SITE CHARACTERIZATION
FOR
HORIZONTAL DIRECTIONAL DRILLING

Prepared for:

THE DIRECTIONAL CROSSING CONTRACTORS ASSOCIATION
SECOND ANNUAL
SPRING SYMPOSIUM

15 through 17 April 1993
Brownsville, Texas

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1 April 1993

Not to be published without the prior written approval of the author and the Directional Crossing Contractors Association, Dallas, Texas
INTRODUCTION

As the number of successful horizontal directionally drilled (HDD) installations continues expanding worldwide, the process is increasingly viewed as the method of first choice for an ever wider array of crossing applications. However, even though HDD is becoming an engineerable, i.e. plannable, construction technique; its' sensitivity to site conditions still remains the major detriment to procedural employment. Consequentially, for process usage to increase beyond present day bounds, its engineerability 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 characterization must be founded on:

- definition of the obstacle to be crossed. Physical dimensions of - as well as time dependent changes in - the feature to be transited must be considered.
- knowledge of conditions which must be negotiated by the HDD process. Both passive factors - i.e. the site's constituency - as well as active factors - i.e. the various responses to the construction process - must be analyzed.

In essence, HDD's effects on the site and, in turn, the site's effects on the HDD construction/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 trenchless construction projects; this paper summarizes site conditions typically affecting HDD installations. With much of the discussion extracted from previous publications, especially Refs. 1 and 2; overall intent is to provide a framework for structuring future HDD site characterizations. A case history, based on a recently completed project, illustrates several of the suggested investigative procedure's nonstandard aspects.

SITE CONDITIONS

In characterizing a site for HDD, the two salient considerations are:

- in situ natural features, man-made facilities, and earth materials dictate the manner in which HDD construction is configured
- application of HDD construction elicits responses from the site's natural and artificial aspects during both the short and long terms.

Consequently, site conditions can be divided into two major groups - passive and active.

Passive Conditions. These are the site's constituency - i.e. its"makeup" - prior to construction. While the subsurface profile - i.e. the earth material types, stratification, anomalies/impediments to drilling, groundwater conditions, etc. - predominates the geophysical factors, also included in this category are the obstacle to be crossed plus the various aspects of the site's surface: topographic relief, presence of humankind's activities, etc. In essence, passive conditions are the site's in-place characteristics which must be negotiated by the HDD placement.

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.). Simply stated, active conditions at a given location are the site's responses - i.e. behavior when subjected to drilling - plus HDD's performance peculiarities.

PASSIVE CONDITIONS

While the primary thrust of this presentation is on Active Conditions, a summary of Passive Conditions is presented for completeness. Primary considerations here are:

- geological factors
- obstacle definition
- geotechnical aspects.

The following paragraphs describe each consideration.

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), lacustrine (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.

Obstacle Definition. In general, HDD is used to negotiate two types of obstacles: alluvial (river) and non-alluvial (highway and railroad embankments, flood protection levees, environmentally sensitive surface areas, contaminated subsurface zones, etc.). Primary concern in evaluating either type is determination of the feature's extent, both in space and in time. Potamology - the study of rivers - yields an alluvial obstacle's potential for horizontal displacement and vertical penetration; i.e. the stream's meandering and scouring characteristics during a selected time span, see Ref. 3. By the same token, non-alluvial obstacle effects with the passage of time - i.e. uncompleted consolidation settlement of a massive highway embankment, integrity maintenance of a flood protection levee, etc. - must also be evaluated. In concert with a site's geotechnically related passive conditions, a thorough definition of the obstacle to be crossed will dictate geometry of the directional bore plus the steps necessary to restore site integrity following HDD completion.

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, 4 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 - see Refs. 4, 5, 6, and 7 - present additional details and test methodologies. Subsurface stratification defines the manner in which the earth material parameters are distributed throughout the site. Here-to-fore, both such informational items - acting in concert - have provided the primary focus for HDD design and construction planning.

<table>
<thead>
<tr>
<th>Classifications</th>
<th>EM1110-2-1906</th>
<th>ASTM D-2116</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight</td>
<td></td>
<td>Unconfined Compression</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>ASTM D-2216</td>
<td>Unconsolidated, Undrained</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>ASTM D-4318</td>
<td>Triaxial Compression</td>
</tr>
<tr>
<td>Silt Analysis</td>
<td>ASTM D-422</td>
<td>Consolidated, Undrained</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Triaxial Compression</td>
</tr>
<tr>
<td>Deformations</td>
<td></td>
<td>Groundwater</td>
</tr>
<tr>
<td>Incremental Consolidation</td>
<td>ASTM D-2435</td>
<td>Falling or Constant Head</td>
</tr>
<tr>
<td>Constant Rate of Strain</td>
<td></td>
<td>Permeability</td>
</tr>
<tr>
<td>Consolidation</td>
<td>ASTM D-4186</td>
<td>Flexible Wall Permeameter</td>
</tr>
</tbody>
</table>

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

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>
<thead>
<tr>
<th>Rock Quality Designation (RQD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>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 1/2 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:</td>
</tr>
<tr>
<td>RQD (%)</td>
</tr>
<tr>
<td>90 - 100</td>
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<tr>
<td>75 - 90</td>
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<td>50 - 75</td>
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<td>25 - 50</td>
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<tr>
<td>0 - 25</td>
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</table>
TABLE 2
(Continued)

<table>
<thead>
<tr>
<th>Original Version</th>
<th>Revised And Expanded Version</th>
</tr>
</thead>
<tbody>
<tr>
<td>Talc</td>
<td>Talc</td>
</tr>
<tr>
<td>Gypsum</td>
<td>Fingernail</td>
</tr>
<tr>
<td>Calcite</td>
<td>Copper Penny</td>
</tr>
<tr>
<td>Flourite</td>
<td>Flourite</td>
</tr>
<tr>
<td>Apatite</td>
<td>Knife Blade</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>Glass</td>
</tr>
<tr>
<td>Quartz</td>
<td>Steel Flie</td>
</tr>
<tr>
<td>Topaz</td>
<td>Vitreous Pure Silica</td>
</tr>
<tr>
<td>Conundum</td>
<td>Quartz</td>
</tr>
<tr>
<td>Diamond</td>
<td>Topaz</td>
</tr>
<tr>
<td></td>
<td>Garnet</td>
</tr>
<tr>
<td></td>
<td>Fused Zirconium Oxide</td>
</tr>
<tr>
<td></td>
<td>Fused Alumina</td>
</tr>
<tr>
<td></td>
<td>Silicon Carbide</td>
</tr>
<tr>
<td></td>
<td>Boron Carbide</td>
</tr>
<tr>
<td></td>
<td>Diamond</td>
</tr>
</tbody>
</table>

**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, $\phi$ - 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 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 in service 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. Once 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 in-place 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 - see Ref. 8. 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.

With this distinction in mind, it is obvious that the site's passive conditions affect all phases of HDD. 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>
<thead>
<tr>
<th>Earth Material Type</th>
<th>Gravel Constituency Range, Percent by Weight</th>
<th>Directional Drilling Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft to hard strength, possibly slickensided (profractured via desiccation) clay.</td>
<td>N/A</td>
<td>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 &quot;skipping&quot; along the weak/strong interface. Pilot stem steering difficulties are likely to result during passage through very soft layers.</td>
</tr>
<tr>
<td>Very loose to very dense sand with or without gravel traces.</td>
<td>0 to 30</td>
<td>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.</td>
</tr>
<tr>
<td>Very loose to very dense gravelly sand.</td>
<td>30 to 50</td>
<td>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.</td>
</tr>
</tbody>
</table>
TABLE 3  
(Continued)

<table>
<thead>
<tr>
<th>Earth Material Type</th>
<th>Gravel Constituency Range, Percent by Weight</th>
<th>Directional Drilling Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose to very dense sandy gravel</td>
<td>50 to 85</td>
<td>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.</td>
</tr>
<tr>
<td>Very loose to very dense gravel</td>
<td>85 to 100</td>
<td>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.</td>
</tr>
<tr>
<td>Rock</td>
<td>N/A</td>
<td>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 &quot;skip&quot; along the lower hard surface. If in &quot;rounded&quot; cobble form, competent rock is virtually impossible to drill.</td>
</tr>
</tbody>
</table>

**Summary.** On balance, drillable passive conditions 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**

Astute project planning, innovative engineering design, and pragmatic construction execution - all based on a complete understanding of the site's passive subsurface conditions - are necessary for defining and negotiating active conditions. Crucial to successful execution are geometrical monitoring of the hole as it is installed plus continual evaluation of such factors as drilling mud pressurization downhole, carrier pipe buoyancy, and the construction's effects on site integrity. Overall objective is to minimize, or possibly eliminate, undesirable "responses" - both long term as well as short term - to HDD conduct.

**Crossing Design.** 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 slatted 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/carryer 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.

The construction'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/blowdown 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. 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. The Case Study Section 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 - see Ref. 9. Specifically:

- the pipe's inherent stiffness will resist its being pulled around the crossing sagbends
- 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 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. *Drilling Mud* 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, see Ref. 10. 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 berm/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.** 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.

**SITE CONDITIONS DETERMINATION**

In terms of characterizing a specific site, both types of conditions - passive as well as active - require engineering evaluation of field exploration and laboratory testing results.

**Passive 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 3, 4, and 5 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.
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.

**Passive 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.

**Passive/Active Engineering Evaluation.** Based on field exploration/laboratory testing results, plus data developed during actual HDD conduct; 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 project the followon active subsurface conditions. Engineering "products" are germane not only to designing the operation's conduct, but also necessary to obtaining construction permits and planning reduction/mitigation of HDD's few adverse effects.

Data for active engineering evaluation while HDD is in progress has traditionally stemmed from monitoring pull/torque forces plus geometrical readouts from positioning devices. In a few instances, data has been extracted from downhole geometrical surveys (subsequent to pilot hole completion) plus evaluation of drilling mud parameters. In a very few cases, post construction analysis - involving testing specimens of annulus plug grout, observing performance (water tightness) of grout seals, etc. - has been accomplished.

**CASE STUDY**

**Project Description.** Basically, the project entailed the horizontal directional drilling of dual - 8 and 24 inch nominal diameters - natural gas pipelines in a common borehole beneath the Tees River mouth on the United Kingdom's east-central coast. Crossing length approximated 820 meters at a maximum penetration beneath bank level of slightly more than 40 meters. The negotiated subsurface profile consisted of made-ground (dredge spoil, clinker slag, etc.) over
estuarine clay/silt and weakly lithified mudstone/siltstone. Extensive industrial activities - mainly chemical production facilities - were in place on both bank surfaces.

**Construction Sequence.** Begun in mid 1991, pipe placement first involved establishing a pilot hole to a design configuration between preselected points on the riverbanks. During this initial stage of construction, it was determined that the drill bit had horizontally "wandered" off course. Corrective actions were therefore necessary to insure the pilot bore exited the far shore's bank surface within the required target area.

After attainment of an acceptable pilot hole, borehole reaming - in successive stages to a 42 inch nominal diameter - was accomplished to allow pullin of the carrier pipes.

The initial attempt at pullin - involving the two empty, closed end pipes joined only at the pullhead - proceeded no farther than approximately 110 meters. At that point, cobble (clinker slag) "fallback" from the pipe-side apparently jammed the pullhead so that further advancement was impossible. As a result of this, the contractor withdrew the carrier pipes leaving only the washpipe string in the reamed bore. Following several days of deliberation, the contractor elected to suspend the pullin attempt until a pit could be excavated on the pipe-side to clear the source of fallback.

Shortly after completion of the pipe-side excavation, the bored hole was re-reamed - i.e. swept out - to 42 inches in diameters. A gyroscopic survey was then performed to quantitatively assess the borehole's "as drilled" geometry. Afterwards, the carrier pipes - in essentially the same configuration as for the initial pullin attempt - were successfully drawn into the hole to complete the project. During this "final" construction stage, high pull forces were encountered at the 110 meters distance in from the pipe-side as well as during the last 100 meters on the rig-side.

**Analytical Method.** Objective was to quantify possible reasons for the difficulties experienced during pullin. Study basis was:

- pilot bore geometry data derived from contractor measurements during initial drilling operations
- borehole reaming force information, i.e. torque and pullback measurements, supplied by the contractor
- carrier pipe pullin forces (torque and pullback) also obtained from the contractor
- borehole geometry details from the gyroscopic survey performed prior to carrier pipe pullin.

Analysis conduct entailed evaluating the information base using a two step process. First, the borehole's geometry - i.e. its path, both horizontally and vertically, between the riverbank penetration points - was determined. Second, borehole reaming/carrier pipe pullin performance specifics were compared to the bore's geometry in order to quantify reasons for the encountered difficulties. Results are the series of plots on figures 7 through 12.

**Borehole Geometry.** Figure 7 shows the borehole geometry measured during the gyroscopic survey. An overplot depicts curvature radii - computed for 2 meter long arc segments - for the surveyed borehole's horizontal and vertical configurations. Details of curvature radius computation are shown on the figure's upper portion. Since reaming stages/carrier pipe pullback operations were stated in terms of "wash pipe" units, the horizontal scale is so depicted. Wash pipe "numbering" was used in this - and all later figures - because the exact length of each individual section was unknown. Consequently, approximate horizontal distances can be determined by multiplying the "wash pipe" unit scale by 10 meters.
Significant aspects of figure 7 are:

- the vertical plot is relatively smooth and indicates borehole entrance/exit angles within the planned $81/2^\circ$ to $11^\circ$ range

- horizontal shape of the plot shows aberrations, i.e. kinks, at the locations where pullin problems were encountered on the rig-side as well as the pipe-side.

Some variations in hole shape are most likely due to precision of the measuring technique/equipment plus downhole location of the gyroscopic survey instrument (reportedly in the 5 inch diameter washpipe string "somewhere" within the 42 diameter reamed bore). However, despite such "errors", the gyroscopic survey does, in fact, accurately depict the reamed borehole's geometry.

Since the gyroscopic survey data is demonstratably viable, its component "dogleg" angular offsets for 2 meter arc segments were therefore used to compute incremental curvature radii for the overall bore. Results show that the smaller radii are concentrated in the crossing's vertical sagbends and horizontal curves, or "kinks". Of interest is that a reduced curvature radius is readily apparent at points roughly 100 to 110 meters in from the bore's pipe-side and rig-side surface penetrations.

Salient feature of the curvature radius parameter is that the borehole is "straight" where the incremental radii are large and "bent" where such values are small. To satisfy the standard rule of thumb for directional drilling design (see above), a curvature radius of roughly 730 meters is necessary. This is not met chiefly in the areas of the reamed bore's horizontal kinks.

**Reaming - Pullin Forces.** Figures 8 through 12 present plots comparing torques and pull forces (required during the various installation stages) with the borehole geometry computed from the gyroscopic survey data. The torque and pullback force ordinates (vertical scales) are unitless, i.e. in both cases only relative magnitudes, rather than absolute values, are stated. Also shown on these figures are schematics of the "downhole" equipment specific to each construction phase.

With these factors in mind, major points stemming from these figures are:

- Overall magnitudes of both the torques and the pull forces generally were lessened as the borehole was enlarged, i.e. as reaming progressed to each successive stage.

- During an individual reaming stage, the pull force was either constant or showed a general increase at the ends and middle of the borehole trace. Conversely, the torque force generally decreased as the downhole assembly progressed from the pipe-side to the rig-side.

- During carrier pipe pull-in, the magnitudes of both the pull and torque forces were greater than during the reaming stages. Also, the pull force parameter uniformly increased from start to finish while the torque remained at a relatively constant level.

- Spikes - i.e. short duration increases - in both the torque and pull force plots occurred at areas of reduced radii, especially at the kinks in the bore's horizontal trace.

- The number and frequency of such spikes, principally in the torque force plots, diminished as the bore was reamed.
Generally speaking, a torque force increase indicated that the reamer was cutting "new hole" while an elevated pull force showed that friction between the downhole assembly and the hole wall was increasing.

**Evaluation.** Considering the foregoing data in concert, the cause of carrier pipe pullin difficulty was the difference in downhole equipment configuration between the borehole reamings and the carrier pipe pullin attempts. Specifically, because the carrier pipes were pulled in empty, both positive buoyancy - in the case of the 24 inch diameter line - and negative buoyancy - for the 8 inch diameter pipe - were generated. Furthermore, by joining them only at the pullhead, the two carrier pipes essentially acted independently relative to their respective buoyancies. These factors had the effects of:

- generating an increasing longitudinal resistance as the carrier pipes were pulled into the borehole
- altering the pullhead reamer's intrahole alignment such that the cutting of new hole was necessary
- magnifying the effort necessary to negotiate any downhole anomalie (i.e. horizontal kinks, cobble fallback, etc.).

By contrast - because the reamer in each stage of hole enlargement and/or sweep out was followed only by the wash pipe string - a different, more consistent pattern of torque/pull force response was established during the preceding, multi-stage construction phase.

Computations indicate that when completely pulled into the slurry filled borehole, the 24 inch diameter pipe could generate nearly 150,000 pounds of positive buoyancy. Concurrently, the 8 inch diameter pipe could produce close to 50,000 pounds of negative buoyancy. Overall effect of these buoyancy forces was translated into normal force against the perimeter of the reamed bore. In turn, this produced longitudinal frictional resistance which increased as additional lengths of carrier pipe were drawn into the borehole. It has been our experience with other projects that - to counter these forces - larger diameter carrier pipes should be partly, or totally, filled with water prior to their being pulled into place.

That these buoyancy differences also produced independent carrier pipe "action" is confirmed by comparing relative positioning of the two unbound pipes: upon entering the ground, the smaller line was on the western side of the larger one while such positions reportedly were reversed when the pull head emerged from the river's opposite bank. This revealed that the 24 inch pipe sought the top of the reamed bore while the 8 inch line tended to lay on the hole's bottom. Furthermore, as the bore's horizontal kinks were negotiated, the larger diameter line was positioned toward the upper, inside of such curves while the smaller pipe gravitated to the lower, outside of the bends.

Likewise, the effects of carrier pipe buoyancy tended to misalign - i.e. cock - the pull head with respect to the borehole and thus necessitated the reamer's cutting new hole during pullback. Since the buoyancy condition of the followon washpipe string did not change during the borehole's reaming stages, pull head orientation downhole remained essentially constant. Therefore new hole was cut during reaming primarily when a larger size reamer was drawn into place or when borehole kinks were negotiated: compare torque performance during both stages of 42 inch diameter reaming. Consequently, the uniformly high torque forces during carrier pipe pullin can be attributed to the reamer's cutting new hole because of buoyancy induced misalignment, while the torque spikes in all cases were due mainly to borehole geometrical anomalities.

Regarding geometrical anomalities, hang up of the carrier pipes' initial pullin attempt 110 meters in from the pipe-side penetration point was probably precipitated by such a condition: in this instance, a "constricted" borehole likely due to a zone of increased hardness mudstone. A
reduced diameter borehole - in conjunction with a horizontal kink - is implied: the zone of curvature radius reduction there is relatively short in comparison to the length of other geometrical kinks elsewhere. Presence of a constriction also would have enhanced any cobble/clinker fallback's ability to jam the carrier pipe string. Both cases - possible zones of hardened earth material (which could have been compensated by "overreaming") plus the presence of cobbles/clinker slag near the pipe-side's surface - were forecasted by the site's preconstruction geotechnical investigation.

Finally, pulling the carrier pipes around kinks in the borehole generated bending stresses in the lines themselves, especially the large modulus 24 inch diameter one. Such bending stress further increased the pull force necessary to drag the carrier pipe assembly across the river. A complete understanding of the relationship between carrier pipe stiffness, shortened borehole curvature radius, and pull force development requires considerable additional research and is therefore outside this present study's scope.

Summary. Difficulties with the carrier pipe pullin phase of this river crossing stemmed mainly from active conditions: the downhole assembly's configuration and buoyancy differing fundamentally from those of the pullhead employed for the pre-reams. Borehole geometric anomalies - primarily horizontal kinks and a diameter constriction - amplified the configuration/buoyancy effects to generate an adverse pullback performance.

A final note on the data used for evaluating this project's active conditions: to our knowledge no such downhole survey of an installation borehole had previously been performed. Conduct of such procedure in this particular case - by quantifying the active condition difficulties associated with the final pullin - has revealed the efficacy of performing similar measurements on future HDD projects. Furthermore, the need for additional research into the meaning/nature of carrier pipe pullback forces is also indicated. In essence, a better understanding of active conditions - principally the pipe's interaction with the borehole's geometry - will facilitate forthcoming HDD installations.

CONCLUSION

A thorough understanding of in situ conditions, both active and passive, is mandatory for a HDD installation in today's climate of engineered construction. In essence, concurrent understandings of the site's constituency - i.e. the passive conditions - together with the various response phenomena stemming from the construction itself - i.e. the active conditions - are crucial to designing and executing any HDD project. Furthermore, continued advancements in site characterization technology and engineering procedures will allow collateral improvements in the overall construction process. Avenues now available for augmenting such understandings will lead to enhanced chances for construction success, lower costs, and greater application of horizontal directional drilling.
REFERENCES


7. Department of the Army, Office of the Chief of Engineers. **EM 1110-2-1906 Laboratory Soils Testing.** Periodically Updated. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.


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**FILE NO.: 93-0**
FIGURE 2
TYPICAL LABORATORY TESTING RESULTS
Granular Soil Particle Size Distribution
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.
### LOG OF BORING

- **PROJECT:** Any River Crossing
  - Anywhere Feasible
- **COMPANY:** Any Client
  - Any City
- **BORE NUMBERS:** 1
  - FILL: 93-0
  - DATE: 28 Jan 93
  - TECHNICIAN: US

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| 30        | Firm tan sand with gravel traces
  - Penetration Resistance: 22 blows per foot (5/9/13) |
| 35        | Dense tan sandy gravel
  - Penetration Resistance: 41 blows per foot (10/19/22) |
| 40        | Stiff light gray very silty clay |
| 45        | Stiff light gray silty clay
  - Rock encountered at 45\(\frac{1}{4}\) feet
  - Gray schist
  - 71% recovery (black diamond bit) |
| 50        | Cement-bentonite grout backfill top 25 feet |

**Surface Elevation 3.0 Feet, NGVD**

**BORING DEPTH:** 55 Feet

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**LOUIS J. CAPOZZOLI & ASSOCIATES, INC.**
Geotechnical Engineers

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**FIGURE 4**

**TYPICAL FIELD EXPLORATION**

Log of Onland Boring
**LOG OF BORING**

**PROJECT:** Any River Crossing  
**Anywhere Feasible**  
**FOR:** Any Client  
**Any City**  

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**Casing top 4 feet above water**  
**Approximate water depth 14 feet**

- **20**  
  - Mudline  
  - Very loose gray silty fine sand with 1 inch clay layers

- **25**  
  - Firm tan sand  
  - Penetration Resistance 18 blows per foot (6/8/10)

- **30**  
  - Medium gray slightly organic clay

- **35**  
  - Very dense tan sandy gravel
  - Penetration Resistance 50 blows per 8\(\frac{1}{4}\) inches (20/18/32 for 2\(\frac{1}{4}\) inches)

- **40**  
  - Very dense tan coarse sand  
  - Penetration Resistance 25 blows per 5 inches (10/25 for 5 inches)  
  - Clay encountered at 40\(\frac{1}{4}\) feet

- **45**  
  - Very stiff 11ight gray and white clay  
  - Very stiff 11ight gray and white clay (SLS)

- **50**  
  - Very stiff 11ight gray and white clay (SLS)

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**FIGURE 5**  
**TYPICAL FIELD EXPLORATION**  
Log of Inriver Boring
**LOG OF BORING**

**PROJECT**: Any River Crossing  
**Anywhere Feasible**  
**Any Client**  
**Any City**

**BORING**: 3  
**FILE**: 93-0  
**DATE**: 28 Jan 93  
**TECHNICIAN**: US

<table>
<thead>
<tr>
<th>FEET</th>
<th>SAMPLING</th>
<th>PENETRATION TEST</th>
<th>BORING DEPTH</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td><strong>6&quot; Diameter Sample Obtained</strong></td>
<td><strong>UNDISTURBED SAMPLE</strong></td>
<td><strong>70 Feet</strong></td>
<td>Cement-bentonite grout backfill top 25 feet</td>
</tr>
<tr>
<td>5</td>
<td>Very soft dark gray peat with clay traces</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 10   | Very loose gray clayey sand  
Penetration Resistance 2 blows per foot (2/1/1) |
| 15   | Very soft gray clay with 4 inch sand layer |
| 20   | Firm gray fine sand with clay traces |
| 25   | Loose tan gravelly sand  
Penetration Resistance 6 blows per foot (2/3/3) |
| 30   | Very dense tan gravelly sand  
Penetration Resistance 50 blows per 11 inches (7/17/33 for 5 inches) |
| 35   | Soft gray clay with gravel traces |
| 40   | Very dense tan sand  
Penetration Resistance 30 blows per 5½ inches (30 for 5½ inches) |
| 45   | Stiff gray clay  
Penetration Resistance 18 blows per foot (5/7/11) |
| 50   | Stiff gray clay with sand streaks |

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**FIGURE 8**  
**TYPICAL FIELD EXPLORATION**  
Log of Onland Boring.
GEOMETRICAL RELATIONSHIPS

FOR LARGE RADIUS CIRCLES:

Short Chord = Subsected Arc

THEREFORE: \[ \frac{AC}{Chord} = \frac{AC}{Arc} \]

PER SURVEY: \[ \frac{AC}{Arc} = 2 \text{ Meters} \]

\[ \therefore \Delta = \text{ Dogleg Angle For 2 Meter Arc} \]

Number of Arcs = \[ \frac{360^\circ}{\Delta} \]

Circumference = \[ \text{Number of Arcs} \times 2 \text{ Meters/Arc} = \frac{360^\circ}{\Delta} \times 2 \text{ Meters} \]

Radius = \[ \frac{\text{Circumference}}{2 \times \pi} = \frac{360^\circ}{\Delta} \times \frac{2 \text{ Meters}}{\pi} \]

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FIGURE 7
BOREHOLE GEOMETRY
Refer to Figure 3 for Details
FIGURE 8
17 INCH/26 INCH REAMING PERFORMANCE
Refer to Figure 3 and 7 for Details
FIGURE 9
32 INCH REAMING PERFORMANCE
Refer to Figure 3 and 7 for Details
FIGURE 10
FIRST 42 INCH REAMING PERFORMANCE
Refer to Figure 3 and 7 for Details
FIGURE 11
SECOND 42 INCH REAMING PERFORMANCE
Refer to Figure 3 and 7 for Details
FIGURE 12
CARRIER PIPE PULLIN PERFORMANCE
Refer to Figure 3 and 7 for Details