APPENDIX J

HORIZONTAL DIRECTIONAL DRILLING

The Horizontal Directional Drilling Process

The tools and techniques used in the horizontal directional drilling (HDD) process are an outgrowth of the oil well drilling industry. The components of a horizontal drilling rig used for pipeline construction are similar to those of an oil well drilling rig with the major exception being that a horizontal drilling rig is equipped with an inclined ramp as opposed to a vertical mast. HDD pilot hole operations are not unlike those involved in drilling a directional oil well. Drill pipe and downhole tools are generally interchangeable and drilling fluid is used throughout the operation to transport drilled spoil, reduce friction, stabilize the hole, etc. Because of these similarities, the process is generally referred to as drilling as opposed to boring.

Installation of a pipeline by HDD is generally accomplished in three stages as illustrated in Figure 1. The first stage consists of directionally drilling a small diameter pilot hole along a designed directional path. The second stage involves enlarging this pilot hole to a diameter suitable for installation of the pipeline. The third stage consists of pulling the pipeline back into the enlarged hole.

Pilot Hole Directional Drilling

Pilot hole directional control is achieved by using a non-rotating drill string with an asymmetrical leading edge. The asymmetry of the leading edge creates a steering bias while the non-rotating aspect of the drill string allows the steering bias to be held in a specific position while drilling. If a change in direction is required, the drill string is rolled so that the direction of bias is the same as the desired change in direction. The direction of bias is referred to as the tool face. Straight progress may be achieved by drilling with a series of offsetting tool face positions. The drill string may also be continually rotated where directional control is not required. Leading edge asymmetry can be accomplished by several methods. Typically, the leading edge will have an angular offset created by a bent sub or bent motor housing. This is illustrated schematically in Figure 2.

It is common in soft soils to achieve drilling progress by hydraulic cutting with a jet nozzle. In this case, the direction of flow from the nozzle can be offset from the central axis of the drill string thereby creating a steering bias. This may be accomplished by blocking selected nozzles on a standard roller cone bit or by custom fabricating a jet deflection bit. If hard spots are encountered, the drill string may be rotated to drill without directional control until the hard spot has been penetrated.

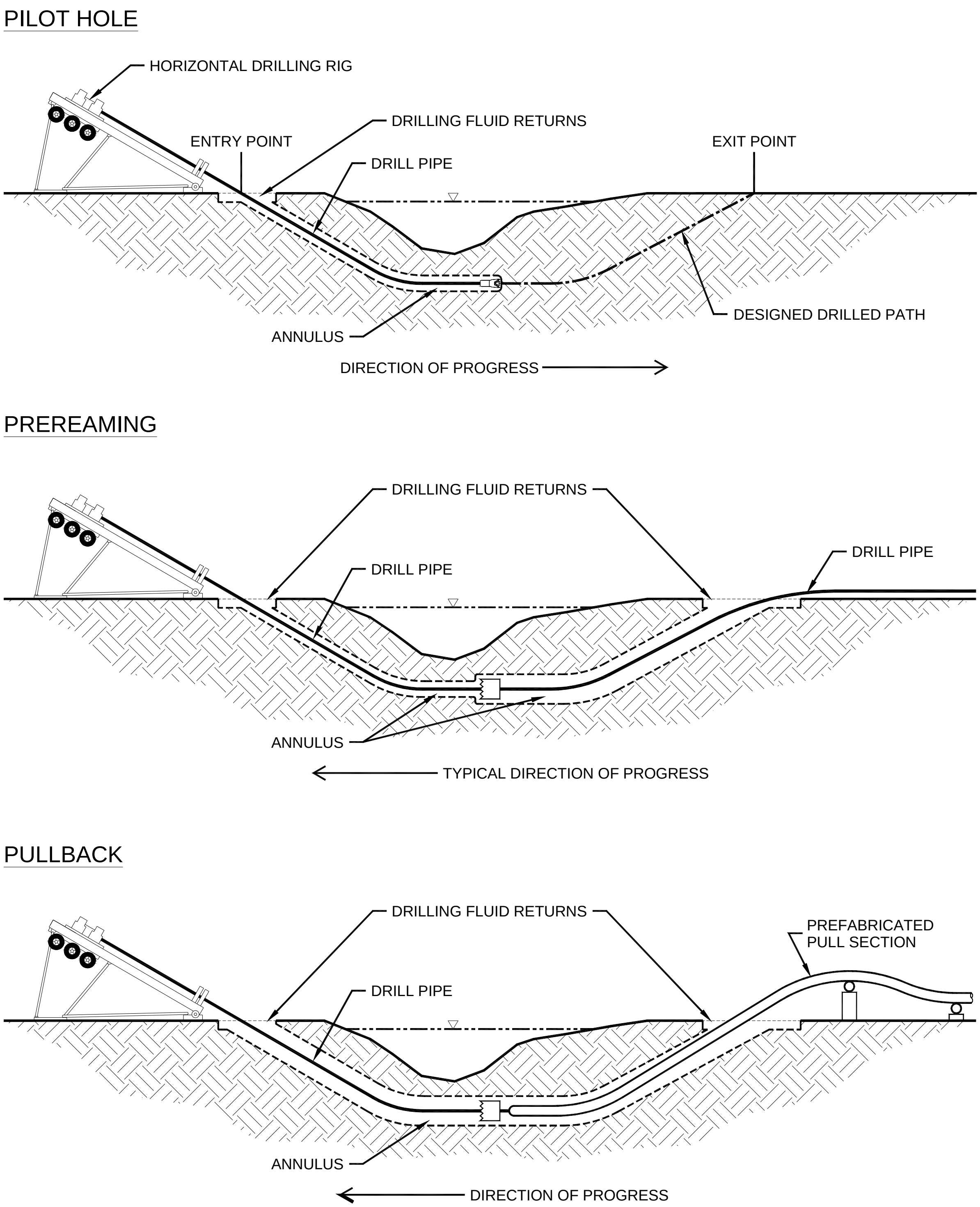

Figure 1
The HDD Process

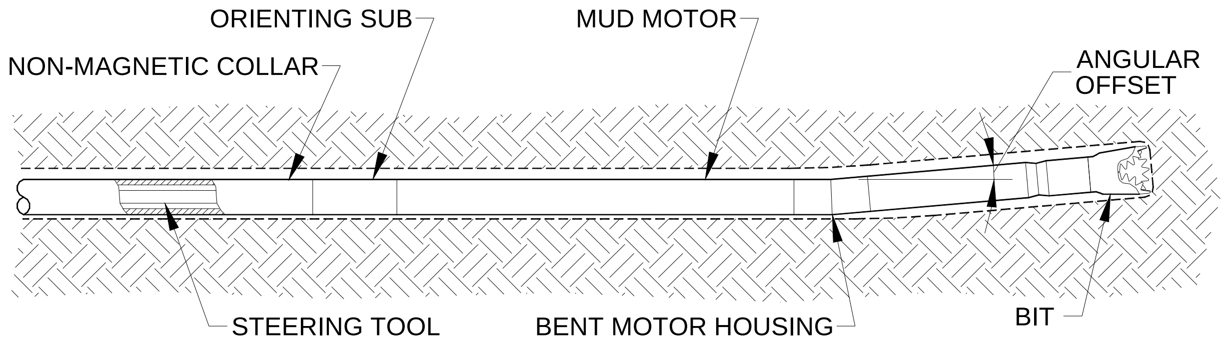

Figure 2
Bottom Hole Assembly

Downhole Motors

Downhole mechanical cutting action required for harder soils is provided by downhole hydraulic motors. Downhole hydraulic motors, commonly referred to as mud motors, convert hydraulic energy from drilling mud pumped from the surface to mechanical energy at the bit. This allows for bit rotation without drill string rotation. There are two basic types of mud motors; positive displacement and turbine. Positive displacement motors are typically used in HDD applications. Basically, a positive displacement mud motor consists of a spiral-shaped stator containing a sinusoidal shaped rotor. Mud flow through the stator imparts rotation to the rotor which is in turn connected through a linkage to the bit.

In some cases, a larger diameter wash pipe may be rotated concentrically over the non-rotating steerable drill string. This serves to prevent sticking of the steerable string and allows its tool face to be freely oriented. It also maintains the pilot hole if it becomes necessary to withdraw the steerable string.

Downhole Surveying

The actual path of the pilot hole is monitored during drilling by taking periodic readings of the inclination and azimuth of the leading edge. Readings are taken with an instrument, commonly referred to as a probe, inserted in a drill collar as close as possible to the drill bit. Transmission of downhole probe survey readings to the surface is generally accomplished through a wire running inside the drill string. These readings, in conjunction with measurements of the distance drilled since the last survey, are used to calculate the horizontal and vertical coordinates along the pilot hole relative to the initial entry point on the surface.

Azimuth readings are taken from the earth's magnetic field and are subject to interference from downhole tools, drill pipe, and magnetic fields created by adjacent structures. Therefore, the probe must be inserted in a non magnetic collar and positioned in the string so that it is adequately isolated from downhole tools and drill pipe. The combination of bit, mud motor (if used), subs, survey probe, and non magnetic collars is referred to as the Bottom Hole Assembly or BHA. A typical bottom hole assembly is shown as Figure 2.

Surface Monitoring

The pilot hole path may also be tracked using a surface monitoring system. Surface monitoring systems determine the location of the probe downhole by taking measurements from a grid or point on the surface. An example of this is the TruTracker System. This system uses a surface coil of known location to induce a magnetic field. The probe senses its location relative to this induced magnetic field and communicates this information to the surface. This is shown schematically in Figure 3.

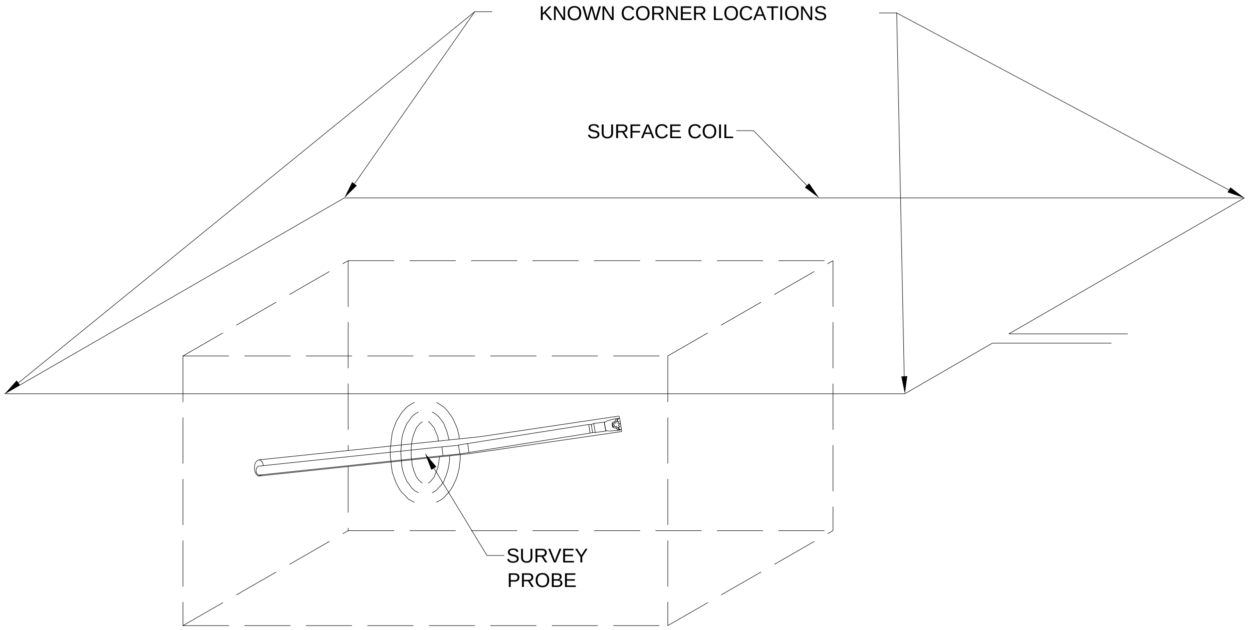

Figure 3
TruTracker Surface Monitoring System

Reaming & Pullback

Enlarging the pilot hole is accomplished using either prereaming passes prior to pipe installation or simultaneously during pipe installation. Reaming tools typically consist of a circular array of cutters and drilling fluid jets and are often custom made by contractors for a particular hole size or type of soil.

Prereaming

Most contractors will opt to preream a pilot hole before attempting to install pipe. For a prereaming pass, reamers attached to the drill string at the exit point are rotated and drawn to the drilling rig thus enlarging the pilot hole. Drill pipe is added behind the reamers as they progress toward the drill rig. This insures that a string of pipe is always maintained in the drilled hole. It is also possible to ream away from the drill rig. In this case, reamers fitted into the drill string at the rig are rotated and thrust away from it.

Pullback

Pipe installation is accomplished by attaching the prefabricated pipeline pull section behind a reaming assembly at the exit point and pulling the reaming assembly and pull section back to the drilling rig. This is undertaken after completion of prereaming or, for smaller diameter lines in soft soils, directly after completion of the pilot hole. A swivel is utilized to connect the pull section to the leading reaming assembly to minimize torsion transmitted to the pipe. The pull section is supported using some combination of roller stands, pipe handling equipment, or a flotation ditch to minimize tension and prevent damage to the pipe.

Buoyancy Control

Uplift forces resulting from the buoyancy of larger diameter lines can be very substantial. High pulling forces may be required to overcome drag resulting from buoyancy uplift. Therefore, contractors will often implement measures to control the buoyancy of pipe 30 inches or over in diameter. The most common method of controlling buoyancy is to fill the pipe with water as it enters the hole. This requires an internal fill line to discharge water at the leading edge of the pull section (after the breakover point). An air line may also be required to break the vacuum which may form at the leading edge as the pull section is pulled up to the rig. The amount of water placed in the pipe is controlled to provide the most advantageous distribution of buoyant forces. Some contractors may choose to establish a constant buoyancy. This can be accomplished by inserting a smaller diameter line into the pull section and filling the smaller line with water. The smaller line is sized to hold the volume of water required per lineal foot to offset the uplift forces.

SITE INVESTIGATION REQUIREMENTS

FOR

LARGE DIAMETER HDD PROJECTS

J.D.Hair&Associates,Inc.

Consulting Engineers

SITE INVESTIGATION REQUIREMENTS
FOR
LARGE DIAMETER HDD PROJECTS

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ABSTRACT

An investigative procedure for generating site characterization information relative to the design, permitting, execution, and certification of horizontal directionally drilled (HDD) pipeline installations is presented by this paper. Concentration is on projects involving large - i.e. greater than 20 inches - diameter pipe. Developed during the last 13 years via conduct of more than 200 studies for HDD employment throughout the continental United States, the investigation process is directed toward defining a particular site's geological, topographical/hydrographical, and geotechnical aspects affecting pipeline placement. The in situ obstacle - i.e. the reason for implementing the crossing in the first place - plus the site's "responses" to HDD construction are also addressed. Means of developing such rationale are then examined through discussion of various investigative/analytical techniques now in use and/or likely to become available in the near future. The paper concludes with a case study illustrating procedural application to a recent, astutely planned, efficiently executed, large diameter HDD installation.

INTRODUCTION

As the number of successful horizontal directionally drilled (HDD) installations continues to expand worldwide, the construction technique is increasingly viewed as the *method of first choice* for an ever widening array of crossing applications. However, even though HDD is becoming an engineerable, i.e. plannable, construction procedure; its' sensitivity to site conditions still remains the major detriment to its' employment. This is especially true for large diameter pipe placements - i.e. those projects entailing carrier pipe diameters exceeding 20 inches which require multiple hole-opening reams plus maintenance of a large diameter downhole bore prior to pull-in. Consequently, for HDD usage to increase beyond present day bounds, its engineering will require better definition of site conditions to:

- enhance/streamline design and permitting procedures
- increase the chances for construction installation success
- augment prospects for the completed facility's long term performance/integrity.

To achieve such ends, the requisite site investigation must provide:

- definition of the obstacle to be crossed. The natural or manmade feature to be negotiated must be characterized in terms of its' existent physical dimensions as well as the possibility for such parameters to change with the passage of time.

- knowledge of conditions which must be transited by the HDD process. Both passive features - i.e. the site's constituency - as well as active factors - i.e. the various responses to the construction process - must be analyzed.

Stated differently, HDD's effects on the site as well as the site's effects on the construction process/completed facility must be assessed in order to adequately engineer and efficiently execute any such project.

Per the foregoing points, and in light of more than a decade's experience in geotechnically engineering over 200 trenchless construction projects nation-wide; this paper summarizes site investigative aspects inherent to all HDD installations - and especially those projects involving large diameter pipes. With much of the discussion extracted from previous publications (Hair, 1993a and 1993b), overall intent is to provide a framework for structuring and improving future HDD site evaluations. A case study, based on a recently completed project, points out several of this continually evolving procedure's crucial aspects.

SITE INVESTIGATION OVERVIEW

Objective of the site investigation inherent to HDD construction (or, for that matter, the exploration involved in any project) is determination and portrayal - i.e. **characterization** - of the location specific aspects relevant to selecting, designing, and executing the installation methodology. To attain such objective, three categories - or classes - of data are produced:

- Class 1. Raw data, i.e. direct *measurements*

- Class 2. Processed data, i.e. *information* stemming from test results or computations performed on Class 1 data

- Class 3. Evaluated data, i.e. rationalized *opinions* - emanating from Class 1/Class 2 results - for input to construction designs, drawings, specifications, bid documents, permit applications, etc.

Study accomplishment responsibility originally rested almost exclusively with the artesian-practioners, i.e. the HDD contractors. However, as trenchless technology became more "engineerable", such responsibility shifted towards the design consultants/owners. At present, since the site study is the foundation for the detailed plans and specifications necessary to effect an HDD installation, the latter group largely shoulders responsibility for the sequentially staged generation of raw, processed, and evaluated information.

Obstacle Definition

First step in the process is definition of the obstacle to be crossed. Basically, two obstacle types are negotiated via HDD:

- Time Dependent. Obstacles such as rivers (alluvial), zones of migrating subsurface contamination, etc. possessing the capability of expanding and/or relocating with the passage of time.

- Feature Dependent. Obstacles such as highway and/or railroad embankments, flood protection levees, environmentally sensitive surface areas, etc. having essentially fixed boundaries.

Primary concern in evaluating either type is determination of the feature's spatial extent. In the former case, such determination must include assessment of the obstacle's boundaries throughout the design life of the HDD installation.

Potamology - the study of rivers - yields a time dependent alluvial obstacle's potential for horizontal displacement and vertical penetration; i.e. the stream's meandering and scouring characteristics during a selected period (Hair, C., 1991). By the same token, some feature dependent obstacles will also exhibit effects with the passage of time - i.e. uncompleted consolidation settlement of a massive highway embankment, integrity maintenance of a flood protection levee, etc. - which must be evaluated. In concert with a site's conditions, a thorough definition of the obstacle to be crossed will therefore dictate the directional bore's geometry plus delineate many of the steps necessary to restore site integrity following HDD completion.

Site Conditions Determination

Selection of the HDD methodology for use on a particular project - plus the procedure's design, permitting, execution, and post-construction follow-up aspects - must be predicated on a thorough understanding of the site's constituency. Since:

- in situ features, both natural as well as artificial, dictate the manner in which HDD construction is configured

- application of the HDD construction process elicits responses from the site's features during both the short and long terms;

site conditions can be divided into two major groups - *passive* and *active*. Because this paper is mainly concerned with the pre-construction investigative aspects of a HDD project, primary emphasis of the following discussions is on the former set of conditions.

PASSIVE CONDITIONS

These are the site's constituency - i.e. its' "makeup"/inplace characteristics - *independent* of whether or not obstacle negotiation will be via HDD. Primary considerations are:

- geological factors

- topographic/hydrographic details

- geotechnical aspects.

In context, such features are expressed as the site's subsurface profile - i.e. its' *stratification*. A thorough understanding of this aspect is the key to effective, project-specific HDD design and execution.

Geological Factors

Chief informational item is an understanding of the site's origin, i.e. how the site came into being. This is important not only to project the site's effects on HDD, but also to plan an effective site characterization study. Understanding the mechanism by which the site was developed - whether by aeolian (airborne), colluvial (gravity), alluvial (river), laucustrine (lake), glacial, or marine (saltwater sea) depositional processes - will forecast the types of materials to be expected as well as the potential for anomalous impediments (boulders, cobble fields, buried logs, stumps, etc.) affecting HDD construction. Geological evaluation thus provides the impetus/background for assessing the obstacle itself.

Topographical/Hydrographical Data

Essential items of information stemming from these considerations are the site's/obstacle's surface configurations. Not only do such data allow definition of the obstacle to be crossed, but rational decisions regarding actual conduct of the construction can be made. Information products include the dry land/underwater configuration of the site/obstacle as well as in situ artificial features/the works of man. Basically, results enable detailing of the obstacle together with a forecast of the efficacy of a HDD installation.

Geotechnical Aspects

Traditionally regarded as the geophysical, or "subsurface conditions", aspect of a site; geotechnical characteristics can be divided into two types: earth material parameters and subsurface stratification. In terms of *earth material parameters*, four principal categories are:

- material classifications
- strength properties
- deformation properties
- groundwater table behavior.

Table 1 lists commonly used procedures for quantifying these factors while Figures 1 and 2 depict typical test results. Standard manuals (AASHTO; ASTM; DA,OCE; and Lambe, 1951) present additional details and test methodologies. *Subsurface stratification* defines the manner in which the earth material parameters are distributed throughout the site. Both such informational items - acting in concert with definition of the obstacle - provide the primary focus for HDD design and construction planning.

TABLE 1
EARTH MATERIAL PARAMETERS
TYPICAL REFERENCES

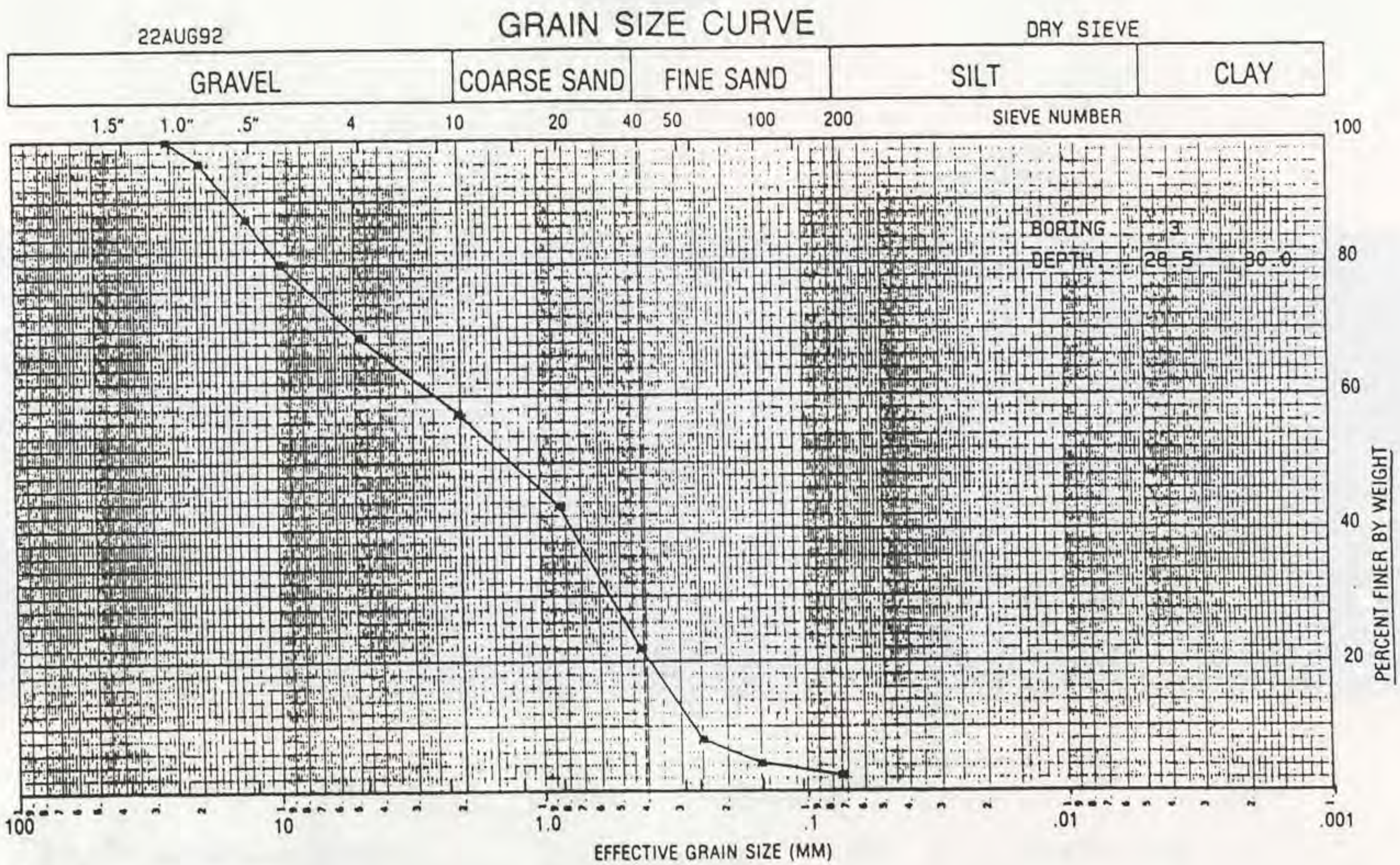
Classifications		**Strengths**	
Unit Weight	EM1110-2-1906	Unconfined Compression	ASTM D-2116
Moisture Content	ASTM D-2216	Unconsolidated, Undrained Triaxial Compression	ASTM D-2850
Atterberg Limits	ASTM D-4318	Consolidated, Undrained Triaxial Compression	ASTM D-4767
Sieve Analysis	ASTM D-422		
Deformations		**Groundwater**	
Incremental Consolidation	ASTM D-2435	Falling or Constant Head Permeability	EM1110-2-1906
Constant Rate of Strain Consolidation	ASTM D-4186	Flexible Wall Permeameter	ASTM D-5084

Note: ASTM refers to *The American Society for Testing and Materials*
EM denotes *Engineer Manual, Laboratory Soils Testing*, U.S. Army Corps of Engineers

ANY RIVER PIPELINE CROSSING LABORATORY DATA TABLE 1 FILE NO.: 94-00

BORING NO.	DEPTH FEET	MOIST %	DEN-PCF WET	DEN-PCF DRY	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	COMPRESSION TEST TSF	COMPRESSION TEST % STRAIN	COMPRESSION TEST PRESSURE-KSF START	COMPRESSION TEST PRESSURE-KSF FAIL	TYPE FAILURE	OTHER TSTNG		MOHS' HRDNS SCALE
1	3 - 5	121	75.0	33.9	107	44	63	0.01	10			YIELD	U		
1	8 - 10	101	84.1	41.9				0.07	10			YIELD	U		
1	13 - 15	84	111.5	60.7				0.07	8	0.82		BULGE	QD		
1	18 - 20	88	89.7	47.8				0.06	10			YIELD	U		
1	23 - 25	77	93.9	53.2				0.08	10			YIELD	U		
1	38 - 40	27	132.1	104.4	33	19	14	1.73	10			YIELD	U,FH	9.11E-08	
1	43 - 45	28	129.6	101.5				1.61	10			YIELD	U		
1	46 - 47		188.8	188.8				1456.64				MULTIPLE SHEAR	U		5 1/2
1	51 - 52		191.2	191.2				686.71				MULTIPLE SHEAR	U		4
1	54 - 55		192.7	192.7				2143.34				MULTIPLE SHEAR	U		6
2	18 - 20	29	108.9	84.6	22	22	0	0.56	10	1.09		YIELD	QD,FH	4.73E-04	
2	28 - 30	64	97.2	59.4				0.53	10			YIELD	U		
2	41 - 43	33	124.4	93.4	45	23	22	0.98	8			SLS(50 DEGREES)	U,FH	3.49E-08	
2	41 - 43							2.01	10			YIELD	REM		
2	43 - 45	30	123.0	94.4				3.02	10			YIELD	U		
2	48 - 50	30	123.5	94.9				3.37	8			SLS(60 DEGREES)	U		
2	51 - 52		193.7	193.7				1123.70				MULTIPLE SHEAR	U		6
2	54 - 55		189.0	189.0				894.80				MULTIPLE SHEAR	U		5
2	59 - 60		187.6	187.6				541.04				MULTIPLE SHEAR	U		4
3	3 - 5	251	62.5	17.8				0.02	10			YIELD	U		
3	13 - 15	38	108.3	78.5	45	27	18	0.16	9			MULTIPLE SHEAR	U		
3	18 - 20	27	112.9	88.7				1.20	10	1.09		YIELD	QD		
3	28 - 30	39	113.4	81.5				0.25	10			YIELD	U,SPT		
3	44 - 45	23	137.0	111.4	36	21	15	1.72	10			YIELD	U		
3	48 - 50	21	127.5	105.7				1.39	10			YIELD	U		
3	53 - 55	34	123.0	92.1				1.78	5			SLS(55 DEGREES)	U		
3	58 - 60	34	122.6	91.6	71	24	47	2.23	8			SLS(55 DEGREES)	U,FH	1.22E-09	
3	63 - 65	32	125.9	95.2				1.76	4			SLS(65 DEGREES)	U		
3	68 - 70	30	130.1	100.2				2.96	6			SLS(65 DEGREES)	U		

FIGURE 1
CLASS 2 DATA
TYPICAL LABORATORY TESTING RESULTS
Classifications/Strengths/Permeabilities



FILE 94-00
FIGURE . . 10

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

FIGURE 2
CLASS 2 DATA
TYPICAL LABORATORY TESTING RESULTS
Granular Soil Particle Size Distribution

Classifications. For HDD projects, principally required are the unit weight/moisture content and Atterberg limits of cohesive clay (colloidal particle size) soils as well as the in situ density (Standard Penetration Test "blow counts") of granular silt, sand, and gravel earth materials. Another key factor for granular soils is determination of the grain (particle) size distribution through sieve analysis. In the case of lithified (rock) materials; unit weight, material hardness - generally by Mohs' Scale of Hardness, and in situ condition - via Rock Quality Designation - are necessary. Table 2 provides Mohs Hardness and Rock Quality Designation details. Finally, determination of earth material electrical resistivity or mineralogical constituency may be necessary if problems - such as high corrosion rates or calcareous sands in an acidic groundwater environment - are anticipated.

TABLE 2

LITHIFIED EARTH MATERIAL CLASSIFICATION PARAMETERS

Rock Quality Designation (RQD)

In situ rock quality is indicated by a modified core recovery ratio known as Rock Quality Designation (RQD). This ratio is determined by considering only pieces of core that are at least 4 inches long and are hard and sound. Breaks obviously caused by drilling are ignored. The diameter of the core should preferably be not less than $2^1/_8$ inch. The percentage ratio between the total length of such core recovered and the length of core drilled on a given run is RQD. Rock quality description as related to the RQD is:

RQD (%)	Rock Quality
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

Mohs' Scale Of Hardness

Original Version		Revised And Expanded Version	
Talc	1		Talc
Gypsum	2	Fingernail	Gypsum
Calcite	3	Copper Penny	Calcite
Flourite	4		Flourite
Apatite	5	Knife Blade	Apatite
Orthoclase	6	Glass	Orthoclase
Quartz	7	Steel File	Vitreous Pure Silica
Topaz	8		Quartz
Corundum	9		Topaz
Diamond	10		Garnet
	11		Fused Zirconium Oxide
	12		Fused Alumina
	13		Silicon Carbide
	14		Boron Carbide
	15		Diamond

Strengths. Shear strength determination via laboratory unconfined compression testing of undisturbed clay and rock specimens usually provides adequate definition for HDD construction. Especially important to analyzing clay strength, though, is determination of sensitivity: the material's initial strength compared to its "remolded" strength. High strength - usually via overconsolidation due to desiccation - clay generally features "healed" prefractures.

Termed "slickensides", these passive condition anomalies pose consequences for stability during the conduct of horizontal drilling (as is described below) plus allow generation of *active conditions*: mainly mud seeps (again, see below). Granular material strength - angle of internal friction, ϕ - can usually be extracted from in situ Standard Penetration Test (SPT blow counts) correlations and/or triaxial laboratory testing.

Post construction assessment of clay strength - for active condition evaluation - generally entails conduct of triaxial shear testing on consolidated specimens. Depending on the evaluation's purpose, detailed pore water pressure measurements should be made during the conduct of shearing. Also, several shearing stress systems - i.e. compression loading, extension unloading, etc. - should be imposed. The resulting data will provide "performance" of the clay strength parameter as consolidation (drainage of pressurized pore water) occurs. Since rock and granular soil strengths are relatively insensitive to consolidation, long term strength parameters for these materials are largely irrelevant.

Once directly measurable strength parameters are determined, they may be used to compute other useful data: bearing pressures, active and passive earth pressures, etc. In this manner not only can the site's ability to "support" the desired construction technique can be analyzed, but many active conditions can be predicted.

Deformations. Construction related - i.e. short term or "immediate" - earth material deformations, both elastic (recoverable) and plastic (permanent), can be assessed through various numerical techniques (finite element, etc.). Moduli determined from unconfined compression and triaxial shear testing should be used. Assessment of longer term, time dependent deformational behavior, i.e. settlement, should be determined through the conduct of incremental or constant rate of strain consolidation tests. Performance of the former type testing - in which a load increment is held through several cycles of "secondary" consolidation - will also allow evaluation of ultra long term deflection, i.e. "creep", characteristics.

Groundwater. Trenchless construction conduct - plus inservice performance of the completed project - will largely depend upon proximity to (whether above or below) the free water surface. Consequently, the potential for fluctuation of the groundwater table - due to natural as well as manmade causes: rainfall, river stage variation, human induced area dewatering, etc. - must be determined. Furthermore, the potential for a perched water table must be assessed: unchecked borehole flow during HDD conduct could jeopardize successful construction completion. Facility design and execution must also consider both total as well as buoyant soil unit weights. Finally, because regulatory bodies are beginning to question the effects of directional drilling on groundwater quality, such factor is now evoking study efforts varying from cursory to extensive. In light of all these considerations, earth material permeability is a parameter which must be assessed.

Normally, the phreatic surface is measured in situ. However, the potential for variation must be derived from review of long term site specific records. Permeability can be determined through laboratory testing; either via direct measurement (falling head, constant head, or triaxial permeability testing) or extracted from consolidation test time rate analysis.

Stratification

After geotechnical material parameters have been defined, the manner in which they are dispersed throughout the site, i.e. the subsurface profile, can then be determined. In essence, earth materials will form two types of interfaces: *material* and *conditional*. A material interface is the demarcation between two different classifications - clay/sand, rock/gravel, etc. - while a conditional interface is the differentiation, based on inplace state, within a particular earth material type - loose/dense sand, soft/hard clay, etc. Also a part of stratification determination is assessment of the possibilities for natural as well as manmade anomalous "impediments" to HDD conduct. Buried logs, stumps, small areal extent gravel pockets/cobble fields, boulders, etc. exemplify natural anomalies. Manmade impediments consist of existing pipelines, sunken barges, bulkhead/bridge pier piling, etc. In essence, determination of the subsurface profile - incorporating the site's geological/potamological and geotechnical aspects - completes definition of the site's passive conditions relative to HDD.

Passive Conditions Effects

Relating passive conditions to the HHD process must be based on an understanding of the "product" produced. In essence, HDD boreholes fall into two classes: "impermeable" cohesive soils and lithified strata produce an *open hole* structure while permeable cohesionless (granular) soils result in a *fluid hole* condition (Hair and Hair, 1988). For the most part, the discrete *open hole* is, in fact, "filled" with slurry: drilling mud, cuttings, etc. In the same vein, the much less distinct *fluid hole* is a linear zone of extremely low density in the penetrated stratum. Of special concern to large diameter HDD installations are passive condition impacts to HDD's latter stages. With this distinction in mind, it is obvious that the site's passive conditions affect all phases of HDD. This is especially true of the latter stages of a large diameter pipe installation. Pilot hole accomplishment, barring contact with anomalous impediments, is possible in virtually all earth material types and conditions. However, certain materials produce difficulties for steering the pilot stem; for reaming the pilot bore - especially the "final" hole enlargement required for carrier pipe pull-back; and for the followon carrier pipe pull-in/construction completion activities. Major sources of pilot bore steering imprecision are:

- pilot stem "skipping" when transitioning the bore from a soft/loose stratum to a hard/dense layer

- pilot bit uncontrollability during bore passage through extremely soft/loose soils.

Hole maintenance problems during reaming are due mainly to "collapse" of weak cohesive soil into an open hole structure and/or sediment "consolidation" in a fluid hole condition. Conversely, boring/reaming relatively strong, impervious clay - especially if slickensided prefractures are present - potentially generates drilling mud surface seeps: see Table 3 and the *Active Conditions* section. Likewise, since carrier pipe pull-in/construction completion difficulties are largely "responses" of the in situ conditions to a HDD installation, they will be detailed in the next section. Consequently, in light of the foregoing factors, passive conditions chiefly affecting successful directional drilling are: gravel constituency - inversely proportional to the ability to ream fluid hole conditions - and rock strength/hardness - impacting cost and installation timing of open hole structures.

In terms of non-lithified soils: because gravel particles are too heavy for entrainment by the drilling fluid (mud), and since their tendency to rotate in place prevents them from being broken up by the reamer bit, they must be physically displaced during hole enlargement. Normally, displacement is radially outward into voids formed by entrainment of finer grained (sand and smaller size) particles. Because naturally dense, high gravel percentage soils contain little entrainable material, insufficient voids are developed during pilot hole accomplishment to permit followon passages by the larger diameter reams. In instances where the pilot hole is sloped (i.e. penetrates the gravel stratum at an angle), gravel particle displacement longitudinally - due to gravity after dislodgement by the reamer - will occur. This is advantageous only if voids exist, or can be formed, in the soils at the hole's vertical curves, i.e. the crossing sagbends. If such is not the case, displaced gravel will collect in these pilot hole "sumps" to form impenetrable blocks. Based on soil particle size distribution, assessment of directional drilling feasibility yields Table 3.

As to lithified earth materials: exceptionally strong and hard rock will hamper all phases of a HDD project. Experience has shown competent rock with unconfined compressive strengths exceeding 12,000 psi and Mohs' Scale of Hardness factors ranging somewhat above 7 can be negotiated with today's technology. However, entry of such materials at depth is usually difficult: the pilot stem bit tends to deflect rather than penetrate. Conversely, directionally drilling poor quality (extensively fractured or jointed) rock - sometimes akin to negotiating gravel/cobble deposits - usually requires large quantities of high quality drilling mud. In any case, overall rock drilling costs are usually high.

TABLE 3
CLASS 3 DATA
HDD ASSESSMENT PARAMETERS

Earth Material Type	Gravel Constituency Range, Percent by Weight	Directional Drilling Applicability
Very soft to hard strength, possibly slickensided (prefractured via desiccation) clay.	N/A	Good To Excellent. Plugging of the annulus surrounding the stem during pilot hole drilling may allow downhole drilling fluid pressurization sufficient to produce mud seeps through slickensides. Also, at depth penetration of strong clay surrounded by considerably weaker and/or looser soils - if not conducted at a sufficiently steep angle and/or into a preformed slot - may result in the pilot stem bit "skipping" along the weak/strong interface. Pilot stem steering difficulties are likely to result during passage through very soft layers.
Very loose to very dense sand with or without gravel traces.	0 to 30	Good to Excellent. Gravel constituency may cause slight pilot stem steering problems. Some steering imprecision may also result during passage through very loose material. Drilling mud - with viscosity, pressure, and volume matched to conditions - necessary for hole maintenance during reaming, especially in the looser strata.
Very loose to very dense gravelly sand.	30 to 50	Marginally Acceptable. Drilling mud characteristics and handling critical for successful horizontal penetration and/or conduct of horizontal/vertical curves. Stratum penetration at an angle normally presents few problems with proper drilling mud. Additional surging will probably be required to clean the reamed hole prior to carrier pipe pull-back.
Very loose to very dense sandy gravel.	50 to 85	Questionable. Horizontal penetration for any appreciable distance, plus conduct of curves, will be extremely difficult regardless of drilling mud quality. Angled penetration to/from a horizontally drillable layer is possible but pilot hole steering may be imprecise.
Very loose to very dense gravel.	85 to 100	Unacceptable. With present technology and experience, horizontal penetration, especially in the denser strata, is almost impossible. Such materials must be avoided or transited at a steep angle.
Rock.	N/A	Excellent to Unacceptable. Softer and/or partially weathered lithified materials offer HDD performance akin to that of hard strength clay. If in a solid state, boring technology - although time consuming and expensive - is available to drill through more competent rock, especially in the weaker horizontal plane. However, penetrating solid rock after passing through non-lithified soil may be difficult due to the pilot bit's tendency to "skip" along the lower hard surface. If in "rounded" cobble form, competent rock is virtually impossible to drill.

Summary - Passive Conditions

On balance, an understanding of a site's *passive conditions* is crucial to HDD success can be negotiated via judicious selection of the bore's geometry/routing in addition to "correct" matching of the drilling procedures and fluid (i.e. mud) to the inplace materials' peculiarities.

ACTIVE CONDITIONS

Broadly defined as the "products" - whether intended or not - of the HDD construction process; this category of subsurface conditions includes: shape/condition of the bored hole (the directional drilling's actual "geometry"); the various efforts/procedures necessary to complete the HDD installation (pull force/torque requirements; carrier pipe buoyancy adjustment; downhole equipment alterations, etc.); response of the passive conditions to the directional boring process (drilling mud surface seeps, deformation/destabilization of surface embankments, flow of groundwater along the soil-pipe annulus, development of underground voids, groundwater quality alterations, etc.); plus short/long term effects on the installed pipe (placement stresses, corrosion potential, loadings/deformations due to future construction at the site surface, etc.) (Hair, 1993b and 1994b). Simply stated, active conditions are the construction *dependent* phenomena at a given location: the site's responses - i.e. behavior - when subjected to drilling plus HDD's performance peculiarities. Because a lengthened construction time and greater physical effort are involved, active conditions are more manifest during large diameter HDD installations. In fact, the diversity and severity of a site's active responses constitute two of the major differences between large and small diameter HDD projects. Consequently, knowledge of *active conditions* is necessary to adequately configure the site-specific *passive conditions* investigation inherent to any HDD project - especially those involving large diameter pipe.

Crossing Geometry

In planning a HDD installation, the geometry of the bore must be matched to the pipe being installed. Minimum radius for subsurface curves (both horizontal as well as vertical, i.e. sagbends) - can nominally be based on 100 feet of bend radius per inch of carrier pipe diameter. However, pipe stress-strain analysis - incorporating inservice pressures combined with induced bending loadings - should be performed. Evaluation of the potential for pipe section collapse during pull-in plus a shorter curvature radius are the primary analytical goals. Furthermore, when conditions permit, any subsurface curves (especially the sagbends) should be slated for execution entirely within the same earth material layer/zone. This latter measure is intended to enhance pilot bore steering precision, and thereby facilitate crossing installation.

Drilling Conduct

Pilot hole boring direction - and consequently the direction (in the opposite way) of reaming/carrier pipe pull-back - should be established by the site's geotechnical and topographical conditions plus the practitioner's (contractor's) experience and expertise. Of importance here is that precision and ease of drilling - i.e. use of steeper penetration angles, more positive control of steering, easier negotiation of adverse subsurface conditions, more efficient handling of problems, etc. - are greater close to the drilling rig. An additional consideration is that the far shore pipe laydown/makeup operation will require a long, narrow work space: makeup/pressure testing of the carrier pipe string should result in a single section to help preclude stopping the pull-back (for pipe joining/coating) and thereby risk not being able to restart it.

HDD's manner/sequencing should also account for existing pipelines, support piling, bulkhead sheeting, etc. whose steel mass may magnetically interfere with pilot bore guidance/positioning instrumentation. For safety, any inservice pipelines should be deactivated/blowndown when construction operations - and particularly pilot hole drilling - are in close proximity. Possible below ground presence of contamination could dictate a drilling fluid monitoring/testing program. In turn, this could affect not only slurry handling and disposal procedures (see the *Drilling Mud* subsection below), but also mandate HDD drilling directions to minimize contact with/generation of "contaminated" materials. Further, if unbalanced hydrostatic forces - due to bank surface elevation differential, a perched water table, high river stage, etc. - are possible anywhere along the bore's length; steps must be planned/taken to control or halt any flow which may develop. Such steps could include the use of weighted drilling fluid; the blinding-off (grout sealing) of intercepted water bearing strata; etc. In this regard, both the short term "during crossing installation" plus long term "inservice" (discussed in detail below) cases must be addressed.

Pilot hole establishment should be "completed" via performance of a downhole survey - see Figure 3. Accomplished using any of a variety of equipment/techniques (horizontal accelerometers, inertial gyroscopes, a simple drilling records compilation, etc.); the "as constructed" borehole geometry picture will:

- detail the drilled crossing alignment's actual position
- generate data for planning the followon reaming and pull-back construction phases
- provide quantitative information for analyzing/solving unexpected problems.

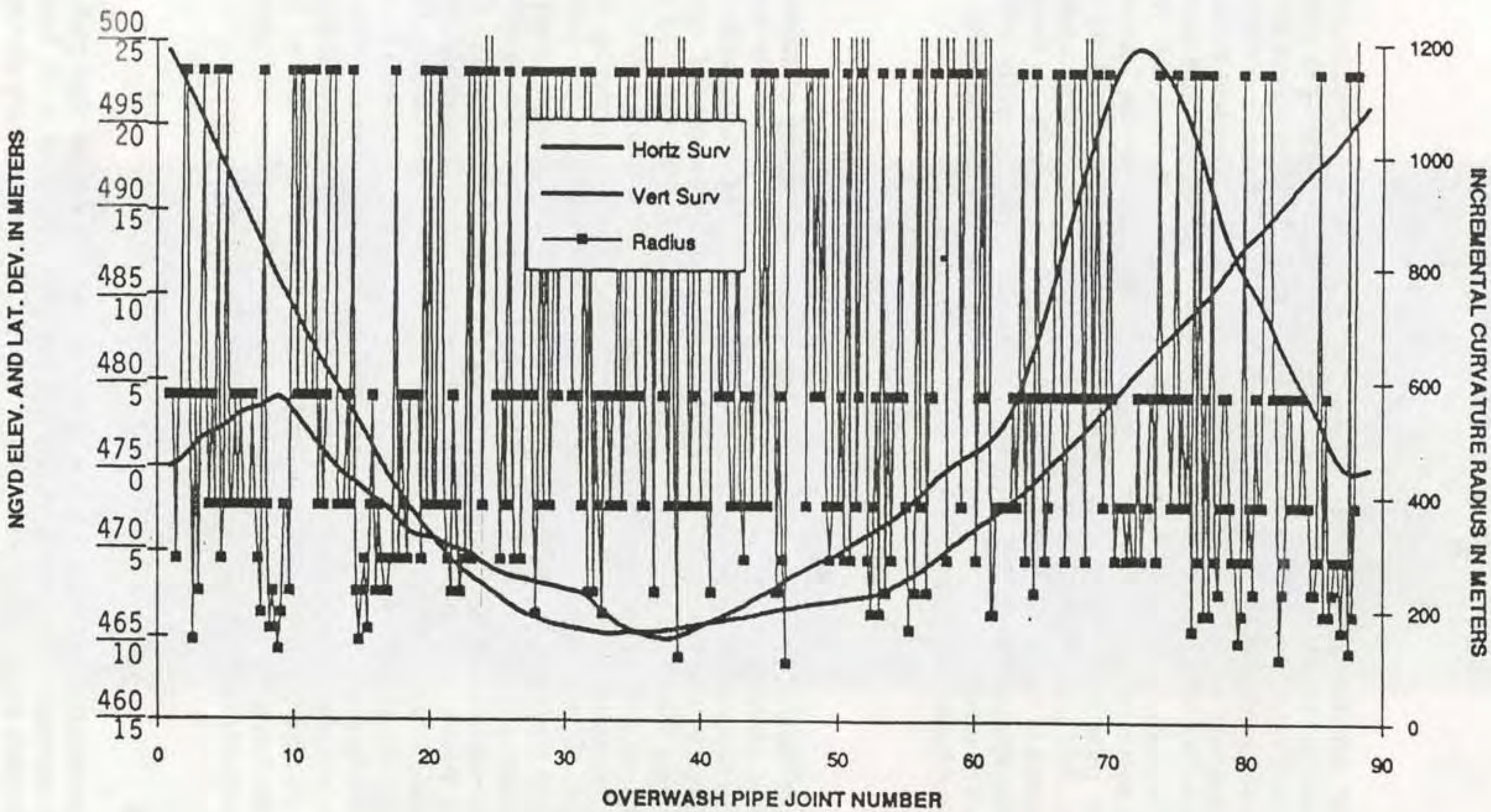
In effect, having such a quantitative geometrical characterization of the site's response to HDD's initial phase could save considerable time/expense in the event reaming/pull-back adversities develop presents the results from - and demonstrates the use of - such a survey.

Especially regarding the "responses" generated by pull-back of larger diameter, i.e. in excess of 20 inches, carrier pipes: recent experience has shown that efficient pulling of a long steel member having a relatively large section modulus and potentially high buoyancy requires considerable forethought (Langlinais, 1992). Specifically:

- the pipe's inherent stiffness will resist its being pulled around the crossing sagbends and any down hole horizontal curves
- considerable flotation will result if the empty, closed end pipe is pulled into the drilling slurry filled borehole.

In consideration of these factors, planning for carrier pipe pull-back - as well as pilot bore reaming - should include modeling of the pulling/torque forces. By basing such study on pilot bore survey data plus results of the stress-strain analysis for sagbend radius determination, accurate and efficient evaluation of possible problems arising during the crossing construction's latter, critical stages will ensue. Pulling/torque force modeling will also help develop procedures and equipment for expediting the pull-back process itself. Controlled variation of the large diameter carrier pipe's buoyancy can be used to assist sagbend negotiation and overcome flotation induced line pull loads. Also, configuration of the downhole pullhead assembly can be adjusted to assist "centralization" of the carrier pipe in the reamed bore.

Finally, while performing any phase of drilling, encountered obstacles/obstructions of limited areal extent (buried stumps, logs, concentrated gravel pockets, etc.) can likely be



NOTE: The *Incremental Curvature Radius* plot depicts the radii of 3 meter nominal length arc segments. These were mathematically determined by combining the horizontal and vertical survey data of 3 adjacent points spaced equi-distant along successive 3 meter incremental lengths of the pilot bore's overwash pipe.

FIGURE 3
CLASS 2 DATA
EXAMPLE PILOT HOLE SURVEY RESULTS
HORIZONTAL/VERTICAL DRILLED TRACE
Combined Horizontal/Vertical Trace Incremental Curvature Radii

bypassed through minimal rerouting of the bore. If such measure proves unsuccessful in obviating adverse/undesirable responses of the drilling's conduct, altering the pilot-reamer bits and/or the drilling mud characteristics may be attempted. Because of numerous adverse effects, excavation should not be utilized - except as a means of last resort - to reduce impediments.

Subsurface Voids

Regardless of whether an open or a fluid hole is in effect; characteristics of the penetrated earth materials in conjunction with the manner by which drilling is executed offer the possibility of "voids" formation. Defined as water/drilling slurry filled spaces (or less dense zones) in the penetrated media; voids are possible during all phases of the construction process: while drilling is in progress; as the result of carrier pipe pull-in; and subsequent to the crossing's placement. *During-drilling* voids stem mainly from creation of downhole fluid (drilling mud) seeps at the ground surface - see the *Drilling Mud* subsection below - plus uncontrolled ablative erosion of the soils surrounding the directional bore. *Pull-in* voids are the "annular" spaces between the carrier pipe and surrounding soils primarily occurring when an open hole structure is in effect. *Post-placement* voids are the potential consequences of groundwater flow through soil-pipe annuli or any other hydraulic paths - natural or artificial - in the installation media. Chief manifestation of these latter active subsurface conditions is cavity formation due to dissolution of such chemically active and/or water soluble materials as calcareous sand, lime rock, weakly cemented (aeolian deposited) loess, etc.

Site Integrity

Completion of the drilled installation must address the possibility of subsurface void occurrence. In essence, physically and/or statutorily mandated restoration of site integrity may necessitate sealing the annular space between the carrier pipe and surrounding soil. Depending on a variety of factors, i.e. the (above mentioned) possible presence of contaminated soil/groundwater zones, the likelihood of imbalanced hydrostatic forces, the need to preserve groundwater quality; etc.; efforts to eliminate this type of active subsurface condition could range from merely plugging the bore's surface penetration points to grouting the annular space's entire length. Since successful HDD is heavily dependent upon the drilling fluid's characteristics (see the *Drilling Mud* subsection below), establishing a viable grout seal more rigorous than a surface penetration plug will require: a considerable design effort, a high degree of contractor expertise, and a comprehensive field inspection program. Conversely, grout plugging the bore's drill rig side surface penetration can reasonably (though not absolutely) be achieved through adding Portland cement to the bentonite drilling fluid during the final 50 to 100 feet of carrier pipe pull-in. Plugging the pipe side (far shore) surface can be accomplished by introducing small diameter pipes for 25 to 50 feet into the annulus - via either attachment to the carrier pipe "tail" or forward thrust insertion subsequent to pull-in completion - and then injecting Portland cement-bentonite grout as such tubes are withdrawn. In any case, addressing loss of site integrity - whether void induced or caused by other factors outside this paper's scope - requires considerable engineering and practitioner input.

Installation Timing

While the construction's critical stages - final reaming and carrier pipe pull-in - are in progress; sufficient contractor personnel should be available to allow a 24 hour per day operation. Uninterrupted, around-the-clock activity is normally not necessary during the other phases of horizontal directional drilling. Preventing untenable HDD responses - mainly borehole loss because of "avoidable" delays and/or operational errors induced by personnel fatigue - is the primary intent of rigorous execution during the job's crucial periods.

Drilling Mud

Especially important to successfully overcoming both active as well as passive conditions is the contractor's correct "handling" of the drilling mud's density, viscosity, pressure, and quantity. Although drilling through clay "spontaneously" creates a slurry; "manufactured" drilling mud is necessary when penetrating silt, sand, and (especially) gravel/rock. For this reason, contractor staffing should include an experienced mud engineer to minimize the chance of borehole loss due to the use of an inappropriate drilling fluid.

During all boring operations, drilling fluid return should be maintained - if at all possible - to preclude mud pressurization downhole. However; experience elsewhere with horizontally drilled installations indicates maintenance of drilling fluid circulation - and prevention of downhole pressurization sufficient to "fracture" drilling mud into the stronger, more brittle strata - may not be possible once a series of granular/colloidal, i.e. pervious/impervious, materials is transited (Hair, J., 1991). Consequently, mud seeps onto the ground surface may not be preventable. Although the possibility of this active condition's occurrence may be slight, the contractor should still be prepared to perform clean-up.

Also important to drilling mud containment are the "pits" at both ground surface penetration points. Intended to collect and hold returns prior to recycling and/or disposal, these usually consist of bermed/lined excavations extending below existing grade. Based on surface soil permeability, a synthetic membrane or imported earth material liner may be required: pits in pervious silts/sands may be bounded by either plastic/rubber sheeting or imported clay compacted in place. Conversely, naturally impervious clay soil can generally accept pits "lined" with scarified/recompacted inplace earth material. Furthermore, below ground side slopes plus dimensions of above ground pit edge berms require design forethought: types and strengths of the near surface soils plus location(s) of the groundwater table are the chief factors in such planning. Finally, proper closure of the pits must be accomplished. This can be done using either imported borrow or onsite earth material free of drilling slurry.

The concluding "response condition" aspect of drilling mud is its proper disposal. Recent experience has been that public agencies closely monitor construction sites when such fluids are involved. While not normally toxic; particulate material constituting the mud may cause environmental distress to wildlife, create an unsightly mess, plus subject the owner/contractor to fines and penalties if not properly disposed. For this reason - and especially if "contaminated" stratification is to be transmitted; detailed drilling fluid handling/testing procedures, plus intended disposal methodology/site location(s), should the established prior to construction.

Summary - Active Conditions

Recognition of *active conditions*, coupled with better definition of these nontraditional "site response" aspects of HDD, will significantly advance the overall procedure's chances for success in an increasingly wider array of applications and locations. In the final analysis, the character and extent of *active conditions* are the primary factors differentiating large diameter HDD installations from smaller sized placements.

SITE CONDITIONS DETERMINATION

In terms of characterizing a specific site, both passive as well as active conditions must be stated and presented in a manner which will allow efficient design and execution of the HDD methodology. The three-phased procedure presently in use involves:

- Phase I. Review of available published information
- Phase II. Conduct of field exploration/laboratory testing to produce the necessary Class 1 and Class 2 data
- Phase III. Performance of various engineering evaluations to generate the required Class 3 data inherent to project design, permitting, execution, and certification.

The following paragraphs concentrate on *passive conditions* determinations. *Obstacle definition* and *active conditions assessment* are largely outside this paper scope.

Published Data Review

As detailed in the *Site Conditions* section, overall products of this phase amount to assessment of what should be expected at the site in terms of HDD design as well as project permitting. Additionally, results allow preliminary assessment of the obstacle plus configuration of the followon plan to physically investigate the site.

Site Investigation

Composed of surface as well as subsurface determinations, this investigative phase will produce the Class 2 data necessary to fully define both the site as well as the obstacle. Additionally information for permit acquisition will ensue. Topographic/hydrographic study results will stem from traditional survey means plus incorporation of emerging technologies: Global Positioning Systems (GPS), satellite mapping, side scan sonar surveying, etc. Ideally, results should be used to configure the subsurface investigation so that a more rational site exploration may be conducted.

The subsurface condition investigation is directed at determining:

- Material Interfaces. Differentiation between different types of earth materials
- Conditional Interfaces. Differentiation between different states of a single earth material.
- Inplace Anomalies. Discrete inclusions of dissimilar and/or conditionally different materials from within the enveloping earth material mass.

Definition of such items, when brought together in the context of one another, produce a subsurface profile. In turn, such profile enables assessment of the obstacle's bounds - both time dependent as well as feature dependent - plus the site specific efficacy of the HDD process. Overall results are the "design/construction" specifications inherent to performance of HDD together with forecasting of the active conditions to be expected onsite. Concerted presentation of such information will allow efficient execution as well as an expeditious pre-construction permitting/post-construction certification process.

Basically, two techniques are available: intrusive (i.e. borings, penetometer, soundings, etc.), and non-intrusive (i.e. reflective-refractive surveying, ground penetrating, radar, etc.) The remainder of this paper concentrates on investigative techniques/procedures for determining *passive subsurface conditions* inherent to an effective large diameter HDD site characterization.

Field Exploration. At the present time, this mainly involves vertical borings to produce specimens for physical testing, see Table 4 for a list of field sampling specifications plus Figures 4, 5, and 6 for examples of drilling logs. Borehole conduct procedure(s), spacing, depth, and sampling frequency generally depend on a project's extent and the subsurface profile's potential for variation (as defined by the previously mentioned geological/potamological evaluations). Material properties of clay and rock are determined via securing undisturbed test specimens. Granular materials - silt, sand, gravel - are subjected to in situ density determinations (primarily Standard Penetration Testing) which also produce samples for laboratory classification (mainly grain size analyses). Of particular concern to exploring granular soils is that a hydraulic gradient outward from the borehole is maintained at all times: an inward gradient risks "quickening" the in situ soils to produce a false sense of what is actually there. At any rate, material and conditional interfaces are then established via interpolation between boreholes.

TABLE 4
SUBSURFACE CONDITIONS DETERMINATION
TYPICAL REFERENCES

Standard Penetration Test	ASTM D-1586
Thinwalled Tube Sampling	ASTM D-1587
Rock Coring	ASTM D-2113

Note: ASTM refers to *The American Society for Testing and Materials*

Although not yet extensively employed, non-sampled intrusive procedures (penetrometer soundings, cross-borehole electrical resistivity/conductivity and shear wave analyses, etc.) as well as non-intrusive, near surface geophysical techniques (reflective/refractive surveying, sub-bottom acoustic profiling, ground penetrating radar, etc.) are possible candidates to expand field exploration utility. Generally speaking, these exploration methods can enhance data from boreholes by providing a more precise definition of material and (hopefully in the not to distance future) conditional interfaces. In essence, expansion of a sampled borehole program via conduct of numerous non-sampled soundings and/or non-intrusive examinations will improve site characterization efforts. However, drawbacks to using these non-traditional exploration means - regulatory considerations requiring site surface integrity restoration, the lack of physical specimens, etc. - will continue sampled boreholes as the cornerstone of any field investigation.

Laboratory Testing. As detailed in the *Passive Conditions* section plus Tables 1 and 2; earth material parameter determination relies heavily on laboratory testing. In contrast to field procedures, laboratory evaluation offers better control of the test conditions plus the ability to impose a variety of stress systems. In this manner, the site's overall "performance" - its' active conditions during HDD as well as its' post-construction responses - can be better simulated. By contrast, many passive conditions aspects are more precisely defined through field procedures: the disturbance associated with sampling any non-lithified earth material is not a factor. Therefore, a complete investigative program should be based on laboratory testing results in concert with data from field procedures.

Engineering Evaluation. Based on field exploration/laboratory testing results; *passive* engineering evaluation provides the key to both the site's characterization as well as the construction's performance assessment. By and large, site specific passive conditions are used in preconstruction analyses to forecast the followon *active* subsurface conditions. Engineering

LOG OF BORING

PROJECT: Any River Pipeline Crossing
Anywhere Feasible

FOR: Any Client
Any City

BORING 1
FILE 94-00
DATE 28 Jan 94
TECHNICIAN US

RQD Rock Quality Designation
*6" Diameter Sample Obtained

Surface Elevation 3.0 Feet, NGVD

UNDISTURBED SAMPLE — STANDARD PENETRATION TEST — BORING DEPTH 55 Feet

DEPTH FEET	SAMPLES	Description		RQD
0			Cement-bentonite grout backfill top 25 feet	
5	Undisturbed	Very soft dark gray organic clay		
10	Undisturbed	Very soft dark gray slightly organic clay		
15	Undisturbed	Very soft gray clay with organic matter and sand streaks		
20	Undisturbed	Very soft gray slightly organic clay		
25	Undisturbed	Very soft gray slightly organic clay		
30	Standard Penetration Test	Firm tan sand with gravel traces Penetration Resistance	22 blows per foot (5/9/13)	
35	Standard Penetration Test	Dense tan sandy gravel* Penetration Resistance	41 blows per foot (10/19/22)	
40	Undisturbed	Stiff light gray very silty clay		
45	Undisturbed	Stiff light gray silty clay ←Rock encountered at 45½ feet Gray schist	71% recovery (black diamond bit)	62
50				

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FF18125

FIGURE 4
CLASS 1 DATA
TYPICAL FIELD EXPLORATION
Log of Onland Boring

LOG OF BORING

PROJECT: Any River Pipeline Crossing
Anywhere Feasible

FOR: Any Client
Any City

BORING 2
FILE 94-00
DATE 28 Jan 94
TECHNICIAN US

*6 inch diameter sample obtained
(SLS) Slickensided Sample

Nominal River Stage 2 feet, NGVD

UNDISTURBED SAMPLE — STANDARD PENETRATION TEST — BORING DEPTH 60 Feet

DEPTH FEET	DESCRIPTION
0	Casing top 4 feet above water Approximate water depth 14 feet
5	
10	
15	
	Mudline — Cement-bentonite grout backfill top 25 feet
	Very loose gray silty fine sand with 1 inch clay layers
20	
	Firm tan sand Penetration Resistance 18 blows per foot (6/8/10)
25	
	Medium gray slightly organic clay
30	
	Very dense tan sandy gravel* Penetration Resistance 50 blows per 8½ inches (20/18/32 for 2½ inches)
35	
	Very dense tan coarse sand Penetration Resistance 25 blows per 5 inches (10/25 for 5 inches)
40	Clay encountered at 40½ feet
	Very stiff light gray and white clay (SLS)
	Very stiff light gray and white clay
45	
	Very stiff light gray and white clay (SLS)
50	

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PP18125

FIGURE 5
CLASS 1 DATA
TYPICAL FIELD EXPLORATION
Log of Inriver Boring

LOG OF BORING

PROJECT: Any River Pipeline Crossing
Anywhere Feasible

FOR: Any Client
Any City

BORING: 3
FILE: 94-00
DATE: 28 Jan 94
TECHNICIAN: US

*6" Diameter Sample Obtained   Surface Elevation 3.0 Feet, NGVD

UNDISTURBED SAMPLE   STANDARD PENETRATION TEST   BORING DEPTH 70 Feet

DEPTH FEET | SAMPLES

0

Cement-bentonite grout backfill top 25 feet

Very soft dark gray peat with clay traces

5

Very loose gray clayey sand
Penetration Resistance 2 blows per foot (2/1/1)

10

Very soft gray clay with 4 inch sand layer

15

Firm gray fine sand with clay traces

20

Loose tan gravelly sand*
Penetration Resistance 6 blows per foot (2/3/3)

25

Very dense tan gravelly sand*
Penetration Resistance 50 blows per 11 inches (7/17/33 for 5 inches)

30

Soft gray clay with gravel traces

35

Very dense tan sand
Penetration Resistance 30 blows per 5½ inches (30 for 5½ inches)

40

Stiff gray clay
Penetration Resistance 18 blows per foot (5/7/11)

45

Stiff gray clay with sand streaks

50

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FIGURE 6
CLASS 1 DATA
TYPICAL FIELD EXPLORATION
Log of Onland Boring

"products" at this stage are germane not only to designing HDD conduct, but also necessary to obtaining construction permits and planning reduction/mitigation of HDD's few adverse effects.

CASE STUDY

Constructed during September 1994; a large diameter HDD pipeline crossing of the Neches River in southeastern Texas graphically illustrates the value of a thorough site investigation. Study conduct - under auspices of the pipeline's owner, Transcontinental Gas Pipe Line Corporation (TGPL) - was in late 1993/early 1994 (Hair, 1993/1994 and 1994b). Pertinent findings - plus the uses to which they were put - are described below.

Project Description

Impetus for this pipeline crossing replacement was the Neches River's long-standing impacts on the originally installed facility.

Location. Crossing site positioning - approximately 6 air miles east of the community of Lumberton - is astride the border between Hardin and Jasper Counties, Texas. Further site location details are depicted on Figure 7.

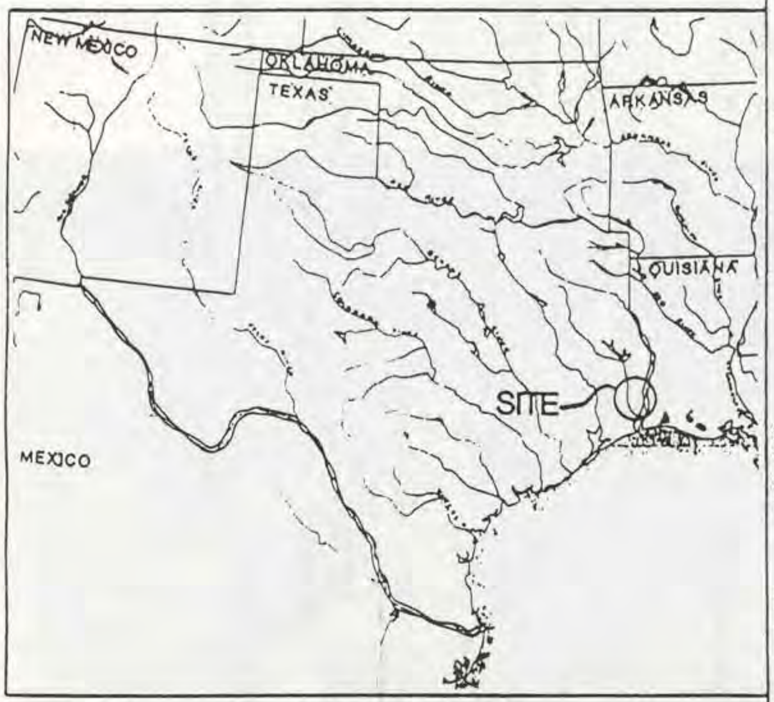
Existing Crossings. Several high pressure natural gas pipelines - plus highway bridges and electrical power transmission lines - traverse the Neches River at, and in the vicinity of, the investigated area. The crossing under study - originally consisted of a single 30 inch nominal diameter pipe. Placement was via conventional cut-and-cover dredging more than two decades ago.

During the period of their existence, the Neches has had varying effects on all inplace crossings. In TGPL's case, alluvial activity - scouring (channel deepening), bank edge washing (slope erosion), meandering (course migration), and alignment relocation (channel repositioning) - has periodically affected the pipeline at several discrete points. Such events resulted in installation of channel training devices - mainly pile supported flow diversion fences. Cumulative outcome was that at least 3 sections of the inservice line in/near the Neches River continued to be of concern: one of these was in the river proper while two were in "subchannels" developing across the southwestern bank's surface. Figure 8 portrays project site specifics.

Replacement Crossing. Since activity by the Neches was ongoing - and because the crossing right-of-way's inclusion 20 plus years ago in the federally managed *Big Thicket National Preserve* virtually prohibited further inchannel remediation construction; TGPL decided to reinstall the partially exposed facility to a more secure configuration. In accomplishing this, the like diameter replacement's positioning and geometry - i.e. the new crossing's horizontal extent and vertical penetration within a slightly offset right-of-way - was chosen to:

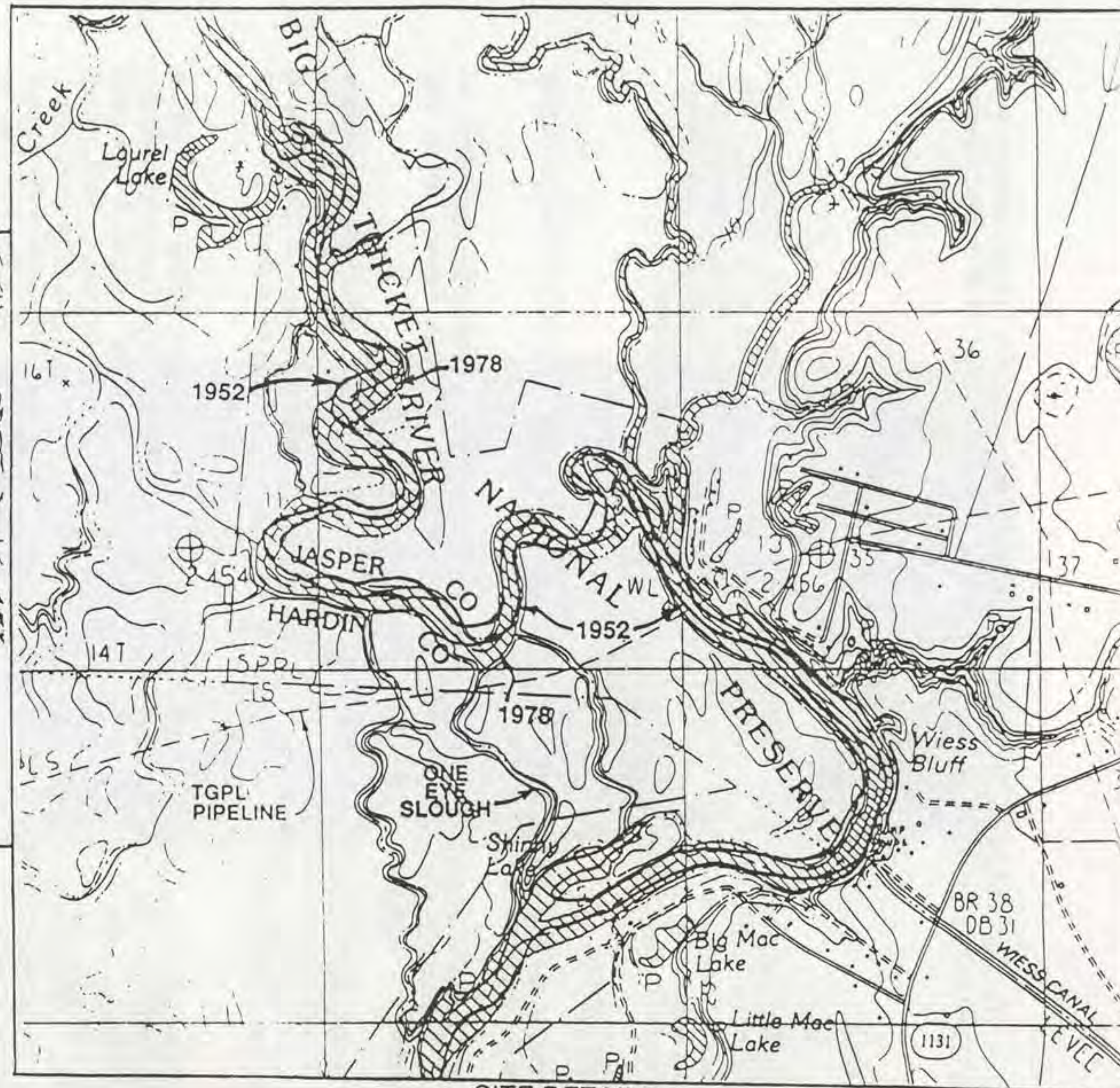
- avoid the river's future meander/scour activity plus
- minimize disturbance of the area's environmental aspects which are largely contained in the federally controlled Big Thicket National preserve.

Preferred reinstallation method was HDD. Overriding objective was a channel impact free, environmentally compatible river crossing not subject to alluvial activity disturbance.



SITE LOCATION

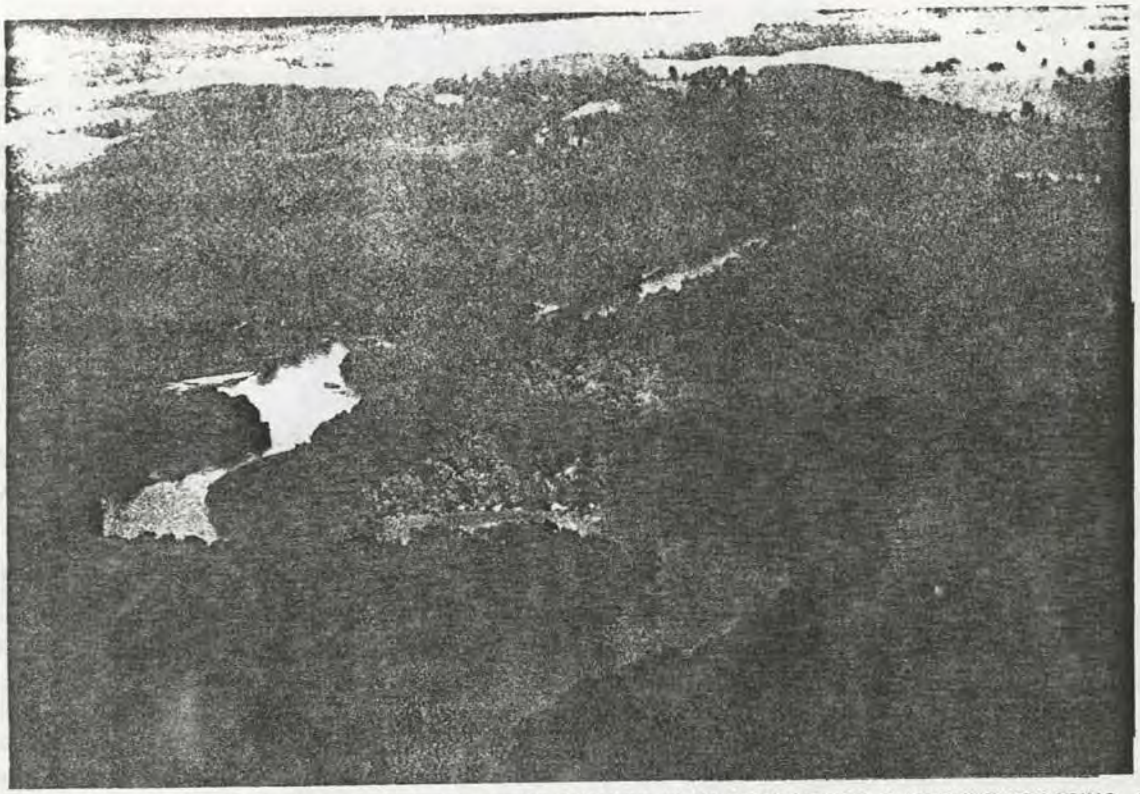
SCALE: N.T.S



SITE DETAILS

SCALE 0 1000 2000 3000

FIGURE 7
CLASS 1 DATA
PROJECT SITE SETTING
Pertinent Top[illegible]aphic/Hydrographic Features



ABOVE. LOOKING EASTWARD AT THE NECHES' ACTIVE CHANNEL UPSTREAM OF TGPL'S PIPELINE CROSSING. THE DEVELOPING "ENTRANCE" TO ONE EYE SLOUGH PLUS SUCH AUXILIARY COURSE'S TRANSITING OF THE PIPELINE ALIGNMENT IS APPARENT NEAR THE COURSE BENDWAY DEPICTED IN THE CENTER FOREGROUND.

BELOW. DETAIL OF THE SOUTHWESTERN BANKS AUXILIARY COURSE "MOUTH" (DEVELOPING ENTRANCE" UPSTREAM OF THE PIPELINE CROSSING. THE PHOTOGRAPHS' ORIENTATION IS NORTHWARD. NOTE SIZE AND CONDITION OF THE ONE EYE SLOUGH SUBCHANNEL.

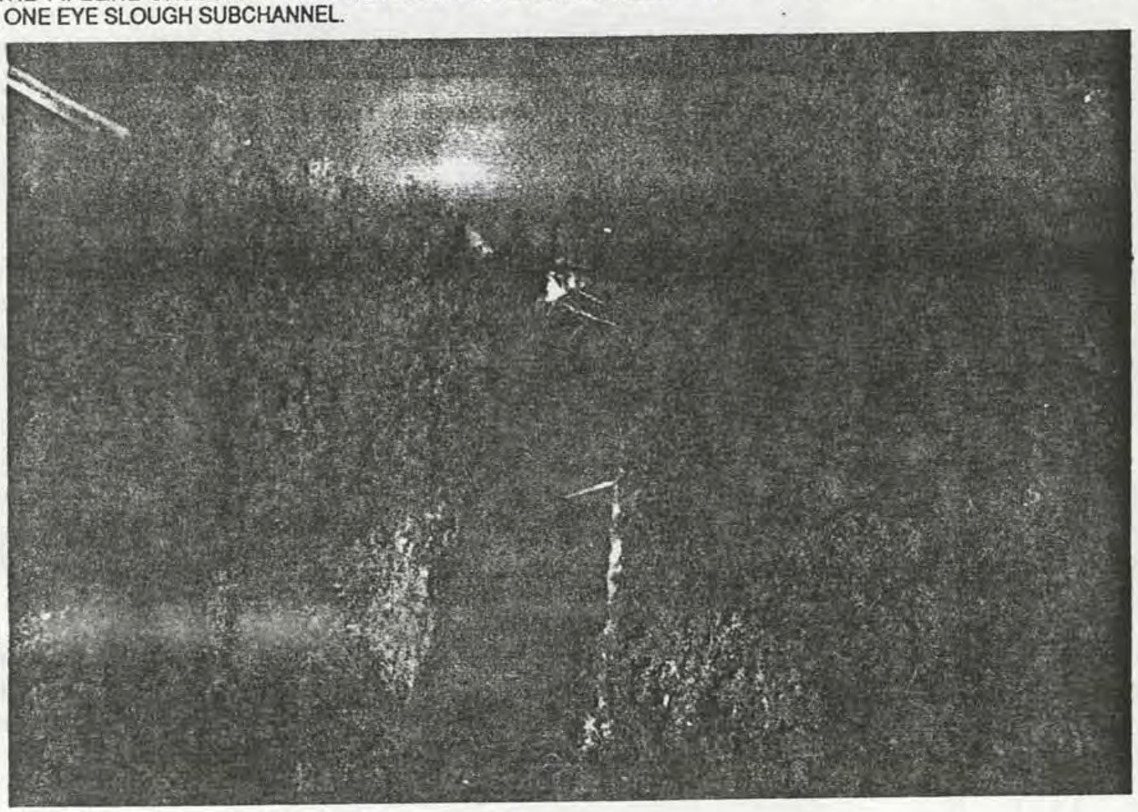


FIGURE 8
CLASS 1 DATA
SITE CONDITIONS
Views of Potamological Aspects

Site Investigation

Due to the rather stringent permitting requirements inherent to exploring/constructing in federally mandated preserves; this particular study featured a two-stage execution. Stage I involved preliminary definition of the river's reconfiguration/relocation possibilities together with development of data for structuring/permitting the followon surveying and field exploration efforts. Based on these findings, Stage II encompassed the geotechnical field work, laboratory testing, and engineering evaluation to validate Stage I's pronouncements plus plan, permit, and implement the replacement crossing. Pertinent findings from both study phases are jointly presented in the following section.

Site Surface Conditions

The Class 1 data base for this study segment consisted of: geological/topographical maps (both current as well as archival); hydrographic surveys; and engineer site reconnaissance observations. Analysis yielded the following Class 2/Class 3 data.

General. At the latitude of the crossing site; the partially flow managed, nominally navigable Neches River is positioned against the eastern edge of its' floodplain: a 4 to 8 mile wide, generally north to south oriented, valley. Formation of the valley was by alluvial scouring of ancient, highly consolidated, Marine (saltwater) deposited very stiff to hard strength clay. Alluvial sedimentation, in thickness sometimes exceeding 100 feet, now partially fills the valley. Elevation of the present day valley floor near TGPL's site varies between 10 and 15 feet, National Geodetic Vertical Datum (NGVD). Valley edge "bluffs" rise 25 to 30 feet higher. Numerous supplemental channels, cut-off old courses, and meander scrolls (curvilinear plan ridges/swales) dominate near-site topography beside the river's relatively convoluted/tortuous main channel. Such alluvial configuration and bank surface features indicated the Neches's past/present propensity for horizontal activity: spontaneous repositioning across its valley floor.

Presently, the Neches River serves as the distributary for B.A. Steinhagen Lake, a quarter century old manmade reservoir. The flow management/flood constraint afforded by operation of its dam - positioned approximately 35 air miles north of the site - have seemingly reduced the Neches's previously demonstrated tendency for course migration. However, as evidenced by TGPL's inplace crossing, channel section alteration/relocation - seemingly driven by flood induced scour penetration/bank edge ablation - is still a significant factor.

Channel. The pipeline right-of-way bisects a 1000 foot long, convex southwestward, slight "bend" in a mostly straight (for the Neches), 3200 foot long, northwest-southeast oriented, course reach. Flow direction in the normally 220 foot wide by 12 to 15 foot deep *active channel* - usually water-filled component of the 6000 to 9000 foot wide *gross channel* or *meander belt* - is southeastward.

About 800 feet upstream (northwestward) of the right-of-way, the active channel describes an abrupt 90 degree bendway from the southwest: this reach segment also includes the remnants of a "cut-off" meander. Significantly, for approximately the next 3500 feet farther upstream, the Neches' convoluted course "parallels" - at distances varying between 100 and 1000 feet - the pipeline right-of-way's southwestern bank alignment.

Roughly 2400 feet downstream of the pipe's active channel crossing; the Neches executes a much less severe, 180 degree bendway oriented convex to the southeast. Downstream of such bend, the minimally convoluted - though obviously still active - river assumes a southwestward strike essentially parallel to, but almost 2000 feet removed from, TGPL's pipeline.

Salient aspect of this particular alluvial configuration is that numerous *auxiliary courses* - in varying stages of evolution - connect the two active channel reaches "paralleling" the right-of-way's upstream and downstream edges. Of these *subchannels*, three are decidedly more prominent than the others: the one most closely positioned to the pipeline's active channel crossing point is about 1000 feet to the southwest while the farthest is more than 2700 feet away. The pipeline was periodically exposed in all three: severest of such activity was at the "rapidly" developing central *auxiliary course* locally termed *One Eye Slough*. Of import is that training fences installed across the upstream ends of the southwestern bank's central and eastern *subchannels* had been disrupted/circumvented by the Neches: the most extensive of such damage was at *One Eye Slough*.

Banks. Both faces of the active channel rise to between 6 and 8 feet above the normal water surface to form natural levees: in this case 1 to 2 foot high, shallow landwardly sloping, course paralleling ridges sedimented from overbank flooding.

Landward of the left descending bank's - i.e. the northeastern edge's - natural levee, the ground surface was under development. A channel paralleling road, houses, etc. were in position on the approximate 800 foot distance between the river and the alluvial valley edge bluff's toe. This seemingly well drained area was not part of the *Big Thicket National Preserve* and therefore not under jurisdiction of the *U.S. National Park Service*.

Conversely, the opposite - i.e southwestern - bank's surface was swampy and featured the above mentioned evolving *auxiliary courses* plus extensive meander scrolls: ridges (former natural levees) and swales denoting the "wake" of the Neches's laterally migrating ancient channel. Westward extent of the alluvial valley floor exceeds $4^1/_2$ miles. Obviously, this bank's surface was generally subject to inundation at high river stage: during an engineer reconnaissance, flowing water was observed in several sloughs plus the developing *subchannels*. Manmade features-of-note were an electrical power transmission line obliquely intersecting the right-of-way. The preserve's heavily timbered surface offered extremely poor trafficability: a swamp buggy/pontoon trailer was needed for positioning exploration equipment there. Access permitting for the investigation was time consuming.

Site Subsurface Conditions

Development of the geotechnical Class 1 data base, production of the Class 2 data results, and several of the pertinent Class 3 data pronouncements are stated in the following paragraphs.

Field Exploration. For this project, 6 soil sample borings - all executed either onland or inswamp - were accomplished in March 1994. Ostensibly spaced on 800 foot centers, borehole locations were adjusted via onsite coordination with the *U.S. National Park Service*. As a result, one of the planned borings was eliminated when it was determined that equipment access would probably cause surface damage exceeding the value of the subsurface information produced. Relative positioning is shown by Figure 9.

Full depth advancement of each nominally 4 inch diameter boring was via the rotary washbore method. Drilling termination was dependent upon penetrating to between 90 and 130 feet. This depth range resulted in definition of the subsurface stratification affecting river activity as well as HDD performance.

Sampling depended on the type of soil encountered. High quality undisturbed - i.e. suitable for laboratory strength testing - specimens of cohesive (clay) material were obtained with a thinwall steel Shelby tube. Granular/semi-cohesive (sand/silty clay) soil classification samples and/or strength testable plugs were extracted via the Standard Penetration Test (SPT blow counts). Full depth sampling of all borings was on 5 foot centers and/or at change of strata. Tabularized field work details are in Table 5.

TABLE 5
FIELD EXPLORATION
SCHEME

Boring Number	Borehole Location	Total Depth (Feet)
1	Onland	90
2	Inswamp	130
3	Inswamp	120
4	Inswamp	130
5	Inswamp	omitted
6	Inswamp	130
7	Onland	100

Laboratory Testing. Immediately upon recovery, all samples were field classified and then prepared for transport to the testing laboratory. There, each undisturbed Shelby tube specimen and SPT plug was lab classified and then subjected to strength, unit weight, and moisture content evaluation. The SPT classification samples were analyzed for grain size distribution. In sum, laboratory evaluations included: 44 unconfined compression tests (each with a unit weight/moisture content determination) plus 98 dry sieve analyses.

Compression testing yielded subsoil shear strength information. Unit weight/moisture content evaluations and sieve analyses results provided more precise subsoil classifications than obtainable through field methods. Taken together, findings of all such laboratory work were used to confirm the subsurface stratigraphy's relationship to alluvial activity potential as well as its suitability for HDD.

Subsurface Stratification. Although undetected anomalies (gravel pockets, buried logs, etc.) are possible, generalization of the stratification derived from the field exploration/laboratory testing is graphically portrayed on Figure 9. Such layering - explained in detail on Table 6 - typifies Neches River alluvial valley subsurface conditions at this latitude. Marine material - the site's foundation stratum - constitutes a coastal prairie terrace formation.

Groundwater. Immediately adjacent to the active channel/subchannels, the water table is usually denoted by the Neches River's stage. By about 100 feet landward, free water is at - or above - the bank surfaces. Furthermore, during periods of rain and/or flooding the water table can substantially exceed such levels. Consequently, when critical to design and/or construction (computation of trench backfill weight, estimation of pipe flotation, design of trench excavation dewatering, construction access planning, etc.), the most adverse groundwater table condition was considered. In the majority of cases, this occurred when the phreatic surface was at/above ground level.

Potamological Analysis

Salient Class 3 data - in light of crossing site geologic history and topographic-hydrographic conditions - are the Neches' continual impacts on TGPL's line plus the stream's potential for such alluviation in the future.

River Regimen. Spontaneous relocation and/or reconfiguration of the Neches River - at this site, a **relatively unstable course** - was judged to stem from *active channel incremental displacement* during the coming 50 years. This behavior should be driven both by *maturation* of the existing active channel as well as the classical *bending process*, (see Hair, C., 1991 for a

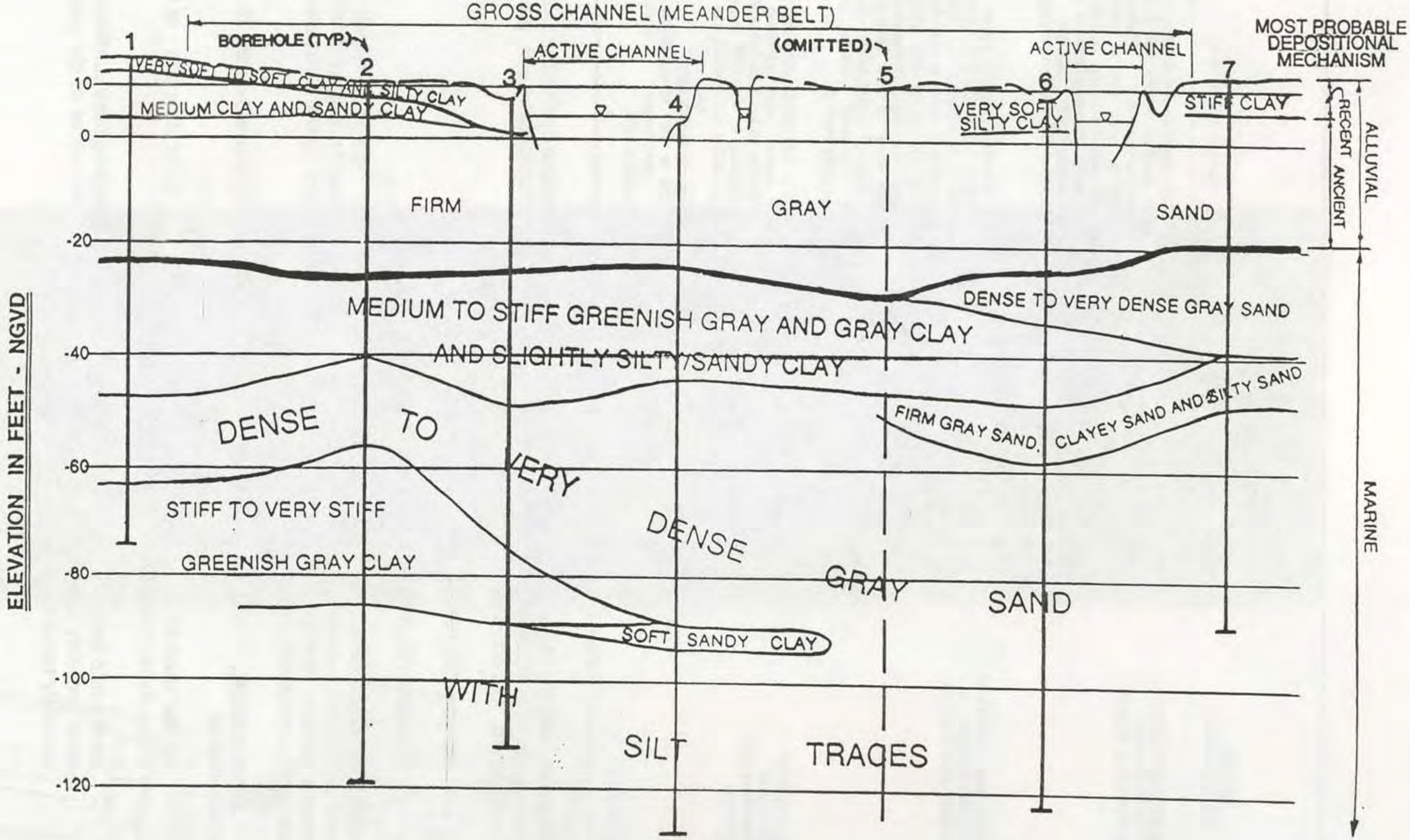


FIGURE 9
CLASS 1, 2, & 3 DATA
SUBSURFACE PROFILE
Geotechnical Field Exploration, Laboratory Testing, and Engineering Analysis Results

TABLE 6
CLASS 3 DATA
SUBSURFACE STRATIFICATION

Stratum Nomenclature	Inclusive Elevations - Feet, NGVD	Stratum Description
<u>Recent Alluvium.</u> Deposition from/in the Present-day Neches River gross channel.	Bank Surfaces (14/10) to 12/-4	Low rise natural levees of clayey sand are in position beside the various channels/subchannels. Underlying material is very soft to medium strength clay containing pockets of loose sand. This overall stratum was not encountered beneath the replacement alignment's northeastern end.
<u>Ancient Alluvium.</u> Deposition from/in remote (previous) Neches River gross channels.	From The Above to -21/-29	Firm sand with pea gravel traces. An intermitten, 4 to 10 foot thick "crust" of medium to stiff strength clay together with pockets of loose to very dense sand and soft strength clay are also featured. The clay's higher strengths are the result of past desiccation which induced slickensides: drying crack "fractures" which have subsequently "healed".
<u>Marine.</u> Deposition in a Pleistocene (late Quaternary) age saltwater sea.	From The Above to The Exploration's Extent	Dense to very dense sand with pea gravel traces. Also contained are several 10 to 30 foot thick layers of: - medium to very stiff strength, somewhat slickensided, clay and - firm clayey/silty sand. The clay is more extensive beneath the replacement alignment's southwestern third.

detailed explanation). Major results will eventually be active channel capture by *One Eye Slough* plus enlargement of other *subchannels* now transiting the line's southwestern bank run, please refer to Figure 10. Occurrence rate will depend on:

- the frequency/extent of future flooding,

- the nature/rigor of flow management by mankind.

Of course the other *subchannels'* potential for development plus the manner in which the exposed pipe and the existing flow diversion fences are handled (restored/maintained, left to deteriorate inplace, partially/completely removed) will also affect any future alluviation's precise configuration and timing.

<u>River Activity.</u> Considering all these factors in context, specific lateral activity limits were projected as:

- **for the short term,** expansion - i.e. widening by 50 feet in either direction - of the active channel now crossed by TGPL's pipeline

- **for the mid term,** capture of the active channel by a southwestern bank *subchannel* - most probably *One Eye Slough* - with commensurate abandonment of the course presently transited by the facility

- **for the long term,** continued bend migration/development of the principal southwestern bank *subchannel* west of *One Eye Slough.*

Basically, during the next half century, the Neches River appeared capable of reconfiguring and/or discretely relocating its' active channel within a 3200 to 3600 foot wide zone extending from near its' present day course to a point several hundred feet beyond the TGPL right-of-way's juncture with the southwestern bank's electrical power transmission line.

Scour penetration vertically as deep as elevation - 29 feet, NGVD - i.e. to the Ancient Alluvium/Marine earth material interface - was judged possible anywhere the active channel is positioned. Although not likely, scour intrusion for a short distance into the Marine clay could occur during prolonged/extreme flooding.

The foregoing alluvial activity limits are displayed on Figures 10 and 11.

Geotechnical Analysis

Class 3 data inherent to forecasting HDD performance - i.e. predicting site-specific responses to the construction - are stated in this section.

Parameters. Specific geotechnical data stemming from analysis of the soil borings/laboratory testing results are presented on Table 7.

Site Integrity. Crucial aspects of HDD at this location were the effects of and on:

- site surface bearing capacity,
- site cleanup-restoration, plus
- groundwater quality preservation.

In essence, the site's active/passive effects on the drilled placement of the new pipe had to be thoroughly addressed.

Initially, installation of the replacement crossing - specifically, the necessary equipment access activities - were affected by the ground's somewhat low supportive capacity/spotty surface trafficability. In fact, HDD site accessibility posed the principal difficulty associated with the overall project.

In line with this, the replacement pipe's installation could not unduly disrupt the right-of-way surface. Scheduling construction during periods of low water - i.e. the "dry" summer months, spates of minimal releases from the *B.A. Steinhagen Lake* reservoir, etc. - and restricting/minimizing site clearing activities helped in such regard. Additionally, astute scheduling facilitated both construction access plus satisfied *U.S. National Park Service* requirements.

As to groundwater quality protection: since the near channel phreatic surface was directly tied into the Neches River, *surface water* quality governed *groundwater* quality throughout the zone of pipeline replacement interest. Consequentially, HDD implementation did not impinge this factor.

In terms of conducting HDD, all conditions presented on Tables 6 and 7 are highly amenable to such process. Consequently, design/employment of trenchless construction for this replacement crossing concentrated on avoiding the projected alluvial activity as well as minimizing the few site-specific adverse phenomena associated with the methodology.

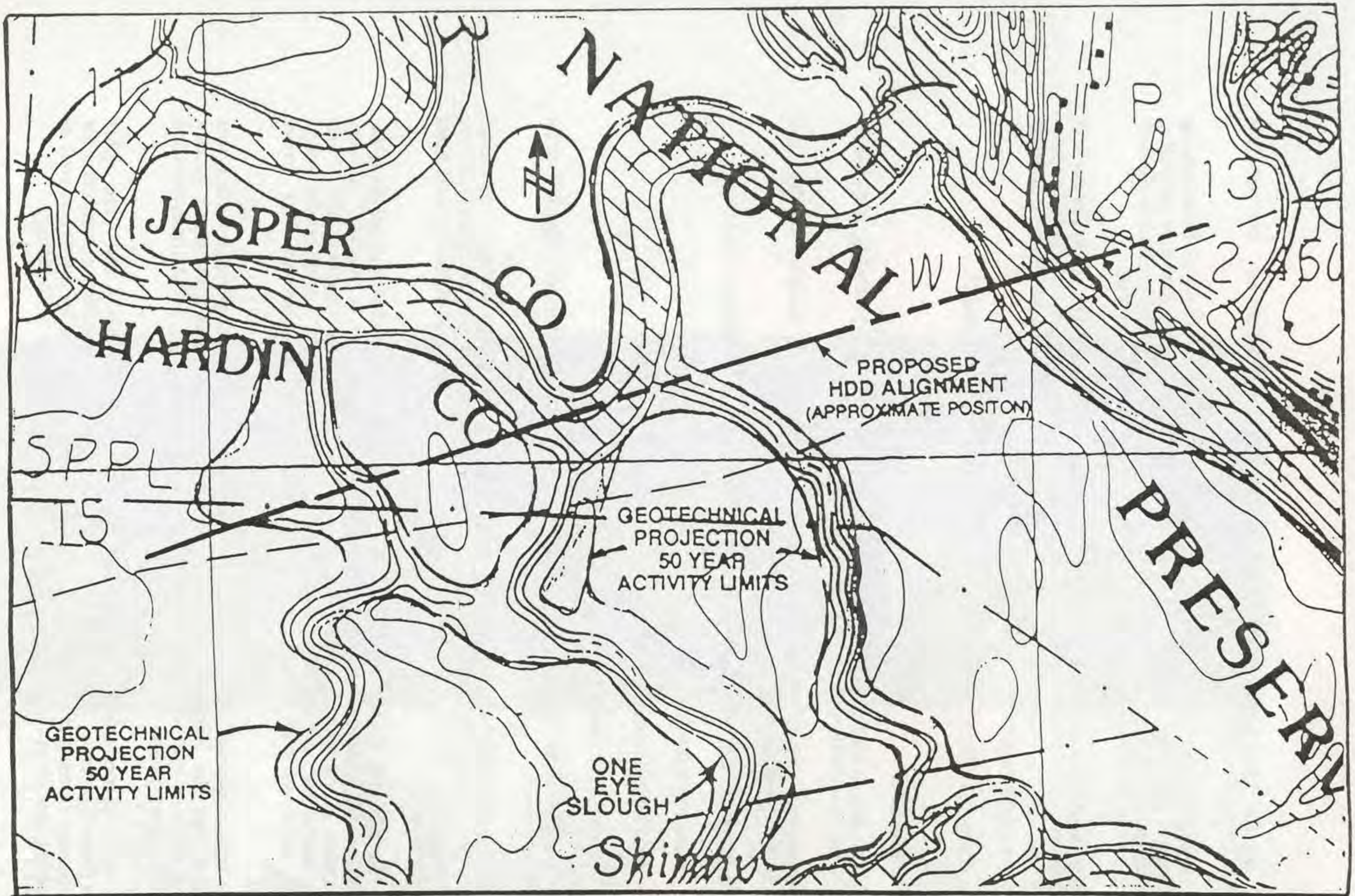


FIGURE 10
CLASS 3 DATA
SITE PLAN
HDD Alignment/Alluvial Activity Projections

HORIZ. SCALE: 200 400 600 800 1000

TABLE 7
CLASS 3 DATA
GEOTECHNICAL PARAMETERS

For the major earth material types identified at this site, design/installation of the crossing replacement were based on:

	Recent/Ancient Alluvium and Marine											
	Clay*							Sand*				
Parameters	Very Soft	Very Soft to Soft	Soft	Medium	Medium to Stiff**	Stiff**	Stiff to Very Stiff**	Loose	Firm***	Dense	Dense to Very Dense	Very Dense
Unit Weight												
- Total (pcf)	118	116	116	118	120	124	126	115	119	123	125	129
- Buoyant (pcf)	56	54	54	56	58	62	64					
Isotropic Permeability Coefficient k (cm/sec)	$5x10^{-9}$	$1x10^{-9}$	$1x10^{-9}$	$5x10^{-9}$	$1x10^{-8}$	$1x10^{-8}$	$1x10^{-8}$	$5x10^{-2}$	$1x10^{-3}$	$6x10^{-3}$	$8x10^{-4}$	$2x10^{-4}$
Strength												
-Cohesion, c (psf)	200	300	450	750	1000	1500	2500	--	--	--	--	--
-Friction Angle, ϕ (degrees)	--	--	--	--	--	--	--	25	29	32	34	36

* Classification is applicable to soil materials containing additional constituents; i.e. sandy clay, silty sand, sandy with gravel traces, etc.
** Values are for undisturbed materials in a confined state. Presence of clay slickensides - i.e. prefractures - significantly increases permeability and reduces unconfined strength by between 25 and 40 percent.
*** Permeability based largely on partical size distribution. Therefore, this lower density granular stratum may be less pervious than more compact formations.

Geotechnical Considerations

The following Class 3 data - stemming from the understanding of alluvial activity/subsurface stratification provided by the study - formed the basis of the HDD design/construction process.

Crossing Geometry. To help avoid future alluvial activity - including the eventual expansion of a relatively small southwestern bank subchannel presently paralleling the TGPL right-of-way's northwestern edge; HDD length was set at almost 4200 feet.

Penetration slopes of between 8 and 12 degrees from the horizontal plus 3500 foot curvature radii for the sagbends were applicable to in situ conditions.

Since the Marine sand and clay are highly suited to HDD, the site's lower - i.e. foundation stratum - was used for the bore's horizontal run. Placement to below elevation -105 feet, NGVD, helped achieve almost absolute security from "time-span-of-interest" alluvial disturbance plus virtually negated the chance of mud seep development at the site's surface - see below.

Recommended geometry is presented on Figure 11. In essence, this Class 3 data is the site investigation's **ultimate product**.

Installation Conduct

Scheduling resulted in construction during September 1994, i.e. the dry time of the year. In view of the site's topographical conditions and the northeastern bank's significantly better trafficability; the contractor - Laney Directional Drilling - opted for drilling equipment access/set-up there. Pilot hole drilling was therefore conducted from northeast to southwest. Carrier pipe makeup was thus on the comparatively long and unobstructed southwestern bank right-of-way.

Although drilling through the upper clay "spontaneously" created a slurry, some "manufactured" drilling mud was necessary for penetrating the sand strata: execution of the vertical exploration borings required normal amounts of bentonite drilling mud since the in situ sands are fairly compact and the penetrated clay "automatically" contributed to the downhole slurry. Furthermore, lubrication provided by the drilling mud in concert with the alluvial/marine sand's - and pea gravel constituent's - nominally rounded particle shapes prevented damage to the carrier pipe's protective coating. Finally, inadvertent drilling fluid returns along the drilled alignment's surface projection did not occur. This was due to the depth of the drilled placement plus conduct of the boring in pervious sand. The latter factor essentially precluded the down-hole drilling fluid from pressurizing.

Conclusion - Case Study

A 30 inch diameter pipeline replacement crossing of the Neches River at this site - trenchlessly installed for about 4200 feet, horizontally, and to almost 120 feet, vertically, below the bank surfaces - should not be disturbed by alluvial activity. In situ earth materials provided excellent conditions for performing the replacement construction via HDD in an economical/site-friendly fashion. Groundwork for successful placement of this large diameter pipe was laid by the site characterization study. Details resulting from such effort allowed the construction to be astutely engineered, efficiently permitted, and economically executed.

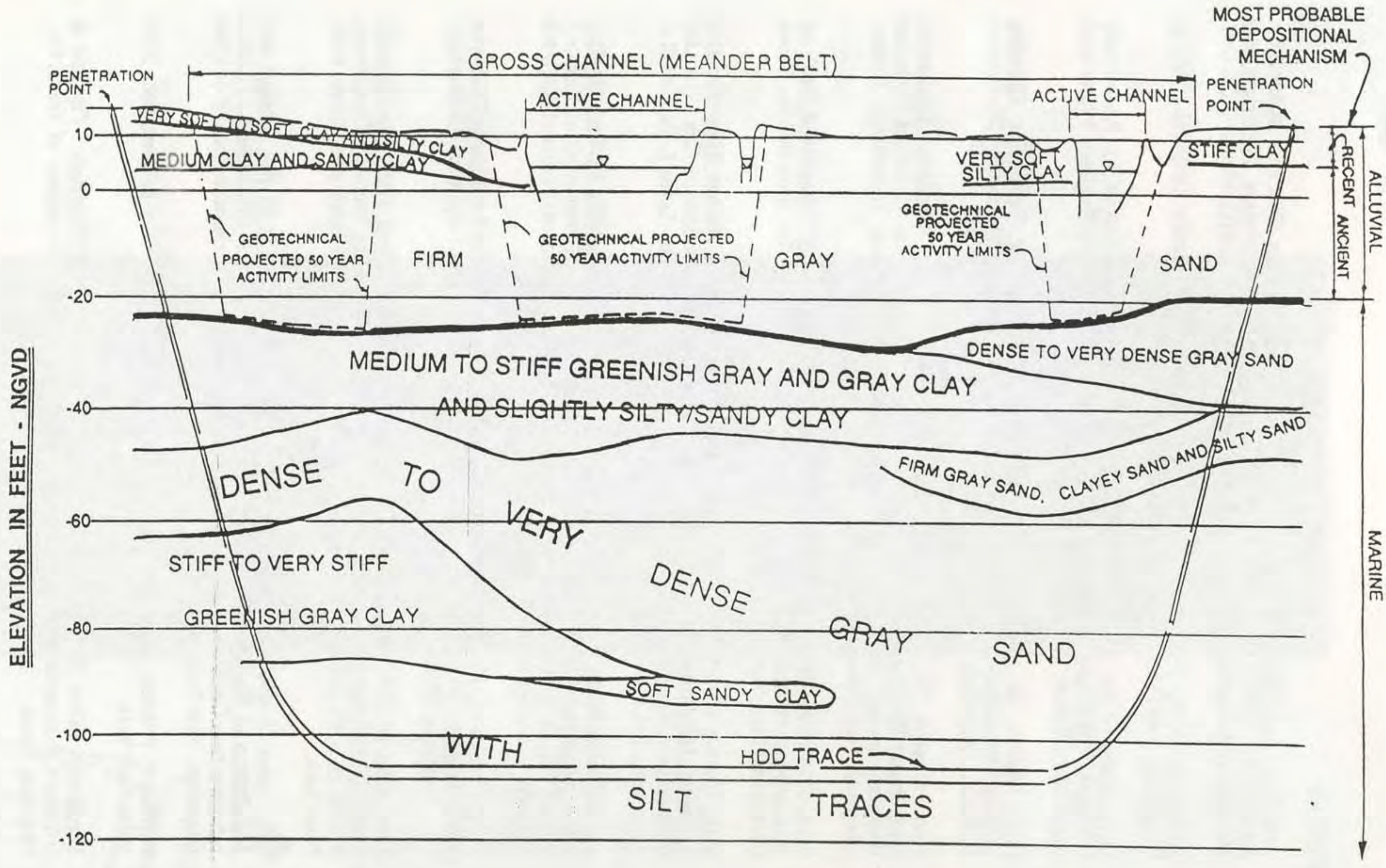


FIGURE 11
CLASS 3 DATA
SUBSURFACE PROFILE
HDD Geometry/Alluvial Activity Projections

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