust through the joint, enables full consideration of frictional and lateral resistance in design.

The restrained length (L) is a function of static thrust (PA), frictional resistance (F_s) and lateral soil resistance (F_N) which is based on the passive soil pressure (P_p).

Frictional resistance is dependent upon surface area of the conduit, normal force, bell resistance and soil friction/cohesion. Studies indicate that the normal force can be determined from the vertical soil pressure and conduit surface area. Formulas for restraint of horizontal and vertical bends, dead-ends, and tees have been developed for system design.

Design procedure for bearing thrust blocks is based on passive soil pressure which is theoretically more accurate than assuming soil bearing strength.

For buried systems, tie rod design is based on axial thrust acting at the joint rather than the total static pressure (PA).

Design tables for restrained joints in various soil types are included to expedite design.

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