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An investigation of trenchless technologies and their interaction with native Iowa soils

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An investigation of trenchless technologies and their interaction with native Iowa soils

by

William M. Conway

A thesis submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Major: Civil Engineering (Geotechnical Engineering)

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ABSTRACT

Trenchless technologies are methods used for the construction and rehabilitation of underground utility pipes. These methods are growing increasingly popular due to their versatility and their potential to lower project costs. The use of trenchless technologies in Iowa and their effects on surrounding soil and nearby structures has not been adequately documented, however.

Surveys and interviews of professionals working in trenchless related industries in Iowa were carried out and the results are analyzed and compared to survey results from the United States as a whole. The surveys focused on method familiarity, pavement distress observed, reliability of trenchless methods, and future improvements. Results indicate that the frequency of pavement distress or other trenchless related problems is an ongoing problem in the industry. Inadequate soils information and QC/QA are partially to blame.

Field work involving the observation of trenchless construction projects was undertaken with the purpose of documenting current practices and applications of trenchless technology in the United States and Iowa. Field testing was performed in which push-in pressure cells were used to measure the soil stresses induced by trenchless construction methods. A program of laboratory soil testing was carried out in conjunction with the field testing.
1 INTRODUCTION

1.1 Problem Statement

Trenchless Technologies are a group of methods for construction and rehabilitation of underground utilities without using open-cut excavation. The use of trenchless technologies is increasing due to the growing need to replace aging utility infrastructure and the need for more flexible solutions for the installation of new pipes. Many applications exist for trenchless construction methods which take advantage of the methods’ ability to dive underneath obstacles. An example is the ability to install utility pipes across busy streets without disrupting traffic.

Limited technical data currently exists on the relationship between the cutting mechanism, conduit materials and dimensions, and their effects on soil properties and performance of subgrade soil and pavement systems. A better understanding of these relationships would allow improvements to be made to the design and practice of these construction methods which could result in the improved performance of overlying pavement and nearby underground structures.

1.2 Objectives

This research was undertaken with two primary aims:

- To document the current practices and applications of trenchless technology in the United States and particularly in Iowa.
- To evaluate the effects of trenchless construction on surrounding soil and adjacent structures.
The project was intended to provide information on the different trenchless methods and to document current construction and QC/QA practices used across Iowa and United States. A testing program was implemented to address these objectives.

1.3 Methodology

This thesis is divided according to the research tasks that were conducted. A literature review was first performed to assemble information on the current practice of trenchless technologies. The literature review examines the rationale for trenchless technology, and introduces the major trenchless construction and rehabilitation methods. Soil investigation methods for trenchless projects, quality control/quality assurance, the effects of trenchless technologies on surrounding soil, and design processes are all discussed. A program of surveying and interviewing trenchless practitioners was undertaken to gain additional insights into experiences in the field, focusing mainly on practices in Iowa. A field investigation was performed which involved observing trenchless construction projects, documenting procedure successes and failures, interviewing personnel, recovering soil samples for laboratory testing, and the measurement of soil stress changes during construction. Laboratory testing was carried out to better understand the interactions between the trenchless construction processes and the soil. Lastly, the results are analyzed and discussed.
2 LITERATURE REVIEW

2.1 Introduction

Trenchless technologies can be defined as a group of methods for construction and rehabilitation of underground utilities that require minimal surface excavation and provide important new alternatives to traditional open-cut methods of utility pipe installation. These techniques offer many unique advantages. Trenchless methods are becoming increasingly important as the number of utility pipes for water, gas, telecommunications, and storm and sanitary sewers multiply beneath roads.

Open-cut methods of utility pipe installation involve excavating a trench along the proposed path of the pipeline and placing the pipe in the trench. These methods are variously called open-cut, open-trench, utility cut, dig-and-install, dig-and-repair, or dig-and-replace. Such methods can involve additional construction complications, such as road detours, traffic control, trench excavation and shoring, dewatering, backfilling and compaction operations, bypass pumping systems, and reinstatement of the surface. This can cause construction effort to be focused on peripheral tasks, rather than the pipe installation itself.

Open-cut methods can have negative effects that may sometimes be avoided by using trenchless technology. Frequently, utility pipe projects are located beneath pavement which must be removed to perform open-cut work. The natural gas industry estimates that almost 60% of their pipes run below pavement (Najafi 2005). Peters (2002) observes that premature distress that is often seen in newly paved utility cuts may include cracks that allow water to enter and soften the base course and cause loss of pavement support which can result in the further deterioration of the pavement. Arudi et al. (2000) adds that such problems have a direct influence on the pavement integrity, life, aesthetic value, and drivers’ safety. Bodocsi et al. (1995) quantifies this by noting that new pavement should last between 15 and 20 years, while pavement over utility cuts exhibits a shortened life span of about 8 years.
Open-cut installations can also carry significant economic disadvantages. The American Public Works Association (APWA, 1997), reported that a study conducted in Burlington, Vermont found that the weakening of pavement caused by utility cuts required an additional asphalt overlay thickness of 0.75 to 1.5 inches of pavement to compensate. The additional cost was $522,000 per year. Additionally, Los Angeles, California reported spending $16.4 million per year on pavement overlays to strengthen pavements damaged by utility cuts (APWA, 1997). In addition to pavement overlays, a report by Najafi (2005) states that up to 70% of the total cost of underground utility projects can be attributed to backfilling, compaction, and the replacement of landscaping and pavement.

Because of the limitations of open-cut methods, the development of trenchless technologies has been encouraged. Trenchless construction methods allow pipe to be installed deeper, avoiding areas of underground pipe congestion. Trenchless methods also have the potential to save both time and money, and offer lower social costs when compared with open-cut methods. However, the difficulty of quantifying the value of social costs can cause decision makers to under-appreciate the value of trenchless methods (Gangavarapu et al., 2003). As technology and expertise continue to improve for this still maturing industry, it is expected that trenchless technologies will be utilized for increasing numbers of underground utility projects (Najafi, 2005).

Additional advantages of trenchless methods in comparison to open cut methods are listed by several researchers (see Stidger, 2002, Barsoom, 1995, Khogali and Mohamed, 1999, and Yung and Sinha, 2007), as summarized below.

- Reduction in required surface restoration
- Reduction of damage to adjoining utilities
- Decrease in disturbance to local residents and businesses
- Increase of flexibility in alignment selection
- Increase of flexibility in choosing depth of new installation which may allow more favorable soil conditions to be used
- Less relationship exists between cost and depth of installation
- Reduction in number of utility relocations
- Reduction in the amount of spoil that requires disposal
- Reduction in the need for dewatering
- Reduction in access requirements, which is advantageous in urban settings and under rivers, etc.
- Improvement in safety for the public and for job site workers
- Ease of renewal of existing pipelines
- Mitigation of air, water, and noise pollution
- Reduction of the disturbance to traffic flow
- Ability to install pipe in frozen ground during cold weather
- Possibility of increased speed of work

The designation, “trenchless technologies”, can be somewhat misleading. Many “trenchless” methods exist, and all share the common characteristic of minimal, but often some, surface disruption. Trenchless technologies are commonly divided into two categories: trenchless construction methods and trenchless rehabilitation methods.

Stidger (2002) states the belief that a lack of understanding of methods and costs seems to be the key reason that open-cut methods are still commonly chosen for some projects. Considering that the focus of utility pipe work is shifting toward established urban areas with aging utility pipes (Thompson, 1993), trenchless construction and rehabilitation methods should be considered by designers. The direct costs associated with construction and reinstatement and the indirect costs associated with disruption must be understood for the various alternatives. As comparisons between open-cut and trenchless methods can be very complicated in practice, designers may rely upon indirect cost estimating equations, such as those developed by Tighe et al. (1999) to quantify the costs associated with traffic delays. These savings can run into the tens or even hundreds of thousands of dollars, depending on
average annual daily traffic (AADT) and expected construction time. McKim (1997), Russell et al. (1999), and Clark and Browning (1992), provide additional methods to economically compare alternatives for given project parameters, an area which is outside the scope of the research.

Although trenchless methods have many advantages over open-cut methods, uncertainties about some trenchless methods can cause concern. Iseley and Gokhale (1997) observe that some trenchless methods carry the risk of subsidence, surface heave, and leaking of drilling fluid. Additionally, the often higher risk inherent in a trenchless project compared to an open-cut project can make a failure considerably more expensive (O’Reilly and Stovin, 1996).

This literature review intends to document the current trenchless construction methods (TCM) and trenchless rehabilitation methods (TRM) with an additional focus on the effects of trenchless construction on adjacent structures, including buried pipes and pavements. Additionally, the literature review will summarize existing soil investigation and QC/QA methods.

2.2 Trenchless Construction Methods (TCM)

Trenchless construction methods encompass a family of methods that are used to install new underground utility pipe without requiring an open cut. While most of these methods do involve a limited amount of surface disruption, they require much less in comparison to open-cut methods.

Myers et al. (1999) estimate that 150,000 miles of new conduit is installed each year in North America. The distribution of new conduit construction by end use (Figure 2.1) shows that gas and telecommunication industries require the most.
Numerous methods of trenchless construction exist and they are still evolving as experience and technology improves. Jeung and Sinha (2007) reported that the most commonly used methods are listed below.

- Microtunneling
- Conventional tunneling
- Compaction methods (impact moling)
- Horizontal directional drilling (HDD)
- Pipe jacking
- Pipe ramming
- Auger boring
- Water jetting
- Pilot-tube microtunneling

Table 2.1 shows important characteristics of the most common trenchless construction methods.
<table>
<thead>
<tr>
<th>Method</th>
<th>Diameter range (in)</th>
<th>Maximum installation (ft)</th>
<th>Pipe material</th>
<th>Applications</th>
<th>Accuracy of Installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe jacking and conventional tunneling</td>
<td>≥42</td>
<td>1500</td>
<td>RCP*, GRP*, steel</td>
<td>Pressure and gravity pipe</td>
<td>± 1 in</td>
</tr>
<tr>
<td>Auger boring</td>
<td>4-60</td>
<td>600</td>
<td>Steel</td>
<td>Road and rail crossing</td>
<td>±1% of the bore length</td>
</tr>
<tr>
<td>Microtunneling</td>
<td>10-136</td>
<td>500-1500</td>
<td>PE, steel, PVC*, clay, FRP*</td>
<td>Gravity pipe</td>
<td>± 1 in</td>
</tr>
<tr>
<td>Mini-HDD</td>
<td>2-12</td>
<td>600</td>
<td>PE, steel, ductile iron</td>
<td>Pressure pipe/cable</td>
<td>Varies</td>
</tr>
<tr>
<td>Midi-HDD</td>
<td>12-24</td>
<td>1000</td>
<td>PE, steel, ductile iron</td>
<td>Pressure pipe</td>
<td>Varies</td>
</tr>
<tr>
<td>Maxi-HDD</td>
<td>24-48</td>
<td>6000</td>
<td>PE, steel</td>
<td>Pressure pipe</td>
<td>Varies</td>
</tr>
<tr>
<td>Pipe ramming</td>
<td>&lt;120</td>
<td>400</td>
<td>Steel</td>
<td>Road and rail crossing</td>
<td>Dependent on setup</td>
</tr>
<tr>
<td>Compaction methods</td>
<td>&lt;8</td>
<td>250</td>
<td>Any</td>
<td>Pipe or cable</td>
<td>±1% of the bore length</td>
</tr>
<tr>
<td>Pilot tube microtunneling</td>
<td>6-10</td>
<td>300</td>
<td>RCP, GRP, VCP, DIP, Steel, PCP</td>
<td>Small diameter gravity pipes</td>
<td>± 1 in</td>
</tr>
</tbody>
</table>

*Abbreviations:
RCP = Reinforced concrete pipe
GRP = Glass fiber reinforced polyester
VCP = Vitrified clay pipe
DIP = Ductile iron pipe
PCP = Polymer concrete pipe
PVC = Poly-vinyl-chloride
FRP = Fiberglass reinforced polyester

As pipe diameter, length of bore, pipe material, and type of utility affects the construction method chosen, so also do the ground conditions. The location of the bore relative to the water table, and also the type of soil both have significant impacts on the effectiveness of the various methods (Kenny et al., 2003). Table 2.2 summarizes how ground conditions influence the suitability of various trenchless construction methods and this table provides a
general guideline on the suitability of common trenchless techniques in different soil types. Table 2 indicates that loose sand, dense sand below the water table, soil with cobbles, and significantly weathered rocks provide the most significant challenges for most trenchless construction techniques. Medium to very stiff clays and silts, and medium to dense sands above the water table are the only soils that are suitable for all trenchless construction methods.

**Table 2.2. Ground conditions and suitability of trenchless road crossing methods (Iseley et al., 1999)**

<table>
<thead>
<tr>
<th>Ground Conditions</th>
<th>Guided Boring</th>
<th>Auger Boring</th>
<th>Pipe Ramming</th>
<th>Horizontal Drilling</th>
<th>Pipe Jacking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft to very soft clays, silts &amp; organic deposits</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Medium to very stiff clays and silts</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Hard clays and highly weathered shales</td>
<td>Y</td>
<td>M</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Very loose to loose sands above water table</td>
<td>M</td>
<td>Y</td>
<td>M</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Medium to dense sands below the water table</td>
<td>N</td>
<td>N</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
</tr>
<tr>
<td>Medium to dense sands above the water table</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Gravels and cobbles less than 50-100 mm diameter</td>
<td>Y</td>
<td>Y</td>
<td>M</td>
<td>M</td>
<td>Y</td>
</tr>
<tr>
<td>Soils with significant cobbles, boulders, and obstructions larger than 100-150 mm diameter</td>
<td>M</td>
<td>Y</td>
<td>M</td>
<td>N</td>
<td>M</td>
</tr>
<tr>
<td>Weathered rocks, marls, chalks, and firmly cemented soils</td>
<td>Y</td>
<td>M</td>
<td>Y</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Slightly weathered to unweathered rocks</td>
<td>Y</td>
<td>M</td>
<td>M</td>
<td>M</td>
<td>N</td>
</tr>
<tr>
<td>Yes</td>
<td>Generally suitable by experienced contractor with suitable equipment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marginal</td>
<td>Difficulties may occur, some modifications of equipment or procedure may be required</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>Substantial problems, generally unsuitable or unintended for these conditions.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Horizontal Auger Boring and Guided Auger Boring**

ASCE (2004) defines horizontal auger boring as, “a technique for forming a bore from a drive pit to a reception pit by a rotating cutting head. Spoil is removed back to the driveshaft by helically wound auger flights rotating in a steel casing”. Horizontal auger boring, also known as ‘jacking and boring’, is one of the oldest trenchless methods, and is also one of the most cost effective according to the North American Society for Trenchless Technology (NASTT 2006). Auger boring is known to be the most widely used trenchless method for installing steel pipes and casings (ASCE, 2004).

Horizontal auger boring can be used to install pipes ranging from 4 inches to 72 inches in diameter for a length averaging about 175 feet to 225 feet, with a maximum distance of 600 feet (ASCE, 2004). There is no limit to the potential depth of installation. Horizontal auger boring is suitable for a variety of soil conditions, but experiences the most difficulty in sands below the water table (Munro and McMurdie, 1985).

Often for highway or railroad crossings, a casing will be installed using trenchless methods. Then the product pipe (the actual utility pipe) will be installed in the casing. This prevents leaks in the product pipe from causing damage to the roadbed.

The auger is a flighted tube which transfers spoil back to the machine and has couplings at each end that transmit torque to the cutting head from the power source located in the bore pit. The auger string is advanced by the jacking action of the horizontal auger boring machine in the launching pit moving forward on a track. A thrust block is located at the back of the launching pit to transmit the thrusting force from the track to the back of the launching pit. After the machine has pushed the segment of auger and casing completely into the ground, it is disconnected and the machine moves to the back of the launching pit and a new segment of auger and pipe are connected (see Figure 2.2). A profile view of a typical auger boring setup is shown in Figure 2.3. The casing supports the soil around it as the spoil is removed. Spoil is deposited out of the auger into the launching pit, where it can then be
removed with a basket attached to an excavator. A product pipe is inserted into the casing upon completion of the bore and the annular space is grouted. (Kenny et al., 2003)

![Auger boring machine](image)

**Figure 2.2.** Launching pit with auger boring machine installing pipe

![Typical auger boring system](image)

**Figure 2.3.** Typical auger boring system (Iseley and Gokhale, 1997)

The machinery may possess varying degrees of steering capability. The most basic types of auger boring operations have limited line and grade control, and rely heavily on the initial setup of the track to launch the bore in the proper direction. This requires the construction of
a stable thrust block and foundation for the track. Generally, three small-diameter metal pipes are attached to the top of the casing as it is augered into the soil (see Figure 2.4). One small pipe is used for a basic waterline system to measure grade. The second pipe is a fluid supply line that could be used in case it becomes necessary to deliver grout or water to the cutting face. Water could be needed if sticky clay was encountered, and grout could be necessary to fill voids outside of the casing caused by the excessive inflow of water and sand into the pipe. The third small pipe is a steering rod that allows the cutting head to pivot up and down. The steering rod is the most common method used to steer during the installation (ASCE, 2004), although it allows little horizontal maneuvering. Some horizontal steering can be achieved by attaching a piece of metal called a “wing” to the side of the casing to induce the pipe to veer in the desired direction as it is jacked.

![Figure 2.4. Waterline grade measurement pipe, steering rod, and fluid supply pipe attached to the top of the casing](image)

The guided auger boring method, also known as the “pilot tube method” and “guided thrust boring”, is defined by ASCE (2004) as, “the term applied to auger boring systems that are similar to microtunneling, but have the guidance mechanism actuator sited in the driveshaft.
This pipe installation method uses pilot tubes and a theodolite to install small-diameter pipes with high accuracy.”

A launching pit as small as 8 feet in width is first constructed and the hydraulic pipe jacking machine is installed at the appropriate depth. A pilot bore is then drilled. This is done by jacking a 1 to 4 inch thick cutting head into the soil along the centerline of the bore. The asymmetric cutting head is spun while jacking to bore in a straight line and displace and compact the soil laterally into the borehole walls (Boschert 2007). Additional sections of drill rod are added in the launching pit as the bore progresses. When the theodolite position monitoring system detects that the pilot bore has shifted off course, steering corrections are made by jacking the asymmetric cutting head at a stationary and specific angle, inducing the bore to veer in the desired direction. This pilot boring process closely resembles that used during HDD.

Once the cutting head has emerged in the retrieval pit, a larger diameter auger boring system is connected to the last section of drill rod and jacked into the soil. Attaching the auger boring cutting head to the pilot bore drill string ensures that the path followed by the pilot bore will also be followed by the auger bore. As with conventional auger boring, no drilling fluid is necessary. Additional sections of casing and auger are connected in the launching pit as the bore progresses. The auger bore enlarges the hole to a diameter slightly larger than that of the pipe to be installed. Once the cutting head emerges in the retrieval pit, a section of product pipe are one by one connected to the casing and jacked into place while the auger and casing sections are simultaneously removed at the retrieval pit (Figure 2.5).
Guided auger boring is not recommended for use in soils with boulders, because the pilot tubes could be deflected. Also, guided boring is not recommended in sands below the water table because of the possibility of settlement due to water flowing out of the soil through the pipe (Fisher, 2003).

More information on horizontal auger boring and guided auger boring is available in ASCE’s Horizontal Auger Boring Projects manual of practice (ASCE, 2004).

Tunneling and Microtunneling
Man-entry tunnel boring machines (TBMs) and remotely-operated microtunnel boring machines (MTBMs) are two methods that may be used for line and grade critical pipelines greater than 42 inches in diameter. These two methods share many common features and equipment, but differ in that MTBMs tend to deliver better accuracy and performance than TBMs, however with an increased cost. Both methods have relative strengths and weaknesses and it is generally thought that the choice between MTBMs and TBMs should be made based on site specific subsoil conditions, as will be discussed.
TBMs can be classified as either stationary shield excavator machines or rotating cutter head wheel machines. A stationary shield TBM requires personnel at the excavation face to remove the soil with hand tools while it is steadily advanced into the borehole by hydraulic jack positioned behind in the launching pit. A rotating cutter head TBM (see Figure 2.6) excavates the soil using cutter heads rather than hand tools. The TBM’s functions are controlled by an operator seated inside the machine. These functions include cutter head rotation, jacking rate, steering, and spoil removal. A laser is generally used to monitor line and grade. Steering is accomplished by adjusting the angle of the cutter head. A variety of cutter heads are available, such as fully open, sand shelves, open face, and closed face cutter heads. Spoil is removed by either a conveyor or a cart on which the cuttings are deposited at the cutting face before they are transported out of the machine for disposal (Mathy and Kahl, 2003). The most common diameters for TBMs are 48 inches to 72 inches, although TBMs can be as large as 12 feet in diameter. A cross section of a TBM is shown in Figure 2.7.

Figure 2.6. Rotating cutter head TBM
MTBMs are mostly used for the installation of gravity pipelines such as for a sanitary or storm sewer. The American Society of Civil Engineers’ Standard Construction Guidelines for Microtunneling (2001) defines the procedure as “a remotely-controlled, guided pipe jacking technique that provides continuous support to the excavation face and does not require personnel entry into the tunnel.” Drilling slurry is typically used to transport spoil. MTBMs (see Figure 2.8) are available with inside diameters ranging from 10 inches to 136 inches or more and have virtually no depth limitation. The cross section of a microtunneling procedure is shown in Figure 2.9.
The most critical risk when conducting either type of tunneling is surface settlement. The two main ways to prevent settlement from occurring are maintaining the stability of the excavation face and avoiding inadvertent loss of soil into the tunnel. To attain these goals, the primary geotechnical concern when performing utility tunneling is accurately describing and predicting soil behavior at the face of the tunnel (Mathy and Kahl 2003). Particularly, the presence of boulders and saturated sandy soils can be problematic.
The uncontrolled inflow of groundwater into the tunnel can lead to tunnel flooding and erosion of the tunnel face, and the loss of flowing soil can cause the formation of voids resulting in surface settlement. The prevention of inflow by de-watering along the tunnel alignment is not always possible in urban environments; however microtunneling provides an effective solution when the MTBM is equipped with a slurry spoil removal system. The computer of the MTBM continuously monitors slurry pressures in the borehole to offset the external hydrostatic groundwater pressure, making microtunneling very effective in areas of groundwater and flowing, highly permeable soils. Conversely, the open-face of TBMs can make them unsuitable for flowing soil and water conditions. TBMs can be fitted with a closed-faced shield that allows groundwater inflow to be controlled in certain soils, such as low permeability clays, but even in these most favorable conditions, TBMs are restricted to working in less than 10 feet of unbalanced groundwater head. In highly permeable granular soils, TBMs may be unable to control any groundwater inflows and would require dewatering or ground treatment for the bore to proceed. (Mathy and Kahl, 2003)

When tunneling with a shallow depth of soil cover, TBMs have the advantage of not using pressurized slurry at the boring face, thereby avoiding the risk of hydrofracture of the soil. TBMs also possess an advantage over MTBMs in certain soil conditions because of the operator’s ability to access the face of the bore, which is not possible in microtunneling. This ability can be useful when encountering cobbles and boulders. Personnel can access the tunnel face where cobbles and boulders can be removed or broken down with hand tools and removed in pieces.

Mathy and Kahl (2003) assert that MTBM pipe jacking is currently being specified for some projects for which conventional open or closed face tunnel boring machines (TBMs) may be a better option. This is because of a perception that microtunneling is a more effective method than conventional tunneling regardless of the soil situation. In practice, the designer should consider the unique characteristics of a project and match it to the relative advantages
of both methods before making a selection. Table 2.3 compares relative differences between TBM and MTBM technology.

**Table 2.3. Relative comparison of TBM and MTBM (from Mathy and Kahl, 2003)**

<table>
<thead>
<tr>
<th>Factors</th>
<th>TBM</th>
<th>MTBM</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. No. of personnel to operate system</td>
<td>x</td>
<td>1.5x</td>
</tr>
<tr>
<td>2. Total power requirements</td>
<td>x</td>
<td>1.5x to 2x</td>
</tr>
<tr>
<td>3. Noise</td>
<td>x</td>
<td>1.5x</td>
</tr>
<tr>
<td>4. Top side equipment space</td>
<td>x</td>
<td>1.5x to 2x</td>
</tr>
<tr>
<td>5. Spoil volume</td>
<td>x</td>
<td>1x to 1.5x</td>
</tr>
<tr>
<td>6. Safety</td>
<td>x</td>
<td>0.5x</td>
</tr>
<tr>
<td>7. Obstructions</td>
<td>x</td>
<td>&gt; 2x</td>
</tr>
<tr>
<td>8. Cutter face torque</td>
<td>x</td>
<td>1x to 1.5x</td>
</tr>
<tr>
<td>9. Production rate:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- firm/hard ground</td>
<td>x</td>
<td>0.5x</td>
</tr>
<tr>
<td>- soft ground</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>- flowing ground</td>
<td>x</td>
<td>2x</td>
</tr>
<tr>
<td>10. Relative cost of tunneling</td>
<td>x</td>
<td>1.5x</td>
</tr>
</tbody>
</table>

**Horizontal Directional Drilling (HDD)**

Horizontal directional drilling is a trenchless construction method utilizing a drilling rig to install pipelines beneath obstructions such as roadways, driveways, historical areas, landscaped areas, rivers, and streams. (Willoughby, 2005). In the private sector, media and communication firms are using HDD to install telephone, fiber optic, and cable conduits and lines. The public sector utilizes HDD for the repair and replacement of potable water mains, reclaimed water mains, storm water piping, sewage gravity piping, and force mains. Advantages of HDD include the ability to expedite projects and minimize impact on critical habitats while requiring smaller project footprints. HDD rigs can install pipes from 2 inches to 48 inches in diameter, and for distances up to 6,000 feet, depending on the size of the drill rig. HDD is effective in a variety of ground conditions, but installation is generally faster in clay soils than in sands. HDD is not effective for soils with a significant number of cobbles and boulders, however, because they can deflect the bore and potentially damage the pipe as
it is pulled into place. Nearly 4,000 HDD rigs operate daily across North America (Baumert and Allouche, 2003).

The drill rig used for HDD can be described as essentially a traditional drill rig for vertical drilling that is turned on its side (Neu, 2004). Complete HDD systems usually included the drill rig, a trailer for transport, a power supply that is separate from the drill rig, a drilling fluid mixing and control system, water and drilling fluid tanks, a variety of drill bits, additional drill rods, and the necessary accessories, including the electronic sending and locating system (Treadway, 1997).

The presence of an existing underground pipe or wires presents a significant hazard to HDD operations. The drill machine operator may avoid these obstacles, however, if their exact location is known (Najafi, 2005). The drill rig is then set up at one end of the planned bore (see Figure 2.10), while a receiving pit can be excavated at the other end to retrieve the equipment at the proper depth.

Figure 2.10. Directional drilling machine
A small diameter pilot bore usually between 1 and 5 inches in diameter is then drilled into the soil from the boring machine to the receiving pit (see Figure 2.11). The bore begins at the ground surface and proceeds downward at an angle of 8 to 15 degrees until the target depth is reached (Treadway, 1997). The drill bit is advanced by pushing and spinning the drill rod using the hydraulic machinery of the drill rig. A sonde (transmitter) attached to the drill bit allows a handheld locator at the surface to monitor the position of the drill bit in the ground. When course correction is not required, the drill rod is spun; this spins the attached bit and cuts the soil. The soil in the path of the drill hole is partially removed to the launching area and partially displaced and compacted into the sides of the borehole. Knowledge of how much of the soil is displaced and how much is compacted into the sides of the borehole was not available. When a course correction is required, the bit is rotated to a specific angle measured by clock position. Then, the bit is pushed into the soil without rotation. The slant-head bit is shaped in such a way that it will deflect the drill rod in the desired direction (see Figure 2.12). Additional sections of drill rod are added as the bore progresses. These drill rods are made of a special alloy steel and are designed to handle the stresses caused by the sag bends and directional changes. The rods are hollow, permitting drilling fluid to be pumped through them to the bit. (Woodroffe and Ariaratnam, 2008)

Figure 2.11. The pilot bore is begins by pushing the drill bit into the soil
The drilling fluid serves both to lubricate the borehole and drilling machinery, and to stabilize the borehole walls. The fluid must also cool the sonde (electronic transmitter) located behind the drill bit. Sometimes, plain water is used for bores of 50 feet or less and under certain geologic conditions (Najafi, 2005). More often, clay polymers (bentonite with additives) or biodegradable chemical polymers that increase the fluid viscosity are added to the water to provide lubrication and improve the stability of the borehole walls. The appropriate drilling fluid mixture is determined by the properties of the soil at the site and the pH and calcium content of the local water.

Once the initial pilot hole has been completed and the drill bit has emerged into the exit pit, it will be replaced with a reamer (also called a “back reamer”) (see Figure 2.13) that will be rotated and pulled back toward the drilling machine. This process will both enlarge the hole and smooth any sharp bends that may have occurred while drilling the pilot hole. The back reaming may be completed in one or several passes with reamers of progressively larger diameters. The product pipe will be attached to the reamer before the final pull back to complete the installation of the pipe (see Figure 2.14). This is usually the operation that causes the highest pullback stresses, due to the friction between the product pipe and the wall of the borehole. (Najafi, 2005)
Project specifications require a weak-link device to be attached between the reamer and product pipe during installation. Newer equipment is also available that will measure the pullback forces and report the results to the operator (see Figure 2.15). Allowable tensile load for setting weak link devices is determined by using ASTM pipe standards.
A slant head drill bit, such as the one visible in Figure 2.11, is used for drilling operations in soft soils. A mud motor drill head fitted with a roller cone drill bit is used for consolidated or tightly compacted formations. Various reamer types exist, such as the barrel, blade, delta, fluted, fly cutter, and spiral reamer. Each reamer type is designed to be appropriate for particular soil conditions. A barrel reamer is used in soft conditions as it assists in creating stable borehole walls. A blade reamer, such as the one shown in Figure 2.13, is used in normal sands and clays and come in sizes up to 26 inches in diameter. A delta reamer is a type of blade reamer that has been optimized for harder soil conditions such as stiff clays. A fly cutter reamer is used for still harder soil conditions, such as sandstone and siltstone. A fluted reamer is suitable for most ground conditions, although it has a risk of “balling up” in clay formation if an improper drilling fluid is used. A spiral reamer is used for loose conditions and for stony soil. Each type of drill bit and reamer contains small nozzles through which a continuous flow of drilling fluid is used during boring to wash unconsolidated material away, providing a pathway for the drill string.

Baumert et al (2002) suggests that current design models fail to account for installations where a significant portion of the borehole is comprised of solid drill cuttings that are not
entrained in the drilling mud. In this situation, annular mud flow is not maintained. Instead of considering this possibility, the design models currently in use to predict pull loads for large, expensive installations are based on assumptions of ideal borehole conditions. Specifically, this assumes a clean, stable borehole filled with low-viscosity drilling mud.

Pipe Jacking

The term “pipe jacking” may be used to describe either a specific trenchless construction method or a process that is used as part of other trenchless methods. When used to describe a specific trenchless method, pipe jacking refers to installation using hydraulic jacks located in the launching pit to push the pipe forward while workers inside the pipe perform the excavation and removal process, using manual or mechanical means (see Figure 2.16). When the term is used to describe a process in a separate trenchless method, pipe jacking describes an operation using a hydraulic jacking system to advance the pipe and cutting mechanism. Auger boring, tunneling, microtunneling, and pipe ramming are examples of separate trenchless methods that use a jacking mechanism to advance the pipe and cutter head.

![Figure 2.16. Pipe jacking with manual soil excavation (WK Construction)](image)

Pipe jacking is used to install pipe that is greater than 42 inches in diameter and for lengths up to 1500 feet. It is suitable for many clay and sandy soils, however the open boring face
makes the method inappropriate for installations beneath the water table, particularly in sandy soils. Additionally, pipe jacking is inappropriate in slightly weathered or unweathered rock. (Najafi, 2005)

Pipe Ramming

Pipe ramming is a trenchless construction procedure which involves pneumatically hammering a steel pipe into the soil formation (Figure 2.17). The leading edge of the pipe can either be closed with a cone tip, or open. The cone shaped end can be used for pipes up to 8 inches in diameter (Najafi, et al. 2003). This limitation exists because the soil is entirely displaced radially during installation, resulting in significantly increased soil pressures on all sides of the pipe and increased risk of surface heave.

![Pipe ramming diagram](Earth Tool Company LLC)

Figure 2.17. Pipe ramming diagram (Earth Tool Company LLC)

Open faced pipe ramming is usually used for pipes with diameters up to 55 inches and lengths up to 150 feet long. Installation should be made at a depth of at least 10 times the diameter of the product pipe. Pipe ramming is most commonly used for shallow installations under roads and railroads. The method can allow cost savings over auger boring and HDD for short bores of under about 60 feet due to the faster setup times and faster installation times. Pipe ramming can be used in nearly all soil types except solid rock. However, it can be unsuitable at depths below the water table, especially in sands, as groundwater can flow through the pipe and enter the insertion pit. A drilling fluid is used similar to that used for HDD installations and is delivered to the cutting face through a small pipe located above the
steel casing pipe. Additional detail about pipe ramming is available in Simicevic and Sterling (2001).

Compaction Methods (Impact Moling)

Impact moling is a trenchless construction method which uses a pneumatic mole to bore a small diameter hole. Impact moling is used to install pipes of up to 10 inch diameter for a length of up to 200 feet. Installation should be made at a depth of at least 10 times the diameter of the product pipe or 3-4 feet, whichever is greater. This precaution is meant to prevent surface heave. The method is most frequently used to install small diameter pipes for gas, water, and cable lines (Simicevic and Sterling, 2001).

No soil is removed during impact moling. Instead the mole compresses the soil in front of the device, resulting in lateral soil deformation as the bore is advanced (see Figure 2.18). This makes impact moling suitable for compressible soils. Similarly, impact moling can be inappropriate in stiff soils which resist deformation. Additionally, loose sands and gravels can be unsuitable for impact moling because of the potential for the borehole to collapse, and rocky soils can cause the mole to deflect from its course. (Clarke, 2004)

Figure 2.18. An impact mole
The procedure for impact moling starts by digging entry and exit pits for launching and retrieval. The next step is to position the mole in the bottom of the entry pit. The mole is laid in a starting cradle and operated to slowly ease itself into the ground while a telescopic aiming frame is used to monitor line and grade (see Figure 2.19). Line and grade are continuously monitored and adjusted until the mole has fully entered the soil. Steering is impossible, so the initial placement is critical. Drilling fluid is not used for impact moling. The mole hydraulically rams itself into the soil and will proceed through the soil to the exit pit without any possibility for further adjustment. The mole can be equipped with a transmitter for monitoring position. If the mole has been deflected from its course, it can be backed out of the borehole and the bore can be reset. When the bore has been completed successfully and the mole has reached the exit pit, the mole is detached from the hydraulic hose which is then used as a string with which to pull the pipe into place.

Impact moling is the most widely used trenchless installation method. Recently, moles equipped with steering systems capable of curved trajectories and direction changes have become available, but have not yet achieved widespread use (Peng et al., 2003). Detailed information on impact moling is available in Simicevic and Sterling (2001).
Pilot Tube Microtunneling

Pilot Tube Microtunneling (PTMT) was first introduced in the United States in 1995 and has been growing increasingly popular as an alternative to microtunneling. PTMT is used for the installation of small diameter pipes which require high accuracy. This method can be considered a hybrid of three existing trenchless boring methods. A pilot bore head with a slanted face is used, similar to HDD (see Figure 2.20). The guidance system is identical to that used in conventional microtunneling, and the auger spoil removal system is similar to that used in auger boring (Boschert, 2007).

![Steering Head](image)

**Figure 2.20. PTMT steering heads (Purdue University)**

Pilot tube microtunneling has been growing in popularity due to low equipment costs, a small surface footprint, accuracy, and small launching pits. Diameters of up to 32 inches can be accommodated, and maximum drive lengths are currently about 400 feet (Boschert, 2007). The maximum lengths and diameters are increasing, however, as the guidance system is gradually improved with better optics, and more thrust is available from more powerful hydraulic jacking systems. PTMT is most effective in soft soil conditions, and it is not considered suitable for soil with significant cobbles and boulders because these can impact steering. PTMT can be used above or below the water table (Najafi, 2005).
A typical pilot tube microtunneling project begins with the excavation of circular jacking and receiving pits which usually measure 6.5 feet to 8 feet in diameter. The jacking frame is then assembled in the launching pit. The PTMT machine is next set up at the correct line and grade using control points established by a conventional surface survey. The boring begins by pushing the pilot tube into the soil at the correct line and grade. The slant head drill bit (also called a steering head) is spun and pushed, and it displaces and compacts soil radially into the formation. The hollow stems of the drill rods provide a clear line of sight for a camera in the launching pit to view an LED target in the steering head and measure the line and grade. Once the steering head has reached the receiving pit on the correct line and grade, the camera guidance system can be removed, as the pilot bore has established the centerline. The next step is to attach the reamer behind the final length of pilot tube in the launching pit (see Figure 2.21). The reamer is slightly larger in diameter than the intended pipe. A casing of the same size as the reamer is connected behind the reamer, with an auger inside the casing to transport the cuttings back to the launching pit. The reamer is jacked into the soil, and the pilot tubing gradually emerges into the receiving pit. Finally, the auger finishes removing all of the spoil in the casing. The product pipe is then attached behind the auger casings in the launching pit, and the pipe is inserted in the borehole. As the pipe is jacked into place, lengths of the auger casing emerge in the receiving pit. Finally, the product pipe completely replaces the auger casing in the borehole and the installation is complete. (Boschert, 2007) and (Force et al., 2003)
2.3 Trenchless Renewal Methods (TRM)

Trenchless renewal methods provide a way to extend the design life of current pipe. These methods can be used to replace, rehabilitate, upgrade, or renovate an existing pipeline system. The basic trenchless renewal methods can be categorized into the following types (Najafi, 2005):

1. Cured-in-place pipe (CIPP)
2. Underground coatings and linings (UCL)
3. Sliplining (SL)
4. Modified Sliplining (MSL)
5. In-line replacement (ILR)
6. Close-fit pipe (CFP)
7. Localized repair (LOR) or point source repair (PSR)
8. Thermoformed pipe (ThP)
9. Lateral renewal (LR)
10. Sewer manhole renewal (SMR)
The selection of these methods depends on the physical conditions of the existing pipeline system. The important factors include pipeline length, type, material, size, type and number of manholes, service connections, bends, and the nature of the problem or problems involved. The problems with an existing pipeline could be structural or non-structural, and could involve infiltration or inflow, exfiltration or outflow, pipe breakage, joint settlement, joint or pipe misalignment, capacity, corrosion, and abrasion problems. (Najafi, 2005)

When considering a trenchless renewal project, factors that should be considered include constructability, cost factors, availability of service providers, life expectancy of new pipe, and future use of the pipe. Table 2.4 summarizes the common trenchless rehabilitation methods.
Table 2.4. Trenchless rehabilitation methods (Najafi, 2005)

<table>
<thead>
<tr>
<th>Method</th>
<th>Diameter range (in)</th>
<th>Maximum installation (ft)</th>
<th>Liner material</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inverted in place (CIPP)</td>
<td>4-108</td>
<td>3000</td>
<td>Thermoset resin/fabric composite</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Winched in place (CIPP)</td>
<td>4-100</td>
<td>1500</td>
<td>Thermoset resin/fabric composite</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Underground coatings and linings (UCL)</td>
<td>3-180</td>
<td>1000</td>
<td>Epoxy, polyester, silicone, vinyl ester, polyurethane, and cementitious materials</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Segmental (SL)</td>
<td>24-160</td>
<td>1000</td>
<td>PE*, PP*, PVC*, GRP* (-EP and -UP)</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Continuous (SL)</td>
<td>4-63</td>
<td>1000</td>
<td>PE, PP, PVC, PE/EPDM*</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Panel lining (SL)</td>
<td>&gt;48</td>
<td>Varies</td>
<td>GRP</td>
<td>Gravity pipelines</td>
</tr>
<tr>
<td>Spiral wound (SL)</td>
<td>6-108</td>
<td>1000</td>
<td>PE, PP, PVC, PVDM</td>
<td>Gravity pipelines</td>
</tr>
<tr>
<td>Formed-in-place (SL)</td>
<td>8-144</td>
<td>Varies</td>
<td>PVC, HDPE*</td>
<td>Gravity pipelines</td>
</tr>
<tr>
<td>Pipe bursting (ILR)</td>
<td>4-48</td>
<td>1500</td>
<td>PE, PP, PVC, GRP</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Pipe removal (ILR)</td>
<td>≤36</td>
<td>300</td>
<td>PE, PP, PVC, GRP</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Pipe insertion method (ILR)</td>
<td>≤24</td>
<td>500</td>
<td>Clay, ductile iron</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Close-fit-pipe structural (CFP)</td>
<td>3-24</td>
<td>1000</td>
<td>HDPE, MDPE*</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Close-fit-pipe nonstructural (CFP)</td>
<td>3-63</td>
<td>1000</td>
<td>HDPE, MDPE</td>
<td>Gravity and pressure pipelines</td>
</tr>
</tbody>
</table>
Table 2.4. (Continued)

<table>
<thead>
<tr>
<th>Method</th>
<th>Diameter range (in)</th>
<th>Maximum installation (ft)</th>
<th>Liner material</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Robotics (PSR)</td>
<td>8-30</td>
<td>NA</td>
<td>Epoxy resins, cement mortar</td>
<td>Gravity</td>
</tr>
<tr>
<td>Grouting (PSR)</td>
<td>NA</td>
<td>NA</td>
<td>Chemical gel grouts, cement based grouts</td>
<td>Any</td>
</tr>
<tr>
<td>Internal seal (PSR)</td>
<td>4-24</td>
<td>NA</td>
<td>Special sleeves</td>
<td>Any</td>
</tr>
<tr>
<td>Point CIPP (PSR/CIPP)</td>
<td>4-48</td>
<td>50</td>
<td>Fiberglass, polyester, etc.</td>
<td>Gravity</td>
</tr>
<tr>
<td>Thermoformed pipe (ThP)</td>
<td>4-30</td>
<td>1500</td>
<td>HDPE, PVC</td>
<td>Gravity and pressure pipelines</td>
</tr>
<tr>
<td>Lateral renewal</td>
<td>4-8</td>
<td>100</td>
<td>Any</td>
<td>Gravity pipelines</td>
</tr>
<tr>
<td>Coatings and linings - cementitious (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>Cementitious</td>
<td></td>
</tr>
<tr>
<td>Coatings and linings - polymers (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>Epoxy, urethane</td>
<td></td>
</tr>
<tr>
<td>Thermoplastic liners (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>PVC</td>
<td></td>
</tr>
<tr>
<td>CIPP (SMR/CIPP)</td>
<td>NA</td>
<td>NA</td>
<td>Resin saturated polyester felt or fiberglass</td>
<td>Sewer manholes</td>
</tr>
<tr>
<td>Pressure grouting (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>Cementitious</td>
<td></td>
</tr>
<tr>
<td>Chemical grouting (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>Polymers</td>
<td></td>
</tr>
<tr>
<td>Inserts (SMR)</td>
<td>NA</td>
<td>NA</td>
<td>Fiberglass</td>
<td></td>
</tr>
</tbody>
</table>

Note that method abbreviations are defined above the table.

*Material abbreviations:
- PE = Polyethylene pipe
- PP = Polypropylene
- PVC = Poly-vinyl-chloride
- GRP = Glass fiber reinforced polyester
- EPDM = Ethylene propylene diene monomer
- HDPE = High density polyethylene
- MDPE = Medium density polyethylene
Cured-in-Place Pipe (CIPP)

The cured-in-place pipe renewal procedure involves the insertion of a resin-impregnated fabric tube into an existing damaged pipe through the action of water inversion (see Figure 2.22), air inversion, or winching (see Figure 2.23). The fabric used in these pipes is polyester felt or fiberglass reinforced material. The inversion process is used to insert the liner, and hot water or steam is then used to effect the curing of the pipe (Bonanotte and Kampbell, 2004).

Cured-in-place pipe can be used for structural or non-structural purposes, and can be made strong enough to act as the sole structural support of the pipeline. The pliable properties of
the liner allow it to easily be positioned around curves and bends in the existing pipe. Additionally, felt-impregnated polyester resin or corrosion resistant fiberglass provides effective corrosion protection. (Barbero and Rangarajan, 2005)

With CIPP, it is also important that the design wall thickness is specifically the thickness of the structural wall layer. A CIPP’s wall is composed of two to three layers depending on its method of installation. For the direct inversion method of installation, there are two components: an inner plastic film layer (either polyurethane or polyethylene) and the outer structural wall layer (which is thermoset resin with an encapsulating polyester felt form). For pull-in-place installations and projects requiring a pre-liner, there is the added outer layer of another plastic film. (Kampbell and Whittle, 2003)

Underground Coatings and Linings (UCL)
Coatings and linings can be used to repair and renew existing damaged water and sewer infrastructure and to protect and increase the service life of new underground infrastructure. The lining is sprayed on for the purpose of improving the pipe’s hydraulic characteristics and corrosion resistance. The materials used for coatings and linings fall into four general categories, including cementitious, polymers, sheet liners, and cured-in-place liners.

Cementitious coatings are used mainly to protect against corrosion in water and sewer applications. Cementitious coatings are used commonly, as they are considered to be cost effective. Various base materials are used for the cement, but the most common are Portland cement and calcium aluminate. The cement coats the underground structure preventing infiltration by reinforcing deteriorated structures and has the additional benefit of creating a relatively smooth internal surface that improves hydraulic conductivity. The alkalinity of the cement inhibits corrosion in metal pipes.

Polymers are commonly used for underground applications due to their ability to be formulated for thick structure enhancing properties while curing and bonding on concrete, brick, steel, and cast iron in damp underground environments and protecting against
aggressive chemical surroundings. Polymers possess superior chemical resistance compared with cementitious products, which can be important in more severely corrosive environments. Polymers possess the flexibility to be formulated for both structural and non-structural use. Polymer linings are also often used to coat cement mortar linings in chemically severe environments.

PVC or polyethylene sheet liners have been in common use for the past 50 years. Today, cured-in-place liners have become successful and cost-effective alternatives to plastic sheet liners for use in existing underground infrastructure, but plastic liners are now used for new construction where they are used in combination with poured-in-place concrete. Najafi (2005) states that recently systems have been developed in which these sheet liners are applied to urethane mastic coatings to create a composite polymer coating – sheet lining system.

*Sliplining (SL)*

Sliplining is a technique wherein a new pipe of smaller diameter than the existing pipeline is inserted to structurally renew the pipeline. This method requires that the existing pipeline has no joint settlements or misalignments. The primary weakness of this method is that it may result in a significant loss of flow capacity because the new pipe has a smaller diameter than the original (Law et al., 2000). This limits the applicability of this method to pipelines with excess flow capacity. However, some of this flow loss is recovered because the smooth inside surface of the new pipe generally improves flow characteristics. Najafi (2005) states that the method is also relatively inexpensive, which makes sliplining an effective measure when viable.

*Modified Sliplining (MSL)*

Modified sliplining is a term encompassing methods involving pipe sections or plastic strips being installed to fit closely with the existing damaged pipe. Three types of MSL exist: panel lining, spiral wound, and formed-in-place.
Panel linings are usually used to structurally renew large diameter pipes. This method requires worker-entry, so the existing pipe must be greater than 48 inches in diameter. Fiberglass is the most commonly used material for panel lining.

Spiral wound pipe is generally used only for gravity sewers. This involves using a layered composite PVC liner and a cementitious grout to renew the damaged pipe. This method produces a strong bond between the PVC liner and the existing pipe.

Formed-in-place pipe is often used for renewal of wastewater, stormwater, and culverts for diameters between 8 inches and 12 feet. This method is appropriate for all normal pipe shapes and materials. Two or more thin sheets of HDPE are installed against the walls of the pipe, and the annular area between the sheets is grouted, forming a new pipe. (Najafi, 2005)

In-Line Replacement (ILR)
In-line replacement is an option for pipeline renewal if the current capacity of the pipeline is inadequate. Water, wastewater, and gas pipelines can be replaced using these methods. Nearly all types of pipe, including concrete, clay, steel, ductile iron, cast iron, asbestos, PVC, and PE can be replaced. In-line replacement techniques can be sub-divided into pipe bursting methods, in which the existing pipe is destroyed and expanded outward leaving a cavity for a new pipe, and pipe removal, which also referred to as “pipe eating”.

Pipe bursting is usually carried out in 300 to 400 foot increments, corresponding roughly to the distance between manholes. Pipes up to 48 inches have been burst, and improvements to the method are making it possible to burst larger diameters. Pipe bursting is not appropriate for use in expansive soils, near other underground structures or pipes with point repairs that have used a ductile material as reinforcement, and for pipes with collapsed sections. It is advised that the bursting head should not pass within 2.5 feet of other buried pipe, and it should not pass within 8 feet of sensitive surface structures (Simicevic and Sterling, 2001).
Research by Atalah (2006), suggests a distance of 2 feet from the bursting head to other pipe or structures rather than 2.5 feet may be adequate.

Pipe bursting uses a conical bursting head to fracture the walls of the existing pipe and expand the fragments outward into the surrounding soil. As the bursting head is advancing through the old pipe, a new pipe, usually PE, is being pulled into the new cavity behind it (see Figure 2.24). Three different types of pipe bursting exist: pneumatic, hydraulic expansion, and static pull. Of these three, pneumatic is the most frequently used (Najafi, 2005). In pneumatic pipe bursting, the bursting head is driven by compressed air to hammer into the existing, brittle pipe. In hydraulic expansion pipe bursting, the conical bursting head with a smaller radius is inserted a distance into the existing pipe with a winch. The bursting head is then hydraulically expanded outward, increasing its diameter and fracturing the pipe. In static pipe bursting, the force of the drilling head against the existing pipe is created by simply pulling the bursting head with a winch.

![Figure 2.24. Pipe bursting (Dayton & Knight Ltd.)](image)

Pipe splitting is a variation of pipe bursting that is used for ductile pipes, such as steel and ductile iron. Instead of using a bursting head that fractures the pipe radially, pipe splitting uses a splitter, which cuts the existing pipe along one line on the bottom side, slicing it open.
rather than fracturing it (see Figure 2.25). The pipe is then expanded outward as the tapered splitter is pulled through (Chapman et al., 2005).

![Image](image.jpg)

**Figure 2.25. Pipe splitting (RJM Company)**

Pipe eating, the other branch of in-line replacement, uses an MTMB to crush the pipe and remove its fragments. The new pipe is installed by jacking it in behind the MTBM. The MTBM is launched from a launching pit and is remotely controlled with a laser guidance system.

Detailed information on pipe bursting is available in Simicevic and Sterling (2001).

*Close-Fit Pipe (CFP)*

This method reduces the size of the new pipe before it is installed inside the damaged pipe (see Figure 2.26). After installation, the new pipe is expanded back to its original size and shape to provide a close-fit with the original pipe. The new pipe can be designed to serve either structural or nonstructural purposes. (Barber et al., 2005)
Localized Repair (LOR)

Localized repair, also referred to as point source repair (PSR), is a method used to repair local defects in a structurally sound pipe. Remote-controlled systems are used to inject resin into defects ranging from 4 to 24 inches in diameter (Bauhan et al., 1997). Grouting is used to solve our basic problems are addressed through localized repair. The first problem is connecting fragmentary pieces of unreinforced pipe. The second problem is providing additional localized structural capacity. The third problem is sealing cracks in the pipe to prevent infiltration and exfiltration. The fourth problem is replacing missing pipe sections (Najafi, 2005).

Thermoformed Pipe (ThP)

Thermoformed pipe has been used extensively in the United States since 1988, with over 21 million feet installed by 2003 (Najafi, 2005). Thermoformed pipe can be used for sewer systems, water mains, and gas lines. Structural and non-structural uses are possible. Advantages of thermoformed pipe include its ability to negotiate bends in the pipeline while causing only a very brief service disruption.
Thermoforming expands a PVC or PE pipe by the addition of heat, in order to fit tightly inside the existing pipe. Three methods of thermoforming exist, as listed below:

1. Fold and formed (F&F)
2. Deformed and reformed (D&R)
3. Fused and expanded (F&E)

The fold and formed method involves flattening PVC pipe in the factory during production and folded over before insertion. Once on the jobsite, the folded PVC pipe is heated with steam to make the pipe flexible, which allows it to be pulled through the existing pipe from one manhole to the next using a winch. Once in place inside the existing pipe, the PVC pipe is expanded using steam and air pressure until it is forced tightly against the existing pipe. This method can be used for gravity or pressure pipelines and can be designed to provide full structural integrity for the existing damaged pipe.

Deformed and reformed thermoformed pipe includes HDPE pipe deformed into a U shape during manufacturing. The deformed HDPE pipe is pulled through the existing damaged pipe from one manhole to the next using a winch. Once it is in place, the pipe is heated with steam to revert to its original round shape, and it is pressurized to expand out against the damaged pipe.

Fused and expanded thermoforming uses PVC pipe that has been fused together prior to installation. Fused and expanded pipe is often used in water mains with high pressures exceeding 150 psi. The PVC pipe is inserted through access pits, and once it is in the desired position, the PVC pipe is heated with a hot liquid and highly pressurized to fit tightly against the existing pipe. (Najafi, 2005)

Lateral Renewal (LR)
The majority of all wastewater pipe leaks occur from service laterals (Kiest Jr. and Flanery, 2003). Lateral renewal can be used to repair and renew sanitary sewer service laterals using
the same methods used for main pipelines. These include cured-in-place pipe, close-fit pipe, pipe bursting, chemical grouting, and spray-on lining. These methods can be used to repair damaged areas as large as 4 to 8 inches in diameter, up to a maximum of 100 feet in length. (Najafi, 2005)

*Sewer Manhole Renewal (SMR)*

Sewer manhole renewal methods provide a means to repair damage resulting in surface water inflow and ground water infiltration, fix structural damage, and protect manhole surfaces from corrosion. SMR can be divided in several methods. These include cementitious coatings, cast-in-place, cured-in-place, and profile PVC. Chemical grout can also be injected to stop inflow into manholes with no structural damage. (Najafi, 2005)

2.4 Soil Investigation Methods

The complexity and limited access to the soil / boring tool interface make trenchless construction methods significantly more sensitive to adverse ground conditions than traditional open-cut methods (Allouche et al., 2001). Temple and Stukhart (1987) cite unexpected subsurface conditions as the leading source of project delays, disputes, claims, and cost overruns for underground construction projects. For this reason, a successful trenchless construction project requires thorough knowledge of the subsurface conditions (Allouche et al., 2001). Trenchless projects require the contractor to possess sufficient subsurface information to select appropriate construction methods and to prepare for likely obstacles.

The quality and quantity of geotechnical information available during the design and bidding stages of trenchless projects has a significant impact on the selection of construction methods. The estimated production rates, ground movements, jacking forces, shaft design, and maximum drive lengths are all dependent on the available subsurface information (Klein et al., 1996). The degree of uncertainty over subsurface conditions will manifest itself in the amounts of contingency money included in the bid.
Geotechnical investigations for trenchless projects should typically have three general phases. These phases progress from planning, to investigation, and finally reporting. These phases are closely coordinated, and an iterative approach is used (Richardson et al., 2003). Figure 2.27 shows a proposed iterative approach with possible inputs and outputs.

![Figure 2.27. Suggested iterative approach for geotechnical investigation for trenchless technology (from Richardson et. al, 2003)](image)

The planning stage of a trenchless construction project requires the development of a preliminary ground surface survey. Existing geological or geotechnical reports, maps, aerial photographs, and depositional history are important tools for developing this preliminary geotechnical survey. Najafi (2005) lists several examples of information that may be inferred from depositional history of an area. For example, if the area has been subjected to
glaciation, the presence of cobbles, boulders, and gravel may be expected. These obstacles have the potential to unexpectedly deflect the path of the bore if they are not accounted for. Additionally, if the area has been subjected to large landslides, trees and other objects could be encountered below ground. If the area has seen low energy, meandering streams and rivers, then fine-grained deposits may be expected. While each trenchless project has unique, site specific requirements, Najafi (2005) suggests that a survey should be conducted for at least 50 feet on either side of the bore path. Najafi suggests that the predesign surface survey should include the following elements.

- Work area requirements
- Existing grade elevation data
- Surface features such as roadways, sidewalks, utility poles
- Boring or test pit locations
- Waterways and wetlands
- Visible subsurface utility landmarks such as manholes or valve boxes
- Structures adjacent to the bore path

The preliminary design stage includes subsurface investigations as the next step after the surface survey. The important subsurface information to note is the presence of existing utilities or other manmade obstructions, methods of placement, and the geotechnical conditions along the proposed trenchless construction alignment (Najafi, 2005). Simple and low risk installations can often utilize an abbreviated program of geotechnical investigation.

The first step of the subsurface investigations usually involves obtaining information about existing utilities along the bore path. Usually, this involves using a local “one-call” service to come to the site to perform the utility locates. In the absence of such a service, local municipalities and utility companies should be contacted to obtain the required information. Methods of confirming subsurface utility locations include surface applied pipe locators, ground penetrating radar (GPR), vacuum excavation equipment, and seismic survey (Najafi, 2005).
The second step of the subsurface investigation for the trenchless construction project is the geotechnical subsurface investigation. The geotechnical subsurface investigation gives more precise information on subsurface conditions on the site. Najafi (2005) specifies that the steps for subsurface investigation should include the following:

- Determining the nature of soil at the site and its stratification
- Obtaining disturbed and undisturbed soil samples for visual identification and laboratory tests
- Determining the depth and nature of bedrock, if encountered
- Performing in situ field tests
- Observing surface drainage conditions from and into the site
- Assessing any special construction problems with respect to the existing structures nearby
- Determining groundwater levels, sources of recharge, and drainage conditions

Various methods are being used to conduct subsurface investigations, with vertical test borings being the most common. Najafi (2005) lists the main methods of geotechnical surveys as follows:

- **Ground penetrating radar (GPR).** Effective in gravels and sands.
- **Acoustic (sonar).** Useful for determining depth of rock, interfaces between soft and hard deposits, and buried objects.
- **Geophysical methods.** Variations in the speed of sound waves or in the electrical resistivity of various soils are useful indicators of the depth of water table and of the bedrock.
- **Test pits or trenches.** This method is suitable for shallow depths only but allows visual observation over a larger area than is possible with samples from borings.
- **Hand augers.** Suitable only for shallow depths; only disturbed or mixed samples of soil can be obtained in this method.
- **Boring test holes and sampling with drill rigs.** This is the principal method for detailed soil investigations. Sampling interval and technique should be set to accurately describe the subsurface material characteristics taking into account the site-specific conditions. Typically, split spoon samples will be taken in soft soil at 5-ft depth intervals in accordance with ASTM D-1586 (Najafi, 2005).

A limitation of conventional geotechnical investigations that drill vertical boreholes is that only a noncontinuous picture of underground conditions is developed. Drilling the large number of vertical boreholes that would be necessary to provide a complete picture of subsurface conditions for horizontal alignments is often not technically or economically feasible. An additional limitation of conventional, vertical site characterization techniques is that they often cannot reach underneath structures, roadways, pipeline right-of-ways, or environmentally sensitive areas (O’Reilly and Stovin, 1996).

In response to these limitations, emerging horizontal site characterization techniques now provide a new alternative to the traditional vertical site investigation methods. These techniques include a family of soil samplers, contact sensing probes, and borehole geophysical tools capable of providing horizontally continuous geotechnical information. These devices are usually advanced into the ground using horizontal directional drilling technology.

Allouche et al. (2001) states that a site characterization project that involves horizontal boring will be economical even for medium-scale microtunneling and tunneling projects. The increasing economic feasibility of this method is directly related to the improvements made in the horizontal directional drilling industry. Allouche et al. (2001) also presents a methodology for the selection and deployment of horizontal site investigation techniques in trenchless construction projects and lists the different state-of-the-art devices available (see Table 2.5).
Table 2.5. State-of-the-art horizontal sampling equipment (from Allouche et al., 2001)

<table>
<thead>
<tr>
<th>Name</th>
<th>Description</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple-port soil sampler</td>
<td>A soil sampler capable of collecting up to six samples from the wall of a horizontal borehole during a single pass. The sampler is controlled from the surface via a laptop computer. High quality samples.</td>
<td>Deployed in a predrilled continuous borehole. Suitable for cohesive and unconsolidated soils. Capable of collecting six samples as well as deploying miniature CPT.</td>
</tr>
<tr>
<td>Ditch-Witch soil sampler</td>
<td>After the pilot bore is completed, the device is advanced into the borehole following original alignment. Soil sample collected is of relatively low quality. Length: 0.5 m; diameter: 62 mm.</td>
<td>Suitable for soft to firm cohesive soils as well as unconsolidated soils. In soft ground sampler may deviate from path of pilot bore.</td>
</tr>
<tr>
<td>Punch-Master 2000</td>
<td>After the pilot bore is completed, the device is advanced into the borehole following original alignment. Operates similarly to a Shelby tube. Samples collected of high quality. Sampler: Length: 7 m; diameter: 120 mm.</td>
<td>Can be used only with medium or large size rigs. Suitable for a wide range of soil conditions.</td>
</tr>
<tr>
<td>Devico continuous rock coring method</td>
<td>Normal drilling techniques used to advance drilling string to target area. Mud motor system collapsed, pulled to surface through drilling pipe, and replaced with a core barrel. Core samples are retrieved using a trip-wire. Coring tube diameter: 50-150 mm; coring tube: 3 m; range: 1,500 m.</td>
<td>Involves specialty equipment and thus is a relatively costly method. Suitable only for rock formation.</td>
</tr>
<tr>
<td>Microdrilling</td>
<td>A 50 mm diameter coiling tubing equipped with a drill bit and mud motor. Range: 100-150 m; suitable for alluvial sediments and rock formations; data collection using miniature geophysical tools.</td>
<td>Currently under development by the U.S. Department of Energy/oil and gas exploration industry.</td>
</tr>
<tr>
<td>Horizontal prebore cone penetration</td>
<td>Cone is pulled through a 100 mm diameter prebored hole. Data related to tip resistance and sleeve friction is transmitted to surface. Length: 1.0 m; diameter: 150 mm.</td>
<td>Susceptible to the disturbance of the formation caused by the drilling operation and drilling fluid.</td>
</tr>
</tbody>
</table>

The preliminary investigation continues with a program of laboratory testing on the recovered soil samples. Najafi (2005) recommends that the following soil information should be determined from the laboratory testing:
• Standard classification of soils
• Gradation curves on granular soils
• Standard penetration test (SPT) values where applicable (generally unconsolidated ground)
• Particle size distribution, including presence of cobbles and boulders
• Shear strength
• Atterberg limits (liquid, plastic, and shrinkage limits)
• Moisture content
• Height and movement of water table
• Permeability
• Cored samples of rock with lithologic description, rock quality designation, and percent recovery
• Unconfined compressive strength for representative rock samples (frequency of testing should be proportionate to the degree of variation encountered in the rock core samples); and Mohs hardness for rock samples. Where rock is encountered, it should be cored in accordance with ASTM D-2113 to the maximum depth of the boring
• Presence of contaminated soils (hydrocarbons, etc.)

In the final stage of the geotechnical investigation, the owner should prepare a Geotechnical Baseline Report (GBR) which sets a common understanding for bidding on the project. The GBR establishes a contractual statement of the geotechnical conditions anticipated to be encountered during underground or subsurface construction (Najafi, 2005). This allows the contractor to make bids using reliable information. The industry standard is to include both basic project data and the GBR as part of the construction contract. Richardson et al. (2003), states that a newer approach is to take design information out of the above reports and include it in a separate design report which is excluded from the contract. This is because the design recommendations are sometimes used by the contractor in a way not intended by the engineer.
However, in practice detailed subsurface investigations are often not done because of the difficulty of quantifying the benefits of a given level of investment in site characterization. This can sometimes lead to insufficient funding and inadequate subsurface information (Allouche et al., 2001).

2.5 Effect of Trenchless Technologies on Surrounding Soil

The effect of different trenchless methods on the surrounding soil is a topic that is still being studied. The uncertainty of what problems might be encountered underground is a common reason that owners will specify open-cutting for a project that might be better suited to trenchless methods. Trenchless construction methods are considered to carry a level of risk for soil related problems.

The primary subsurface risks associated with trenchless construction are heave, subsidence, frac-out, and collision with underground obstacles. Frac-out is a common term for the hydraulic fracture of the borehole walls due to drilling fluid pressure. Hydraulic fracture occurs when fluid pressures within the borehole exceed the shear strength or undrained cohesion of the strata (Lueke and Ariaratnam, 2005). Different models have been developed to simulate and predict borehole pressures (Ariaratnam et al., 2007). Soil settlement may occur mainly as a result of loss-of-soil occurring during tunneling and because of dewatering operations that lead to subsidence. During a trenchless technology project, loss-of-soil may be associated with soil squeezing, fluid running or flowing into the heading, soil losses due to the size of overcut, and steering adjustments. The actual magnitudes of these losses are largely dependent on the type and strength of the soil, groundwater conditions, size and depth of the pipe, equipment capabilities, and the skill of the contractor in operating and steering the machine. If passive earth pressure is exceeded, heave of ground surface may occur, causing damage to nearby utilities and other structures.

HDD is particularly susceptible to subsurface deformations due to the method’s use of drilling fluid and because of the presence of some radial soil displacement. Allowable
drilling pressures and ground improvement protections are considered by researchers to be primary mitigation tools. Cavity expansion theory can be used to create a model that provides a quantitative assessment of drilling fluid limit pressure and minimum depth of cover requirements (Francis et al., 2003).

The effects of radial soil displacement from trenchless construction can have different significance based on the type of adjacent structure and its position. Boring that expands soil radially alters the stress state of the soil. The underground conditions, diameter of new tunnels, types of existing pipe, and the general underground orientation all have effects on the induced stresses and strains. Additionally, different types of pipe have different sensitivities to movement. For instance, asbestos-cement pipes are particularly sensitive, while HDPE pipes are not. Different methodologies exist to try to model this action using cavity expansion theory such as those outlined in Marshall and Knight (2003) and Hunter (2005).

During trenchless construction that uses a jacking force to advance the pipe and cutter head, surface subsidence mainly occurs due to a lack of driving force. Excessive driving force, however, can cause surface heaving if soil is being excavated faster than it can be removed. Additionally, the overburden pressure due to the depth of the pipe is important in determining the proper driving force that will not lead to surface deformations (Shou and Chang, 2006).

Trenchless rehabilitation methods are considered to have little to no effect on the existing soil, with the exceptions of pipe bursting and pipe splitting. These methods both expand the soil outward, so it is important for the designer to understand and predict ground displacements when considering safe distances to existing underground structures and overlying pavement. Chapman et al. (2003) shows that an elliptical expansion of the soil best represents the effects of pipe splitting.
2.6 Quality Control / Quality Assurance (QC/QA) methods

QC/QA is very important in trenchless projects because of the added level of complexity of most methods as compared to traditional open-cutting. Quality control involves techniques and activities aimed both at monitoring processes and eliminating the causes of unsatisfactory performance. Several California municipalities have suspended HDD activities in their jurisdictions as a result of poor performance that resulted in heaved roads, damaged sidewalks and foundations, and repeated collisions with existing buried utilities. With nearly 4,000 HDD rigs operating daily across North America, avoiding hitting existing buried utilities has become a major challenge. Owners of utilities systems have begun prosecuting HDD contractors who repeatedly damage buried lines. (Baumert and Allouche, 2003)

Designing a sufficiently sound trenchless renewal system and monitoring the key elements of the finished product’s installation assures long-term performance. Adequate on-site inspection and post-installation QA/QC is required to confirm compliance with the performance requirements (Kampbell and Whittle, 2003).

Baumert and Allouche (2003) propose implementing a formal QC/QA program using the framework of ISO 9000, a generic series of QC/QA standards accepted worldwide. Individual firms and the trenchless industry as a whole can reap many benefits from the adoption of a formal QC/QA program, a management tool aimed at optimizing day-to-day operations and procedures to minimize or eliminate poor project outcomes. Singh (1997) discusses total quality as it pertains to trenchless construction, with an emphasis on contracting practices.

Problems occurring in HDD usually arise because of unforeseen conditions, and, in some cases, due to poor drilling practices (Bennett et al., 2004). Geotechnical data must be much more accurate for trenchless construction than for open-cut. An overview of the most common operations risks in HDD installations is shown in Table 2.6.
## Table 2.6. Operational risks in HDD installations (Baumert and Allouche 2003)

<table>
<thead>
<tr>
<th>Risk</th>
<th>Cause(s)</th>
<th>Potential consequence(s)</th>
<th>Product type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent deformation</td>
<td>Excessive pulling force during pull-back</td>
<td>Reduced mechanical strength, reduced useful life</td>
<td>MDPE, HDPE</td>
</tr>
<tr>
<td>Rupture</td>
<td>Excessive pulling force during pull-back</td>
<td>Failure of installation, loss of borehole, loss of pipe product</td>
<td>MDPE, HDPE, PVC</td>
</tr>
<tr>
<td>Scratching and denting</td>
<td>Sharp stones or other objects projected into borehole</td>
<td>Reduce pressure rating, initiation of crack propagation</td>
<td>MDPE, HDPE, PVC</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Corrosion</td>
<td>Steel</td>
</tr>
<tr>
<td>Kinks</td>
<td>Failure to maintain minimum bending radius/presence of large obstacles along bore</td>
<td>Onset of local buckling</td>
<td>Steel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reduce ovality</td>
<td>MDPE, HDPE</td>
</tr>
<tr>
<td>Failure of joint</td>
<td>Excessive bending</td>
<td>Failure of installation, loss of product</td>
<td>PVC</td>
</tr>
<tr>
<td>Damaging an existing utility</td>
<td>Poor as-built information, lack of/unsatisfactory locate, poor drilling practices</td>
<td>Injuries to crew/passersby, cost of repair itself, project downtime, secondary damage, downtime cost for utility provider</td>
<td>All pipes and conduits</td>
</tr>
<tr>
<td>Frac-out</td>
<td>Poor soil conditions, poor drill path planning, poor drilling practices</td>
<td>Reduced public satisfaction, hazard to traffic (slippery roads), health hazard (in case of contaminated ground), Environmental damage to local ecosystem (e.g., aquatic environments), visually unpleasant</td>
<td>All installations</td>
</tr>
<tr>
<td>Surface heave</td>
<td>Insufficient burial cover, excessive pilot drilling or reaming rates, failure to use sufficient volume of drilling fluids, borehole enlargement increment too large</td>
<td>Cracked roads and driveways, heaved sidewalks and pedestals, damage to adjacent utilities and foundations</td>
<td>All installations</td>
</tr>
<tr>
<td>Failure of installation to meet technical requirements</td>
<td>Poor drilling practices, inadequate soil investigation, improper selection of pipe product, adverse soil conditions</td>
<td>Failure to complete installation, failure to exit borehole within acceptable window, failure to maintain grade and alignment within prespecified tolerance</td>
<td>All installations</td>
</tr>
</tbody>
</table>
Bennett et al. (1995) state the importance of logging and monitoring the load data associated with HDD construction. However, Baumert and Allouche (2003) state that this quality control measure is rarely done in practice due to the lack of availability of monitoring technology.

Quality control in trenchless rehabilitation projects tends to be simpler and consists mainly of correctly following project specifications and accepted ASTM standards. The ASTM F 1216, “Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube”, controls the design of all of the widely used close-fit liners.

For CIPP, because it is cured in-place, it is important that a sample of the finished wall section be obtained directly after installation and that the sample then be analyzed for the specified finished thickness and the material’s engineering properties. The most important parameter that is used to identify the structural properties of a liner for gravity piping is the flexural modulus of elasticity in bending, which is determined by a flexure test. This value is significant because it is a major factor in the ASTM F 1216 calculations that determine the required wall thickness of a liner installation.

Kampbell and Whittle (2003) state that for pre-manufactured materials such as Fold & Form and Deformed/Reformed liners, quality control in the field is simplified. This is because the structural properties of these thermoplastic materials are established under ASTM prescribed QC/QA protocols common to all plastic pipe production.

Municipalities should inspect their sewer systems every 2 years for older pipes and every 3 to 4 years for relatively new pipes (Najafi, 2005). This is done as part of regular maintenance to determine the condition of lengths of pipeline.
2.7 Design Process

Various decision support systems (DSS) have been developed to assist asset managers and construction practitioners in assessing the different strengths and weaknesses of different construction methods as they relate to characteristics of a specific project (Chung et al., 2004). These systems mathematically evaluate construction options and indicate the most appropriate construction method for the project. A brief comparison of various method evaluation models is given by Allouche and Parhami (2003). The successful design of a trenchless construction project requires following the steps listed below (Najafi, 2005):

- Identification of the requirements for the new pipeline
- Conducting surface and subsurface investigations
- Identification of feasible trenchless technology alignments
- Selection of an appropriate trenchless construction method
- Implementation and modeling

The design of a trenchless pipeline renewal system includes five steps, as listed (Najafi, 2005):

- Identification of pipe conditions and problem recognition and classification
- Prioritization of problem considering strategies and long-term plans
- Selection of an appropriate pipeline renewal method
- Designing renewal methods based on project specific conditions
- Implementation and modeling

McKim (1997), put forth an additional method selection procedure for either trenchless construction or trenchless rehabilitation, that divides the decision making process into assessment, decision, and execution steps. In the assessment step, the necessary or existing (depending on whether the project involves trenchless construction or rehabilitation) hydraulic characteristics and structural characteristics are analyzed. The required function
and capacity are then considered in the design making step, and the decision is made on the repair or upgrade, if a rehabilitation project. A selection of method can then be made and executed.

Zembillas (2003) recommends the increased use of subsurface utility engineering (SUE) for both trenchless construction and rehabilitation. This branch of engineering involves managing risks associated with utility mapping at appropriate quality levels. SUE can be applied to utility coordination, utility relocation, utility condition assessment, communication of utility data to concerned parties, utility relocation cost estimates, implementation of utility accommodation policies, and utility design. A lack of accurate information on the location of existing underground utilities can result in costly conflicts, damages, delays, service disruptions, redesigns, claims, and even injuries and lost lives during construction activities.
3 TRENCHLESS TECHNOLOGIES SURVEY AND INTERVIEWS RESULTS

3.1 Surveys

Additional information about trenchless technologies was obtained by surveying and interviewing professionals working in trenchless-related fields. Findings from these surveys and interviews are discussed in this chapter, and more detailed responses can be found in Appendix A. Three separate surveys were sent to professionals, with each survey targeting a different geographic region. The surveys targeted Iowa, the Midwest, and the entire United States. These surveys and interviews focused on the four major topics listed below.

- Method familiarity
- Observed pavement distress
- Reliability of methods
- Future improvements

An initial 22 question survey was sent to professionals operating in the state of Iowa (see Appendix B). Thirty-four responses were received, and the respondents’ professional backgrounds are shown graphically in Figure 3.1. The majority of respondents were from design fields, with the highest specific groups being city and consulting personnel.
A second survey was released to a wider group that included professionals working in the Midwest regional states bordering Iowa (see Appendix C). This survey contained only 10 questions; however the questions and topics covered were very similar to those in the original Iowa survey. The Midwest survey garnered 32 responses, and the fields of those respondents are shown in Figure 3.2. The similar number of respondents of the Iowa and Midwest surveys (34 and 32, respectively) makes the comparison of results more useful. Observation of the Midwest survey shows that a higher number of contractors took part in this survey, while fewer city workers and roughly the same number of consulting workers responded as compared with the Iowa survey. Overall, it would appear that fewer designers and more contracting and manufacturing/sales personnel responded to the Midwest survey in comparison with the Iowa survey. This slightly different demographic may have resulted in differing responses in the two surveys.
Figure 3.2. Trenchless related backgrounds of respondents to the Midwest survey

Questions from the second survey (see Appendix C) were included in a larger survey conducted by Dr. Mohammad Najafi at the University of Texas at Arlington. This third survey was sent across the United States and gained 14 responses, all from city workers (Figure 3.3). The geographical distribution of the national survey is shown in Figure 3.4.
All three surveys questioned respondents on the types of trenchless technologies they had experienced in practice. Respondents were asked to select from a list each of the methods that they had encountered. The question was slightly different in the three surveys, in that the Midwest and national surveys included compaction tools and tunneling methods as
additional suggested responses that were not present in the Iowa survey question. Despite this slight inconsistency, the questions are useful for understanding what trenchless technologies are commonly used in Iowa, the Midwest, and nationally, and also provide more information on the current level of familiarity with different trenchless methods. Results from the Iowa survey are shown in *These categories were not included in the Iowa survey. Figure 3.5, results from the Midwest survey are shown in Figure 3.6, and results from the national survey are shown in Figure 3.7.

*These categories were not included in the Iowa survey.

**Figure 3.5. Trenchless technologies experienced in practice by respondents to the Iowa survey (some respondents selected more than one method)**
Figure 3.6. Trenchless technologies experienced in practice by respondents to the Midwest survey (some respondents selected more than one method)

Figure 3.7. Trenchless technologies experienced in practice by respondents to the national survey (some respondents selected more than one method)
The survey responses demonstrated that horizontal directional drilling was the method most often encountered by professionals in Iowa and the surrounding Midwest. Iowa respondents also reported significantly higher experience with horizontal auger boring than respondents working in the Midwest or nationally. A high use of cured-in-place pipe and localized repair as trenchless rehabilitation methods reported in the Midwest survey may partially reflect more responses by contractors and manufacturing/sales people who may specialize in trenchless rehabilitation.

The high level of familiarity with sliplining reported by respondents to the national survey suggests that sliplining may be practiced more commonly outside of the Midwest. Additionally, a lower level of HDD use outside the Midwest is also suggested by the results.

The following question dealt with pavement distress caused by trenchless technologies. Just under half of respondents to the Iowa survey reported seeing pavement distress or other problems as a result of using trenchless methods (Figure 3.8). Respondents to the Midwest survey were asked a similar question inquiring whether they had encountered pavement deformations caused by trenchless methods. In this case, less than a third of Midwestern respondents said they had seen deformations occur (Figure 3.9). This question was not present in the national survey.
Respondents to the Iowa survey were later asked to elaborate on their experiences with pavement deformation due to trenchless construction. Respondents explained the causes of soil deformations that can lead to pavement heave or settlement. The most common responses are listed below.
- Soil types on site
- HDD drilling fluid
- Auger boring through wet, sandy material
- Large annular spaces
- Over-excavation by head of auger boring machine
- Shallow depth of bores
- Method unreliability

The Iowa Survey respondents were also asked to document the amount of vertical soil displacement caused by trenchless methods of installation experienced by the respondents. Respondents noted that that the soil type, depth of construction, size of borehole, and method all have an effect. A lower depth was thought to generally cause less surface displacement. Experienced displacements are observed to vary. One respondent reported observing settlement of 0.5 to 3 inches in clay soils. Another respondent reported heave of 1 to 2 inches in clay, about 2 inches in gravel, and no heave in sand. It was noted that pipe bursting, pipe ramming, and HDD have the potential to cause heaving while auger boring, pipe jacking, and microtunneling can potentially cause settlement. CIPP, sliplining, and localized repairs are not prone to cause heaving or settlement. Sands were thought to cause little surface heave, while clay could potentially cause several inches of heave, as could gravel.

Iowa survey respondents were also asked what current QC/QA methods are currently being used for trenchless project. Responses are summarized below.

- Television inspection and pressure test after gravity sewer installation
- Laser, leak, and pressure test
- Pressure test after pressure pipe installation
- Grade and alignment check
- Potholing in HDD.
- Lack of QC/QA reported for auger boring reported by a respondent
- Geophones and ground penetrating radar
- Independent testing labs for rehabilitation materials

The Midwest and national surveys asked respondents if they felt current levels of QC/QA associated with trenchless projects were appropriate. About 40% of respondents to the Midwest survey answered that current levels of QC/QA were not appropriate (Figure 3.10). None of the national survey respondents felt that current levels of QC/QA were appropriate (Figure 3.11), with 6 respondents indicating “no” and 8 respondents with no opinion. These negative responses are surprising considering the positive responses of the Midwest survey.

Figure 3.10. Respondents of the Midwest survey who felt current levels of QC/QA associated with trenchless projects are appropriate
Midwest, Iowa, and national survey respondents were then asked to elaborate on why they felt current levels of QC/QA associated with trenchless projects were or were not appropriate. Responses were mixed, but it was generally felt that current methods are not always adequate. Common answers are listed below.

- Lack of well trained inspectors
- Lack of soil boring along the route of the bore
- Current lack of real-time monitoring of ground movements
- Overall inexperience of personnel involved

Iowa survey respondents were asked about the currently used methods of soil investigation prior to trenchless construction. The most common methods are listed below.

- Vertical soil borings.
- Soil classification and water table depth.
- No soil borings. Instead rely on the experience of the engineer, client, and local contractors.
The Iowa survey asked respondents what lessons could be learned from failures experienced during trenchless operations. Common responses are given below.

- Geotechnical exploration is critical before starting trenchless construction.
- Accurately locating existing utilities is very important.
- Experienced contractors are very important.
- The contractor should monitor the amount of material removed from the casing during auger boring as the casing is advanced to minimize the amount of overexcavation.
- HDD boring should be conducted deeper under sidewalks.
- Use a high quality closed circuit television before placing a liner.

The Iowa survey asked which trenchless method the respondents considered the least favorable (Figure 3.12). Pipe bursting was cited more than any other method. When asked to explain their selection, respondents pointed to concerns about soil displacement around the burst pipe. Also, it was stated that the process requires excavation of service connections and modifications to each manhole. Additionally, there is a large potential for other utilities in the area to be adversely affected, and these negative effects may not be noticed immediately.
These categories were not included in the Iowa survey.

Figure 3.12. Iowa survey responses of which trenchless method the respondents consider the least favorable

The Midwest and national surveys also asked the respondents to rate the reliability of trenchless technology as a rehabilitation and construction solution. Using a rating scale of 1 being poor and 5 being excellent, 90% of Midwest respondents gave the reliability of trenchless technologies a 4 or 5 rating (see Figure 3.13). Exactly 50% of respondents selected a rating of 4, which can be interpreted to mean that there is a feeling among those familiar with trenchless technologies that the reliability of the methods could be improved somewhat. National survey respondents expressed a more negative view of the reliability of trenchless technology. Five of the 14 respondents gave reliability a rating of 3 out of 5 (see Figure 3.14). This more negative view may be due to the lack of experience with trenchless methods expressed earlier, or it may reflect difficulties encountered with new methods.
Respondents to the Iowa survey were asked to list research and improvements that could be made to trenchless technologies to make them more feasible. These are listed below.

- More requirements on exact final location of the piping after installation
- Improved machine control and monitoring systems
- More certification programs for contractors
• Tightening specified tolerances
• Develop a cost effective QC/QA program to reduce risk

3.2 Interviews

Project researchers also individually interviewed people involved in trenchless projects and obtained useful anecdotal information on trenchless technologies and their use. Comments related to trenchless design considerations and soil testing are given below.

• Experienced contractors need to be given the flexibility to use whatever methods they deem necessary to complete a job given the necessary specifications. Contractors’ practical experience can be a valuable design resource and engineers’ designs shouldn’t be too inflexible.
• The HDD and auger boring contractor receives engineers’ soils reports for many projects. They look for blow counts, water content, and location of water table. They interpret soil with blow counts of 1 or 2 to indicate a soil of “toothpaste” like consistency, blow counts of 10-25 to indicate reasonable soil, and blow counts of >50 to indicate rock.
• A contractor stressed that the uncertainties in trenchless construction and the many variables make best practice design guidelines unreliable. Additionally, best practice guidelines add liability.
• Additional soil testing could be useful for HDD in rocky and sandy soils, however, there is usually little benefit to conducting additional soil testing in familiar areas.
• Additional soil testing is not necessary if potholing is conducted. During potholing the contractor can make an assessment of the soil.
• Rule of thumb: stay 2 feet from any other utility.
• In moling, for each 1 inch of borehole diameter, 10 inches to 1 foot of soil cover is required.
• In pipe bursting, you need 2 feet of clearance between the pipe to be burst and the nearest other pipe.
Comments related to difficulties encountered during trenchless construction are summarized below.

- Projects with large pipes are very difficult because large pipes displace so much soil.
- It can be very difficult to predict heave and settlement. This organization has tunneled 1 foot deep with no settlement while at 20 feet deep they have gotten settlement.
- Several city designers told researchers that a franchised utility company installed conduits approximately 4 inches to 6 inches in diameter at shallow depths. These were the biggest threat of surface heave and overlying pavement cracking.
- An HDD equipment vendor explained that “people can do everything right and still get heave”. The vendor also said that asphalt pavement will heave more easily than PCC pavement.
- An HDD contractor said that the three most common causes of frac-out and heave were excessive speed, which could cause outrunning the drilling fluid, using a machine that is too small to execute the pullback process correctly, and incorrect drilling fluid.
- An auger boring contractor stated that soil is unlikely to heave when boring through clay. Heave would occur only if the contractor pushed the casing too fast, and it compressed soil faster than it could be augered out. Except for the previously described circumstance, the bore would need to be very shallow before most experts would have a concern about heaving.

During an interview, a city designer told the research team about an HDD bore that was performed in response to a utility conflict in which the existing line was too shallow and had to be moved. The utility company hired an engineering consulting firm to make the plans. This is an unusual practice as utility companies develop informal plans in-house. The
subcontractor installed 6 PVC pipes of 4 inches in diameter, all in one 21 inch diameter borehole. A 2 to 3 meter distance was allowed between the boring and a nearby retaining wall. Apparently, bentonite was over-pumped through what may have been shale. This led to surface heave issues all along the route. In order to correct the damage, the subcontractor and the utility company split the $140,000 remediation cost. Unfortunately, soil information was not sought by the utility company or the contractor until after the construction was complete.

Another project involved a mistake reading the plans during an HDD operation that resulted in the bore being made at half the specified depth. This resulted in the drill hitting existing underground electrical lines and television cables. The usual practice for soils information on these HDD projects is not to perform soil testing for the project, but rather to rely on previously collected soil data.

On another HDD project, a problem was encountered when a reamer was lost 15 feet underground. An example of pavement cracking resulting from a shallow HDD bore is shown in Figure 3.15.

![Pavement cracking](image)

**Figure 3.15. Pavement cracking caused by an HDD installation**
HDD installations that are not back reamed to a sufficiently large diameter have been observed to cause heave. When the product pipe is pulled into the hole, some of the drilling fluid is displaced and must flow out of the hole. The drilling fluid is expected to pass in the opposite direction that the pipe is being pulled and therefore must travel through the annular space between the outside of the pipe and the edge of the hole. The rule of thumb for HDD is that the diameter of the hole should be 1.5 times the outside diameter of the pipe. However, sometimes contractors do not include the thickness of the pipe and bells or other protrusions on the outside of the pipe when they calculate pipe diameter. If the machine generates high enough pulling force, drilling fluid pressure can become high enough to heave the soil. The designer recalled an instance in which this occurred along an entire installation, heaving soil up to 1.5 feet and disrupting lawns and driveway pavements during the installation of 12 inch diameter conduit at a 5 foot depth (see Figure 3.16).

![Surface Heave](image)

**Figure 3.16. Surface heave on an HDD project**

Settlement was observed during auger boring installations through sandy soil. Soil volume was lost due to sand caving-in just ahead of the advancing casing. This resulted in the contractor extracting more material than is appropriate for the amount that the pipe advanced.
The designer has seen voids as deep as five feet develop under pavements. In one case, a pavement did not settle because the slab actually bridged over the void and the steel reinforcement was strong enough to continue to support vehicles.
4 SITE INVESTIGATION

4.1 Introduction

Numerous trenchless construction projects were observed as part of the study. The research team visited trenchless jobsites in Iowa with the goal of gaining a better understanding of how this work is performed, and to better identify the risks involved in trenchless technologies, and how those risks can be minimized. A total of 19 projects were visited, which included auger boring, horizontal directional drilling, tunneling, pipe jacking, and impact moling (see Figure 4.1).

![Map showing locations of trenchless construction projects](image)

Figure 4.1. The locations of trenchless construction projects visited

The approach of the research team to these projects is divided into two categories. The first category is the “Site Visit” group, in which the research team made visits to a job site to
observe and document construction practices, evaluate soil properties, and document the successes and failures experienced. A total of 13 projects were observed in this manner.

The second category of projects was the “Field Monitoring” portion which involved 6 projects. Field monitoring was confined to central Iowa due to logistical challenges. The goal of the “Field Monitoring” was for a more in-depth study of trenchless construction projects. Undisturbed soil samples were recovered for laboratory testing, soil stresses were measured during the construction, and more in-depth investigations of projects were completed. Falling weight deflectometer testing was planned to examine the effect of trenchless construction on overlying pavements, but scheduling problems prevented this testing from being carried out.

The 19 total projects included 1 pipe jacking, 1 tunneling, 1 impact moling, 5 auger boring, and 11 HDD projects. Details of these projects are provided in Table 4.1. Pipe sizes installed ranged from 0.75 inches in diameter up to a 10 by 5 foot box culvert. Installation lengths ranged from 24 up to 495 feet. Two of these projects experienced ground movement caused by the trenchless construction. These problems both involved frac-out of drilling fluid during HDD, and one of them also involved surface heave. Each of the 19 projects is described in detail in the chapter.
Table 4.1. Summary of projects

<table>
<thead>
<tr>
<th>Site</th>
<th>Trenchless Method</th>
<th>Pipe Diameter (in)</th>
<th>Length (ft)</th>
<th>Depth (ft)</th>
<th>Comments</th>
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<tbody>
<tr>
<td>Des Moines, Keo Way</td>
<td>pipe jacking</td>
<td>10’x5’</td>
<td>24</td>
<td>20</td>
<td>Box culvert</td>
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<td>Chickasaw County</td>
<td>tunneling</td>
<td>66</td>
<td>44</td>
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<td>Ankeny, State Street</td>
<td>auger bore</td>
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<td>170</td>
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<td>16</td>
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<td>110</td>
<td>22</td>
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<td>80, 80, 80</td>
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<td>Ames, Johnny Majors Field</td>
<td>HDD</td>
<td>2-4” together</td>
<td>400, 400</td>
<td>17</td>
<td>Heave, frac-out</td>
</tr>
<tr>
<td>Des Moines, 62nd and Grand</td>
<td>HDD</td>
<td>16</td>
<td>140</td>
<td>12</td>
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<tr>
<td>Ames, Osborn Drive 2</td>
<td>HDD</td>
<td>8</td>
<td>330</td>
<td>6-9</td>
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<td>Ames, Osborn Drive 3</td>
<td>HDD</td>
<td>8</td>
<td>85, 495, 325</td>
<td>6</td>
<td>Some frac-out</td>
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<tr>
<td>Ames, Seed Science Bld.</td>
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<td>6</td>
<td>240, 240</td>
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<td>180, 180</td>
<td>6</td>
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<td>120</td>
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<td>480</td>
<td>6</td>
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</tr>
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<td>4</td>
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</tr>
<tr>
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<td>HDD</td>
<td>8</td>
<td>30</td>
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<td></td>
</tr>
</tbody>
</table>
4.2 Site Visits

Auger Boring at State Street, Ankeny, IA

Project Information
This project was located on State Street in Ankeny, Iowa, between Oralabor Road and Magazine Road (see Figure 1) during April, 2007. The auger boring technique was used to install a 36 inch diameter steel casing for a 24 inch polyvinyl chloride (PVC) sanitary sewer pipe. The casing was bored for a length of 180 feet at a depth to top of pipe of 20 to 30 feet. Accuracy was important because the gravity-flow carrier pipe had to meet slope requirements.

The auger bore was set up at point A and bored west to point B as shown in Figure 4.2.

Figure 4.2. The location of the auger boring project on State Street in Ankeny, IA. (Bore path in red)
Trenchless Method Selection

Trenchless construction was selected to avoid closing the four lane separated State Street (see Figure 4.3). Auger boring was used by the contractor on this project. Although the contractor has experience with horizontal directional drilling, the contractor chose to use the auger boring procedure because the steel casing was 36 inches diameter, which is larger than the normal sizes that the contractor worked with for HDD. Also, the large depth of 20 to 30 feet would require that a direction drilling rig be set up at a considerable distance away from the road in order to have room for the bore to descend from the surface to the prescribed depth.

Soil Conditions

No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to a familiarity with soil in the area. The soil was expected to be mostly weathered shale (blue clay) with sand. The elevation of the bore was below the ground water table.

Trenchless Installation

- After nearby existing utilities were located, the launching pit at point A was dug and steel shoring was installed. Then, track for the boring machine was laid at the proper line and grade in the launching pit on a crushed rock base. The auger boring machine was then lowered into the launching pit and placed on the tracks as shown in Figure 4.4.

- The auger boring machine had a waterline system to measure the grade of the steel casing and a fluid supply line in case it was needed to deliver drilling fluid or water to the cutting face. A steering rod was also attached to the top of the casing to allow slight adjustments to be made to the grade of the steel casing.

- The first 20 foot section of casing pipe was lowered into the launching pit along with the 20 length of auger inside of it. The pipe and auger were connected to the machine and the boring then began (see Figure 4.5). Soil cuttings were transported back through the casing pipe to the launching pit. A backhoe was used to remove the cuttings to the
ground surface. For each new 20 feet long pipe section, the crew checked the line and grade before welding it to the pipeline.

- After about 100 feet of boring, a sand seam (with no boulders) was encountered (see Figure 4.6). The sand was accompanied by a significant quantity of water, which flooded the borehole and left 6 inches of standing water in the launching pit, significantly slowing the installation process. This water flow was due to the high permeability of the sand and the depth of the bore beneath the water table. The loss of volume due to water flow was expected to result in a void forming under the pavement. To avoid future settlement problems and to be able to continue the installation process, the contractor pumped 5 cubic foot of grout to the cutting face through one of the small pipes located immediately above the casing pipe. Pumping of grout continued until the pumping pressure started to build up and reached an acceptable level, at which point the void was considered to be filled. No subsidence or other damage to the pavement was observed, and this would have been a result of the actions of the contractor.

- Boring through the sand continued for about 30 feet until the soil transitioned back to weathered shale until the completion of the installation (see Figure 4.7 and Figure 4.8). The boring through sand had taken about a day and a half. The transition to weathered shale greatly increased the rate of construction progress, and it was observed that while boring, 10 feet of pipe was installed in 10 minutes.

- After the last section of steel casing pipe was placed, the PVC carrier pipe was installed. Casing spacers (guides) were fastened to the PVC pipe to allow the carrier pipe to “float” in the casing to protect the pipe’s joints and allow proper positioning (see Figure 4.9). The PVC pipe was then placed in the casing using a backhoe. This completed the pipeline installation. No additional problems were encountered on this project and no surface heave or settlement was observed.
Figure 4.3. Looking north at the section of State Street above the boring. The insertion pit is visible to the right.

Figure 4.4. The launching pit.

Figure 4.5. The pipe and auger boring machine in operation. The fluid supply line, water line, and steering rod are visible running along the top of the pipe.
Figure 4.6. Sandy soil being deposited out of the auger boring machine

Figure 4.7. Clayey weathered shale being deposited out of the auger boring machine

Figure 4.8. A close-up view of the weathered shale being deposited out of the auger boring machine
Figure 4.9. The 18 inch PVC carrier pipe (spacers which allow the pipe to “float” in the casing are visible)

Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. It was not possible on this project to recover soil samples.

Key Findings
This project provided a good example of the use of horizontal auger boring to install steel casing pipe. A 36 inch diameter steel casing pipe for a 24 inch PVC sanitary sewer pipe was installed at a depth ranging from 20 to 30 feet over a distance of 180 feet through weathered shale that was interrupted in the middle by a 30 feet long sand seam. It was very clear that the sand led to significant delays and the water that flowed into the launching pit from the sand threatened to cause subsidence underneath the street because of the volume reduction. The contractor responded by grouting the void, resulting in no immediate subsidence or other damage to the pavement.

Line and grade were critical to the project because the carrier pipe was going to be gravity-flow. The pipe deflected off course during the transition into and out of the sandy soil, and this was corrected by using the steering rod attached to the pipe and connected to the cutting head.
The installation was completed successfully using auger boring technology. Only minor steering problems were encountered, but the crew adjusted the line and grade to achieve the desired tolerance. The transition from a cohesive (weathered shale classified as a low plasticity clay – CL) to a non-cohesive soil (sand) demonstrated the versatility of auger boring. Additionally, the successful grouting of voids resulted in no damage to the overlying pavement being observed due to the sand and water flow. This project serves as an example of an appropriate use of auger boring technology, and demonstrates the accuracy that can be achieved.

Pipe Jacking at Keo Way and Crocker Street, Des Moines, IA.

Project Information
This project was located at the intersection of Keo Way and Crocker Street in Des Moines, Iowa during May, 2007. The pipe jacking technique was used to install a 24 foot length of 10 foot by 5 foot reinforced concrete box pipe (RCBC) under a telephone vault as part of a storm sewer project. The pipe was installed at a depth to top of pipe of about 13 feet. Precision was required, as the pipe was required to slope at 0.28% grade at an elevation that would match up with the rest of the open-cut culvert installation. The majority of the length of the box pipe installation project was performed using open-cut methods, and pipe jacking was only needed for the section of box pipe below the telephone vault.

The jacking apparatus was set up at point A and jacked northeast to point B as shown in Figure 4.10. The plan view of the project is shown in Figure 4.11.
Figure 4.10. The location of the pipe jacking project at Keo Way in Des Moines, IA. (Bore path in red)

Figure 4.11. Plan view of the project site (Bore path in yellow)

Trenchless Method Selection

Trenchless construction was selected to avoid damaging a telephone vault located above the jacked section of the pipe. Pipe jacking was the trenchless method chosen because of the
method’s flexibility in jack configuration that allows non-circular pipes to be installed. The square shape of the pipe ruled out other methods such as tunneling and auger boring.

Soil Conditions
No soil testing was conducted by the contractor before starting the pipe jacking installation, mainly due to an extensive familiarity with soil in the area. According to the contractor, this installation would be carried out through clay soil. Water infiltration was expected due to the installation depth. The soil at the site at the depth of installation was considered appropriate for auger boring construction, and problems related to soil conditions were not anticipated.

Trenchless Installation
- After several nearby existing utilities were located, the launching pit at point A was dug, and a trench box and steel plates were installed to prevent the walls of the pit from caving in (see Figure 4.12 and Figure 4.13). The area around the overlying telephone vault was stabilized with grout to prevent the development of voids that could undermine the vault. The launching pit took about two to three days to build, which was considered typical for a project of this size. The hydraulic jacking equipment was then lowered into the launch pit and assembled. Four separate jacks were set up, with the purpose of each one delivering thrust to a corner of the box pipe to provide a balanced distribution of jacking force. The jacks used the back wall of the trench box as a thrust block.

- The first 6 feet long box pipe section (see Figure 4.14) was placed into the pit using a backhoe. Connections were made to the jacking equipment and the jacking began. Progress was halted immediately, however, due to problems with the connection between the hydraulic jacks and the back of the trench box, which the jacks push against. The contractor fixed the connection by welding additional steel plates between the jacks and the trench box to stabilize the connection. The jacking then resumed. It is important to launch the tunnel correctly because, if the first section gets out of alignment, it is difficult to correct.
The jacking of the first box pipe then began (see Figure 4.15). The method employed a mini road header to excavate the soil as the pipe was jacked into the soil (see Figure 4.16). The spoil was then moved out of the pipe using a skid loader (see Figure 4.17) back to the trench box where the backhoe removed it. The contractor was hoping to install the pipe at a rate of about 4 feet per day.

Unexpected problems occurred before the first of the four pipe sections could be installed. A leak had occurred in a nearby sanitary sewer pipe that had been sealed off. This leak caused sewage to seep into the soil surrounding the launch pit, leaving two inches of raw sewage standing at the bottom of the pit. This health hazard caused a cessation of progress, and the crew was forced to leave for immunizations.

The crew later returned, and continued the installation, which was completed successfully two and a half days after the pipe jacking began. The grade specifications were met, and no additional significant problems were encountered.

Figure 4.12. Launching pit with trench box and steel plates
Figure 4.13. Trench box before installation began

Figure 4.14. 6 foot segments of 10 foot x 5 foot reinforced concrete box pipes

Figure 4.15. Jacking begins on the first box pipe section
Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. It was not possible on this project to recover soil samples due to the difficulty in accessing the launching pit bottom and due to the contamination caused by the sewer leak.
Key Findings
This project provided a good example of the use of pipe jacking to install reinforced concrete box pipe. A 10 foot x 5 foot rectangular sanitary sewer pipe was installed for a distance of 24 feet at a depth to the top of the pipe of 13 feet. The pipe jacking method was used instead of open-cut methods because of a telephone vault located above the installation depth. Trenchless construction allowed this obstacle to be avoided.

The soil was observed to be a clay, however no formal classification could be made due to the raw sewage contamination of the soil. Clay soils were considered appropriate for pipe jacking.

The installation was completed successfully using pipe jacking technology. The only significant problems encountered were due to outside circumstances unrelated to the pipe jacking method. This project serves as an example of an appropriate use of pipe jacking.

Tunneling under Highway 63, Chickasaw County, IA
Project Information
This project was located on Highway 63 (known locally as McLeod Avenue), five miles north of New Hampton in Chickasaw County, Iowa from late May through early June, 2007. The tunneling technique was used to install a 66 inch steel drainage pipe underneath the two lane highway for a distance of 44 feet and at a depth of 2.6 feet below the overlying pavement. The pipe was meant to slope downward at a 1.3% grade from the east side of the highway to the west. The new pipe was intended to replace an existing 40 inch box culvert located 10 feet to the north that was deemed “non-satisfactory”. This tunneling was part of a larger Iowa DOT HMA resurfacing with milling project.

The tunneling machine was set up at point A and bored west to point B in Figure 4.18. The research team observed the installation and interviewed crew members.
Trenchless Method Selection
Trenchless construction was selected to allow Highway 63 to stay open during the installation, as mandated by the DOT. An advantage of tunneling over other trenchless methods was the ability to install large diameter pipes and the method’s ability to allow workers access to the cutting face by climbing inside the tunneling machine.

Soil Conditions
Four experimental 4 inch diameter vertical boreholes were drilled for the project in order to characterize the soil and determine if tunneling at such a shallow depth would be likely to damage the overlying pavement (see Figure 4.19). One borehole was drilled near point A, one in each lane, and one at point B. Soil classifications resulted in the following profile. The 1.8 feet thick road and subbase was underlain by 4.2 feet of stiff black sandy clay fill. Below this lies 4 feet of stiff black silty clay, and below that lies 7 feet of firm gray brown glacial clay. A very firm dark gray glacial clay extends from a depth of 17 feet below ditch level for
an additional 14 feet, at least. The water table was established at 14 feet below the pavement. Additionally, glacial boulders were expected due to the common presence of these in the area. A much higher than expected concentration of these boulders was discovered and this would cause considerable delays in progress.

**Trenchless Installation**

After nearby existing utilities were located, the launching pit at point A was dug and a 6 inch thick bed of ¾ inch gravel was then laid in the launching pit. The track for the tunnel boring machine (TBM) was then placed on the gravel bed at the correct line and grade. The distance from the top of pipe to the pavement was approximately 2.6 feet at the closest point, which occurs at the beginning of the installation on the east side (see Figure 4.20).

The TBM had limited steering capability, so it relied heavily on the accuracy of the initial track placement (see Figure 4.21). A laser sight was used to determine if the TBM was on target. A large amount of rain occurred during the work, which prevented any progress being made during several days. Sumps were used in the launching pit to keep it dry.

- The TBM was jacked to the start of the bore and the cutter blades were spun, excavating the soil. As progress began, sloughing of the gravel fill above an existing perforated HDPE drainage pipe caused some concern (see Figure 4.22). However, it was thought that the loss of material wouldn’t propagate as far as the pavement surface, so the bore proceeded. Indeed, this did not develop into a serious problem, and new gravel was later added to replace the lost aggregate.

- Shortly after the boring commenced it was discovered that a fiber optic cable running parallel with the west side of the highway had not been properly located. After checking with the utility company, it was discovered that the cable lay closer to the road than it had been labeled, and possibly in the tunneling operation. An excavator was then used to manually locate the cable.
The TBM cut away soil, which dropped through the openings in the cutting face and onto a conveyor belt that deposited the cuttings in the launching pit. As the blade cut into the soil, the TBM was jacked into the bore to advance the excavation. Progress was greatly slowed by the large amount of glacial boulders between 12 and 18 inches in diameter that were encountered (see Figure 4.23). A much larger quantity of these boulders were found than had been expected. The reason they were found at such a high concentration and close to the surface is likely due to old highway building practices that included boulders in the fill.

The boulders were too large to be removed by the normal action of the TBM, so crew members were forced to stop the rotation of the cutter blades and climb inside the machine to manually remove the boulders. Picks, crowbars, air hammers, and jack hammers were used to dislodge the boulders at the soil face. The drill head often needed to be removed to gain access. Once dislodged, the boulders were either placed on the conveyor belt if they were small enough, or pulled out of the tunneling machine by a chain pulled by the backhoe. If very large boulders were encountered (greater than 2 or 3 feet), they were fragmented using rock blaster cartridges.

The boring proceeded slowly, with rarely more than 5 feet of progress per day. The crew switched cutter blades from dirt to rock blades soon after the large amount of boulders were discovered in the fill. This switch had little effect on actual speed of the operation, but prevented the soil blades from experiencing excessive damage. However, the crew planned to switch back to the dirt blades if the boulders subsided and clayey soil dominated, because the rock blades would clog in clayey soil.

The pipe sections were attached behind the TBM and jacked into the borehole as the TBM was advanced (see Figure 4.24). The pipe sections were each 20 feet in length and were welded together. The drill head was 14 feet long.
The boulder-sized rocks encountered on the site were thought to increase the risk of surface heave. However, no evidence of surface cracking was visible directly over the drill path. The research team took before and after construction photographs that revealed no visible change.

The bore was finished successfully after three weeks. This project took far longer than the three days that was expected, due specifically to the large amount of boulders in the soil and to the large amount of rain, which prevented work during several days.

Figure 4.19. View from point A looking west across Highway 63 toward point B. Two 4 inch diameter sample corings through the asphalt are visible. The red lathe visible in the background at the edge of the gravel shoulder marks the end of bore.
Figure 4.20. Partially flooded launching pit at point A with track installed, before beginning of tunneling

Figure 4.21. TBM before starting the bore

Figure 4.22. Existing granular backfill over a drainage pipe begins to slough away
Figure 4.23. Boulders removed from the borehole (12-18 inch diameter)

Figure 4.24. Pipe being jacked into the soil behind the TBM

Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Disturbed soil samples were recovered at a depth of 5 feet, which coincided with the depth of the installation. These samples were removed to the laboratory in sealed plastic bags. Additional soil data was gained from the owner.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.2. The 66 inch pipe was
installed between the depths of 2.6 feet and 8.1 feet. Near these depths, the owner’s soil data indicated that stiff black sandy clay was found between 1.8 feet and 6.0 feet, and stiff black silty clay was found from 6.0 feet to 10.0 foot depth. The water table was located at 14 feet.

The samples recovered by the research team confirmed that sandy clay was located at 5.0 foot depth. The gradation curve is shown in Figure 4.25.

### Table 4.2. Chickasaw County project soil parameters

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<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
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<td>-</td>
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<tr>
<td>6.0 – 10.0</td>
<td>Stiff black silty clay</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>10.0 – 17.0**</td>
<td>Firm gray-brown glacial clay</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>17.0 – 31.0</td>
<td>Very firm dark gray glacial clay</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 66 inch pipe at 2.6 foot depth
**Depth of water table at 14 feet
#From the owner’s soils report
Figure 4.25. Soil gradation curves for depth = 3 to 5 feet

Key Findings

This project provided a good example of the use of tunneling to install large diameter steel pipe. A 66 inch steel drainage pipe was installed underneath a two lane highway for a distance of 44 feet with as little as 2.6 feet of clearance under the overlying pavement. The new pipe was intended to replace an existing 40 inch box pipe located 10 feet to the north that was deemed “non-satisfactory”. This tunneling was part of a larger Iowa DOT HMA resurfacing and milling project.

A tunnel boring machine (TBM) was used to excavate the soil, while hydraulic jacks were used to advance the TBM and the pipe behind it. Problems were encountered as the crew was unable to work for several days due to large amounts of rain, and a large quantity of boulder sized rocks were encountered in the fill beneath the road. It was impossible to remove these boulders by the normal action of the TBM, so workers had to climb into the TBM and remove the boulders manually. The project demonstrated the value of having personnel access to the cutting face.
The soil through which the bore passed was fill that was classified as sandy lean clay, which is an appropriate soil type for tunneling. The glacial boulders encountered are native to northeastern Iowa, but were found in the fill due to construction practices at the time the highway was constructed.

The installation was completed successfully using tunneling technology. This project serves as an example of an appropriate use of tunneling, while also demonstrating one type of problem that may arise.

*Auger Boring at SE 6th Avenue and SE 64th Street in Des Moines, IA.*

**Project Information**

This project was located at the T-intersection of SE 6th Avenue and SE 64th Street in Des Moines, Iowa, during June, 2007. The auger boring technique was used to install a 32 inch diameter concrete casing pipe for a 24 inch storm water drainage pipe. The casing was bored for a length of 85 feet at a depth of 17 feet to the top of the casing pipe. The auger bore was set up at point A and bored northeast to point B in Figure 4.26. No plan view from the bid documents was available.

*Figure 4.26. SE 6th Avenue and SE 64th Street in Des Moines, IA. (Bore path in red)*
Trenchless Method Selection
The general contractor had first intended to cross the intersection using open-cut methods. However, complicating this was the city’s requirement that one lane be kept open at all times. Trenchless methods were then selected to overcome this difficulty. Auger boring was the trenchless method chosen because it was appropriate for the pipe size, installation depth, soil conditions, and cost. Horizontal directional drilling (HDD) was not chosen because the rigidity of concrete pipes is not ideal for HDD.

Soil Conditions
No soil testing was conducted by the contractor or the owner before starting the boring because of familiarity with soil in the area. The risk of running into unexpected soil conditions was considered relatively low, and the expense of soil borings was not considered to be justified. The digging of the launching pit at point A allowed a view of the soil profile down to the depth of bore, which revealed sandy clay toward the bottom. The presence of sand caused the clay to have less cohesion and was therefore more prone to crumbling. This was important for the design of the unbraced sides of the launching pit, which now required more gradual side slopes and, consequently, a larger footprint. It was thought that this sandy soil was a result of the site being located less than a mile from the Skunk River. These soil conditions were considered appropriate for auger boring construction, and problems related to soil conditions were not anticipated.

Trenchless Installation
- The bore had to navigate past existing utilities. The bore crossed paths with an existing 12 inch water main, 600 and 200 cable bundles, gas lines, and sanitary sewer pipe. Potholing was used to identify the exact locations of these utilities (see Figure 4.27). After the nearby existing utilities were located, the launching pit at point A was dug with an excavator. Shoring wasn’t used, and instead the sides of the launching pit were terraced (see Figure 4.28). Then, track for the boring machine was laid at the proper line
and grade in the launching pit on steel slab flooring above a 6 inch thick gravel base. The auger boring machine was then lowered into the launching pit and placed on the tracks.

- The first section of pipe (see Figure 4.29) was lowered into the launching pit and connected to the machine. The boring then began. Soil cuttings were transported by the auger back through the pipe to the launching pit. A backhoe was used to remove the cuttings to the surface. The crew checked the line and grade each time a new pipe section was welded to the pipeline. The sandy clay soil that had been at first encountered transitioned into clay with less sand. This did not negatively affect the boring procedure. Water was also encountered near the middle of the bore. A possible reason for this water was the presence of voids in the soil surrounding an old pipe.

- The bore was kept on course and the installation was completed successfully in two days (see Figure 4.30). The carrier pipe emerged at the manhole vault at point B within the accuracy tolerance. No unusual problems were encountered, and no damage to the overlying pavement was observed.

Figure 4.27. Potholing by edge of pavement
Figure 4.28. Launching pit with terraced sides and a gravel base

Figure 4.29. 24 inch concrete pipe

Figure 4.30. The launching pit at A is filled in as the installation is completed
Research Team Actions

The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Disturbed soil samples were recovered at a depth of 10 feet, which was above the depth of the installation. These samples were removed to the laboratory in sealed plastic bags. Additional soil data was gained from talking to the contractor.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.3. The 32 inch pipe was installed between the depths of 17 feet and 19.7 feet. The samples recovered by the research team confirmed that low plasticity clay was found directly above the installation, from the ground surface down to 16.0 feet. The gradation curve is shown in Figure 4.31. From 16.0 feet down to 20 feet, sandy clay was observed. The water table was located deeper than 20 feet.

Table 4.3. Des Moines SE 6th Avenue and SE 64th Street project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
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</thead>
<tbody>
<tr>
<td>0 – 2.0</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0 – 16.0</td>
<td>Sandy lean clay (CL)</td>
<td>28.7</td>
<td>27.1</td>
<td>15.0</td>
<td>12.1</td>
</tr>
<tr>
<td>16.0 – 20.0*</td>
<td>Sandy clay# *</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 32 inch pipe at 17 foot depth,
Location of water table is below this level
#Reported by the contractor
Key Findings

This project provided a good example of the use of auger boring to install concrete casing pipe. A 32 inch diameter concrete casing pipe was installed at a depth of 17 feet and over a distance of 85 feet through sandy clay soil. The casing pipe was installed to shield a 24 inch gravity flow storm water sewer pipe.

The soil down to a depth near the top of the pipe was classified as low plasticity clay. The soil at the level of the pipe was observed to be sandy clay, which is an appropriate soil type for auger boring. This soil transitioned into a clay of less sand in the middle of the bore. Also, some water flowed into the pipe from a void which may be due to erosion around an existing pipe.

The installation was completed successfully using auger boring technology. This project serves as an example of an appropriate use of auger boring technology.

Figure 4.31. Soil gradation curves for depth = 8 to 10 feet
Auger Boring at NE 22nd Street and NE 47th Street, Ankeny, IA

Project Information

This project was located near the intersection of NE 22nd Street (also known as Delaware Avenue) and NE 47th Street in Ankeny, Iowa, during June 2007. The auger boring technique was used to install a 30 inch diameter steel casing that would later house an 18 inch polyvinyl chloride (PVC) sanitary sewer pipe. The casing was bored for a length of 110 feet at a depth to top of pipe of 24.5 foot to 25.0 foot. Accuracy was important because the gravity-flow carrier pipe had to meet grade requirements.

The purpose of the project was to connect the currently under-construction Otter Creek Golf Course with the sanitary sewer system pipes in the residential area on NE 47th Street. The auger bore was set up at point A and bored west to point B in Figure 4.32. The profile view of the project is shown in Figure 4.33.

Figure 4.32. The location of the auger boring project at NE 22nd Street in Ankeny, Iowa. (Bore path in red)
Trenchless Method Selection

Trenchless construction was selected to avoid closing NE 22\textsuperscript{th} Street, and because of the large depth of installation (24 feet). Auger boring was the trenchless method chosen because it was appropriate for the pipe size, installation depth, soil conditions, and cost. Horizontal directional drilling (HDD) was not chosen because the rigidity of steel pipes is not ideal for HDD.

Soil Conditions

No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to a familiarity with soil in the area. The soil was expected to be glacial till, and these clay soils are well suited to auger boring. Some rock was also present in the soil.

Figure 4.33. Profile view of the project (Bore path in yellow)
Trenchless Installation

- After nearby existing utilities were located, the launching pit at point A was dug (see Figure 4.34). Next, steel shoring was installed (see Figure 4.35), and track for the boring machine was laid at the proper grade in the launching pit on a crushed rock base. The auger boring machine was then lowered into the launching pit and placed on the tracks.

- The first 20 feet long section of pipe was lowered into the launching pit and connected to the machine (see Figure 4.36). The boring then began. The crew checked the line and grade each time a new 20 foot pipe section was welded to the pipeline. A flashlight was placed inside the pipe, and a transit was aimed at the light to measure the bore’s position relative to the planned centerline. The crew discovered that the bore was drifting left, and so it needed to be corrected. A wing was installed on the side of the pipe, which deflected the pipe back on course, and was then removed.

- There was no clear reason why the bore was drifting left. The pipe may have encountered a rock in the bore path. Another reason is the welding that connects each 20 foot section of pipe. Welds can break, which would become a major problem, and welds can cause the pipe to lose its straightness.

- The course of the pipe was corrected, and the installation was completed successfully (see Figure 4.37). The carrier pipe emerged at the manhole vault at point B within accuracy tolerance.

- After the last section of steel casing pipe was placed, the PVC carrier pipe was installed. Casing spacers were fastened to the PVC pipe to allow it to “float” in the casing and correctly position the carrier pipe (see Figure 4.38). The PVC pipe was then placed in the casing using a backhoe. This completed the pipeline. No additional problems were encountered on this project and no surface heave or settlement was observed.
Before and after construction pictures were taken of 22\textsuperscript{nd} Street above the bore path to monitor for any damage to the pavement. A comparison of these photographs showed that the bore had made no visible effect.

Figure 4.34. Launching pit with NE 22\textsuperscript{nd} Street in the background

Figure 4.35. Launching pit with shoring and a gravel base
Figure 4.36. Launching pit before a new 20 foot pipe section is lowered into place

Figure 4.37. The casing protruding after the boring is finished
Figure 4.38. The 18 inch PVC carrier pipe with spacers visible

Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Disturbed soil samples were recovered at a depth of 9 feet, which was above the depth of the installation. These samples were removed to the laboratory in sealed plastic bags. Additional soil data was gained from talking to the contractor.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.4. The 30 inch pipe was installed between the depths of 15 feet and 27.5 feet. The samples recovered by the research team confirmed that silty sand was found directly above the installation, from the ground surface down to 18.0 feet. The gradation curve is shown in Figure 4.39. From 18.0 feet down to 28 feet, gray-black hard clay with gravel was observed. The water table was located deeper than 28 feet.
Table 4.4. Ankeny NE 22nd Street and NE 47th Street project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 1.0</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.0 – 18.0</td>
<td>Silty sand (SM)</td>
<td>28.9</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>18.0 – 28.0*</td>
<td>Gray-black hard clay with gravel#</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 30 inch pipe at 24 foot depth, Location of water table is below this level
#Reported by the contractor

Figure 4.39. Soil gradation curves for depth = 7 to 9 feet

Key Findings
This project provided a good example of the use of horizontal auger boring to install steel casing pipe. A 30 inch diameter steel casing pipe was installed at a depth of 24.5 to 25 feet and for a distance of 110 feet. The casing pipe was installed to shield an 18 inch PVC sanitary sewer pipe.
Line and grade were critical to the project because the sewer pipe needed to be gravity flow. Also, the casing needed to connect with a manhole vault at the termination of the bore. The pipe veered to the left at one point during the installation, but the crew corrected the problem and was able to complete the bore meeting accuracy specifications.

The soil was classified as glacial till, which is an appropriate soil type for auger boring. The rock in this soil may have contributed to the deflection experienced during the boring.

The installation was completed successfully using auger boring technology. This project serves as an example of an appropriate use of auger boring technology and demonstrates the accuracy that can be achieved.

*Horizontal Directional Drilling at Johnny Majors Practice Field, Iowa State University, Ames, IA.*

**Project Information**

Renovations to Jack Trice football stadium at Iowa State University included installing over 1060 ft of electrical conduit under the adjacent outdoor practice field and around part of Jack Trice Stadium itself. Horizontal directional drilling (HDD) was selected to complete this installation which occurred in June, 2007

The electrical conduits were installed in two stages. The first stage was a deep bore to install two side by side 4 inch diameter HDPE pipes for the 560 ft run crossing the Johnny Majors Practice Field from A to D and then D to E (see Figure 4.40). This bore installed the pipes in an 18 inch diameter borehole at a depth of up to 17 feet below ground. The second stage of the project was a shallow bore installing a single 2 inch diameter HDPE pipe in several shorter runs totaling 500 feet around the outside of the stadium. This bore ran between F and G, G and H, and H and I. This smaller bore created a four inch diameter borehole at a depth of only three feet.
Significant problems were encountered during the bore from A to D, which led to high drilling fluid pressures causing fractures in the subsoil. As this released the built up fluid pressures, the excess drilling fluid migrated toward the surface resulting in the appearance of drilling fluid on the ground surface, called “inadvertent returns”, or “frac-out”. Additionally, this mechanism caused soil at the surface to be displaced vertically, resulting in a bulging of the ground called “surface heave”. The shallower bores were completed without incident.

Figure 4.40. The location of the HDD at Johnny Majors Practice Field in Ames, IA. (Bore path in red)

Trenchless Method Selection
Trenchless construction was selected by the owner to install electrical conduit beneath Johnny Majors practice field while allowing it to stay open, which was deemed important because football practice was ongoing. Additionally, trenchless installation allowed a much deeper installation (17 feet) and fast completion. The planned borings can be seen as part of the larger electrical project in Figure 4.41.
Soil Conditions
Although the project site is known to be located on a flood plain and up to 7 to 8 feet of fill was placed on top of original ground level before the construction of the practice field, the owner and the contractor did not perform any geotechnical investigation before the boring. The owner mainly relied on the experience of the contractor to judge the soil properties from potholing and drilling returns and to adjust the construction technique if any problems were encountered.

Trenchless Installation
- The contractor decided to approach the boring from point A near Jack Trice Stadium and connect to point E next to Beach Avenue by performing two separate bores. The contractor began work by setting up their HDD drill rig at point A, in preparation to bore west toward point D. The first run would be 400 feet underneath the practice field as far
as point D at the toe of the hill that led up to Beach Avenue. The contractor’s plan was to bore the remaining 160 feet from D to E afterward. Preparations also included digging an exit pit at point D. This pit would allow the recovery of boring equipment at the proper depth. In the spots where the bore path was intended to cross near existing utilities, potholing was used to visually confirm the existing pipe’s location.

- A drilling fluid high in sodium bentonite was used by the contractor to reduce the risk of borehole collapse if sandy soil was encountered during drilling. Sandy soil was considered by the contractor to be of higher risk than clay, so the selection of drilling fluid materials was focused toward sand. The product used was TRU-BORE™ sodium bentonite. A clay inhibiting polymer was also used in the mix to lubricate and stabilize the bore.

- At a distance of 10 feet from the start of bore, the planned route crosses an existing electrical conduit that has a diameter of 4.5 inches and lies at a depth of 5 feet. To avoid affecting the existing pipe, it was necessary to pothole to the old pipe to verify its true depth and observe the position of the bore as it passes by. When the hole reached the expected depth of the existing pipe, the backhoe excavated 2 inches deeper at a time and then a hand probe is used to ascertain if the pipe is located in the next 2 inches. Clay described as “blue/black” was found at the location where the two pipes intersect. It was verified that the top of the existing pipe is 57 inches below grade at the spot. This depth was acceptable as it meant that a clearance of greater than 1 foot would exist between existing pipe and the new conduit above.

- The pilot bore began by attaching a 4 inch drill bit to the directional drilling machine (see Figure 4.42) and then using the machine to push the drill bit into the ground. The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The pilot bore was conducted successfully and the drill bit emerged in the exit pit at point D during the first day of boring.
The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 12 inch reamer for the prereaming stage (see Figure 4.43). The directional drilling machine was switched from push mode to pull mode, and the drill string with the reamer attached was then pulled back through the pilot bore toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance and enlarged the hole from 4 inches to 12 inches in diameter. Progress was extremely slow, however, as only about 200 feet of borehole had been enlarged during the first 12 hours of prereaming.

The contractor identified that a clay soil high in gravel was being encountered during the pullback. This was a partial cause of the extremely slow progress. After what the contractor estimated to be 60 feet of gravel, the soil switched to a gray clay that is common in the local area. This clay is often very stiff and can cause slow, difficult bores.

About 6 gallons of drilling fluid per minute were pumped through the hollow drill rods and out of small perforations in the reamer. Due to the extremely slow speed of pullback, this had resulted in 200 gallons having been pumped for each 6.5 feet of drill rod. During this process, the excess drilling fluid should emerge at either the entry or exit pit. However, not as much liquid as expected was observed to be exiting the bore. This suggested the possibility of frac-out occurring, and indeed towards the end of the second day of 12 inch prereaming, frac-out was observed on the practice field surface 20 feet away from the bore path. This occurred while boring through the gravelly clay soil. The prereaming process was continued and several hours later, surface heave occurred directly over a portion of the bore path (see Figure 4.44). This occurred about 10 feet after switching from gravelly soil to clay.

The cause of the initial frac-out probably lies in the presence of gravel and stiff clay behind the area in addition to the bentonite drilling fluid. A possible explanation is that the gravel caved in behind the bore, and with the addition of the stiff clayey soil cuttings served to clog the previously enlarged hole and prevent the evacuation of drilling fluid to
the exit pit. The reamer itself was also becoming clogged with clay solids, preventing the drilling fluid from flowing through the reamer and out through the smaller pilot borehole toward the entry pit. This led to fluid pressures rising and the soil eventually fracturing along planes of greatest weakness. This may have resulted in the drilling fluid flowing out of the borehole in a direction roughly perpendicular to the bore path and finally reaching the surface 20 feet away in a visible frac-out. The appearance of these inadvertent returns can be seen in Figure 4.45, which shows the edge of the frac-out. This hypothesis is supported by the later subsurface soil investigation in the area where the frac-out occurred, in which sandy soil was observed around that area. Additional damage was done to subsurface water main pipes at a depth of 10 feet that were part of the sprinkler system.

- The surface heave that followed the frac-out several hours later is thought to have resulted from roughly the same mechanism. At the location of the surface heave, the subsurface investigation showed that stiff clay exists between the ground surface and the location of the HDD construction work. Although the reamer had, by this time, passed the subsurface gravel, stiff clays continued to keep progress slow, allowing fluid pressures in the borehole to reach high enough levels to finally rupture the sides of the borehole. The drilling fluid had no place to escape except by displacing soil upward. The heave mechanism seems to have released enough pressure that a frac-out of drilling fluid was not observed at the surface. Unfortunately, the HDD method lacks a way to directly observe what is happening below the surface. This makes it impossible for us to know exactly what occurred underground.

- The 12 inch diameter prereaming process was completed at the end of the third day of prereaming. Before its completion, late in that third day an additional episode of frac-out occurred; this time in the area of the earlier surface heave. This new frac-out event probably occurred due to the continued high drilling fluid pressures inside the borehole resulting from the borehole and the reamer becoming clogged and no longer allowing drainage at the exit and launching pits. The occurrence of this frac-out at the same
location as the earlier heave suggests that the fluid followed planes of weakness created when the initial heave occurred, and probably represents a continuation of that process. The location of these events is shown again in Figure 4.46.

- After the contractor finished pulling the 12 inch diameter reamer back from the exit pit to the entry pit, the 12 inch reamer was removed. A more powerful drill rig had been rented to increase the speed of the bore and to allow the pullback of an 18 inch diameter reamer with the product pipe attached during the same pull. The 2 inch diameter drill string was first easily pushed back through the borehole, back to the exit pit. There, an 18 inch diameter reamer was attached to the drill string so it could be pulled back through the borehole to increase the diameter (see Figure 4.47). Also connected to the back of the 18 inch reamer were the two 4 inch HDPE conduits. The two conduits pulled simultaneously side by side created an effective conduit outside diameter of 10 inches, which in an 18 inch borehole falls within the HDD rule-of-thumb for borehole to pipe diameter ratio of between 1.5 and 2 to 1.

- The pullback of both reamer and HDPE pipe commenced but after about 100 feet of progress, the pullback forces became excessive and the “weak link” device that is used to protect the pipe from high tensile forces failed and the pullback of the 18 inch diameter reamer from had to be completed without the pipe. The reamer was then removed at the entry pit and the empty drill string was pushed back through the borehole where a second attempt would be made to pullback the reamer and pipe.

- The 18 inch reamer was then reattached to the drill string along with the product pipe. Although having already prereamed once with the 18 inch reamer, reattaching for a second pull-back was necessary to ensure that the borehole would be clear of debris that may have caved in after the last reamer pass. This pullback and installation was completed smoothly as the already enlarged borehole required less pullback force (see Figure 4.48).
• The new drill rig possessed a gauge monitoring pullback forces, and it was observed that the successful pullback of the pipe and reamer through the already enlarged borehole required up to 3,000 lbs, while the previous pullback of the 18 inch reamer through the 12 inch hole required only 500 lb. This indicates that the pullback friction of the pipe against the edges of the borehole significantly increased required pullback force. The 18 inch reamer pullback and the following 18 inch ream with product pipe pullback took a combined 3 days to complete.

• The project then required continuing the two 4 inch electrical conduit installation past the initial exit pit at point D and up a wooded hill for a distance of 160 feet to point E near Beach Avenue. The contractor used a second new drill rig at the top of a hill at E, and pushed the same 4 inch diameter pilot bit down to the exit pit at D used for the earlier practice field crossing. A roughly 6 foot depth was maintained during the bore, and the soil cuttings returned in the drilling fluid confirmed the soil to be high in clay.

• A different mix for the drilling fluid was used for this second bore in light of the difficulties encountered during the practice field drilling. This time, a polymer based drilling fluid with a clay inhibitor additive and less bentonite was used. The pilot bore was quickly and smoothly accomplished, and the effectiveness of the drilling fluid mix encouraged the contractor to back ream a 12 inch diameter reamer while attached to the two 4 inch HDPE pipes attached. Although this created a 12 inch hole for the 10 diameter product, which violated the 1.5 to 1 rule of thumb, it was considered adequate given the ease with which the pilot bore had been accomplished and the relatively short distance (160 feet). This operation was completed successfully and without incident, completing the 160 foot uphill installation in one day (see Figure 4.49).

• The contractor’s drill rig from the bore from A to D was used to complete the additional 500 feet of boring around the outside of Jack Trice Stadium. These bores installed a single 2 inch diameter HDPE electrical conduit by repositioning the drill rig three times to make four separate runs. These bores ran from F to G, G to H, H to I, and I to J.
These series of bores were significantly shallower than the practice field bores, with depths of only 3 feet. Installation was completed by first pushing the 4 inch diameter pilot bore into the soil to a shallow exit pit in a procedure similar to the deeper practice field crossing. A 4 inch reamer was then attached in the exit pit along with the 2 inch HDPE electrical conduit. The reamer and pipe were then pulled back from the exit pit to the entry pit. The installations were completed successfully, and resulted in sections of installed pipe that could be spliced together.

Figure 4.42. Directional drilling machine beginning the pilot bore from point A to D

Figure 4.43. 12 inch diameter reamer
Figure 4.44. Surface heave is observed in the background, and frac-out is observed in the foreground. These occurred during the prereaming stage.

Figure 4.45. The edge of the region of frac-out, demonstrating the presence of drilling fluid at the ground surface.

Figure 4.46. The second occurrence of frac-out is observed near the location of the earlier surface heave.
Figure 4.47. The 18 inch reamer is shown jetting drilling fluid in the exit pit at point D just before pullback.

Figure 4.48. The installation of the two 4 inch product pipes as they are pulled into the borehole at point D, while connected behind the 18 inch reamer.

Figure 4.49. The two 4 inch HDPE pipes capped after installation at point E, next to Beach Street.
Research Team Actions
The research team completed an elevation survey to ascertain the severity of the surface heave. It was found that the elevation of the ground surface had been vertically heaved 9 inches at the highest point.

The research team drilled 5 test boreholes near the areas of frac-out and surface heave to better understand the soil types in these areas, and determine the reasons for the two different responses to the escape of drilling fluid from the borehole (see Figure 4.50). Disturbed samples were taken at various depths, and 2 inch and 3 inch diameter thin-walled Shelby tube were recovered. The soil samples from the Shelby tube and the disturbed bag samples were tested in the laboratory.

Figure 4.50. Drill rig used by the research team to recover soil samples

The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they related to the HDD process.

Soil Characterization
Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.5 for soil from point B,
and Table 4.6 for soil from point C. The gradation curves for point B are shown in Figure 4.51, and the curves for point C are shown in Table 4.6.

### Table 4.5. Ames Johnny Majors Practice Field project soil parameters at point B

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.5</td>
<td>Poorly graded sand (SP)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.5 – 10.0</td>
<td>Sandy clay (CL)</td>
<td>14.8</td>
<td>27.8</td>
<td>14.9</td>
<td>12.9</td>
<td>-</td>
</tr>
<tr>
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<td>Low plasticity clay (CL)</td>
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<td>13.9</td>
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<td>7019</td>
</tr>
<tr>
<td>16.0</td>
<td>Gravel layer</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>16.0 – 18.0*</td>
<td>Clayey sand (SC)</td>
<td>25.7</td>
<td>27.2</td>
<td>13.7</td>
<td>13.5</td>
<td>-</td>
</tr>
<tr>
<td>18.0 – 19.0</td>
<td>Stiff clayey sand (SC)</td>
<td>17.9</td>
<td>28.0</td>
<td>14.4</td>
<td>13.6</td>
<td>-</td>
</tr>
</tbody>
</table>

* Top of the 4 inch pipes at 17 foot depth
* Depth of water table at 14 feet
Figure 4.51. Soil gradation curves for point B

Table 4.6. Ames Johnny Majors Practice Field project soil parameters at point C

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.5</td>
<td>Poorly graded sand (SP)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.5 – 7.0</td>
<td>Sandy clay (CL)</td>
<td>16.1</td>
<td>26.1</td>
<td>15.6</td>
<td>10.5</td>
<td>2300</td>
</tr>
<tr>
<td>7.0 – 15.0*</td>
<td>Silty Sand (SM)</td>
<td>20.8</td>
<td>16.5</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
</tr>
<tr>
<td>15.0 – 19.0*</td>
<td>Silty Sand with gravel (SM)</td>
<td>19.7</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
</tr>
</tbody>
</table>

* Top of the 4 inch pipe at 17 foot depth
*# Depth of water table at 14 feet
Relatively undisturbed soil from the 3 inch diameter Shelby tubes was used for conducting unconfined compressive strength, consolidation, and consolidated-undrained multistage triaxial tests for the soil at the depth of the HDD installation. The measured unconfined compressive strength of the soil at point B was 7019 psf, and the unconfined compressive strength of the soil at point C was 2300 psf.

The consolidation test for a sample from point B at a depth of 11.5 to 12.1 feet revealed an overconsolidation ratio of 0.76, a compression index equal to 0.17, and a preconsolidation pressure of 0.455 ton/ft² (see Figure 4.53). The average coefficient of consolidation is 0.38.
Figure 4.53. Consolidation test graph showing vertical effective stress vs. void ratio for soil from 11.5 to 12.1 foot depth at point B

The consolidation test for a sample from point C at a depth of 6.1 to 6.7 feet revealed an overconsolidation ratio of 2.48, a compression index equal to 0.11, and a preconsolidation pressure of 0.86 ton/ft² (see Figure 4.54). The average coefficient of consolidation is 0.46.
Figure 4.54. Consolidation test graph showing vertical effective stress vs. void ratio for soil from 6.1 to 6.7 foot depth at point C

A multi-stage consolidated undrained test was conducted at on the soil at point B using confining pressures of, about 4, 10, and 14 psi, which represent initial lateral earth pressure and the range of expected lateral earth pressures during HDD installation. The stress-strain curves resulting from the test are provided in Figure 4.55. These results indicate a friction angle of 32.0° and a cohesion of 2.8 psi. The test picked up very small pore water pressure readings, so the effective friction angle was calculated to be 32.0° and the effective cohesion was calculated to be 2.8 psi. The initial modulus of the soil was 70, 240, and 240 tsf at confining pressures of 3.6, 8.9, and 14.0 psi, respectively.
Results found by comparing the $q'$ vs. $p'$ relationship for the point B soil is given in Figure 4.56. The parameter $\alpha$ was found to equal 33.1° and the intercept “a” was found to equal zero.
Figure 4.56. Multi-stage consolidated undrained test $q'$ vs. $p'$ graph for soil at point B

A similar multi-stage consolidated undrained test was conducted at on the soil at point C using confining pressures of 2, 4, 10, and 18 psi, which represent initial lateral earth pressure and the range of expected lateral earth pressures during HDD installation. The stress-strain curves resulting from the test are provided in Figure 4.57.
Figure 4.57. Multi-stage consolidated undrained test stress vs. strain graph for soil at point C

Results found by comparing the q’ vs. p’ relationship for the point C soil is given in Figure 4.58. The parameter α was found to equal 33.3° and the intercept “a” was found to equal 0.5 psi.
Key Findings

The several horizontal direction drilling bores used to install HDPE water pipe provided examples of the use of HDD. During the first bore, the contractor installed two 4 inch diameter pipes in the same 18 inch diameter borehole at a depth of about 17 feet, but encountered difficulties that led to slow progress, and later frac-out and surface heave.

This frac-out resulted from a pressure increase inside the bore caused by the continued pumping of drilling fluid despite extremely slow progress. A possible explanation for this is that gravel collapsed into the borehole and sealed it off in the direction of the exit pit. This, combined with the clay cuttings clogging the reamer, effectively sealed off the fluid’s path to the entry pit. The ensuing pressure build up eventually resulted in the frac-out surface appearance of drilling mud at a location 40 feet from the ensuing surface heave and 20 feet from the nearest point on the bore path. Similar circumstances soon led to surface heave occurring 35 feet further along the bore path. The probable reason for heave occurring
instead of frac-out in the second instance is the presence of a more impermeable soil. Despite this, the veins of weakness created by the heave eventually gave way to another frac-out event after continued pumping of drilling fluid was combined with slow drilling progress.

It is thought that these problems would not have occurred had a polymer drilling fluid that emulsifies clay been used instead of the sodium bentonite slurry. Instead, the clay remained sticky and coated the pilot bore drill bit and later the reamer. The clay-caked pilot bore drill bit can be seen in Figure 4.59. However, the reason the contractor used bentonite was because they were concerned about possibly encountering sand. A polymer slurry would not have been as effective as bentonite at stabilizing the borehole, and a collapse could have resulted. Bentonite was considered to be a lower-risk option.

![Figure 4.59. 4 inch drill bit caked with clay, shown after emerging into the exit pit at point D](image)

Despite some problems, the installation was completed using horizontal directional drilling. This project serves as an example of the applications of horizontal directional drilling, but it also demonstrated problems that can occur.
Auger Boring at Grand Avenue near 62\textsuperscript{nd} Street, Des Moines, IA

Project Information

This project was located at Grand Avenue, near its intersection with 62\textsuperscript{nd} Street in Des Moines, Iowa, during late July 2007. The auger boring technique was used to install a 16 inch diameter steel casing that would later accommodate a water main. The casing was bored for a length of 140 feet at a depth to top of pipe of 12 feet under Grand Avenue. The bore also passed under Iowa Interstate Railroad tracks.

The purpose of the project was to connect a newly constructed retail store to the city’s water distribution network. The auger bore was set up at point A and bored northeast to point B in Figure 4.60. No plan view was available from the bid documents.

![Figure 4.60. The location of the auger boring project at Grand Avenue in Des Moines, IA. (Bore path in red)](image)

Trenchless Method Selection

Trenchless construction was selected to avoid closing Grand Avenue and Iowa Interstate Railroad tracks (see Figure 4.61). This gave trenchless construction a tremendous social cost...
savings over trenching. Auger boring was the trenchless method chosen because it was appropriate for the pipe size, installation depth, soil conditions, and cost. Horizontal directional drilling (HDD) was not chosen because the rigidity of steel pipes is not ideal for HDD.

**Soil Conditions**

No soil testing was conducted by the contractor or the owner before starting the project. Instead, the contractor consulted other contractors that had worked in the area to get an idea of the soil to be expected. Cohesive soil was expected, and that proved to be the soil type present for the entire bore.

**Trenchless Installation**

- After nearby existing utilities were located, potholing was used to confirm the position of utilities considered potentially dangerously close to the bore path (see Figure 4.62). Next the launching pit at point A and the receiving pit at point D were dug and steel shoring was installed. Track for the boring machine was laid at the proper line and grade in the launching pit on a crushed rock base, and the auger boring machine was lowered into the launching pit and placed on the tracks.

- The first 20 foot section of pipe was lowered into the launching pit and connected to the machine (see Figure 4.63). The boring began. The crew checked the line and grade each time a new 20 foot pipe section was welded to the pipeline.

- The bore proceeded quickly (see Figure 4.64), taking only two days to complete the boring. Part of the reason for the quick boring was optimal soil conditions as clay soil is much faster and easier to bore through than is sand. Another factor in the speed of installation was the relatively small size of the 16 inch steel pipe, compared to larger casings that are often auger bored. The 20 feet long pipe sections must be welded together, and the time spent welding is directly proportional to the circumference of the pipe. In this case, the 16 inch steel pipe required about 30 minutes to weld each section.
• Heave was not expected to be a concern unless the casing was pushed too fast and compressed the soil in front of it faster than it could be augered out. Also, heave would not be generally expected because of the adequate depth (12 feet).

• The bore emerged at the receiving pit at point B after two days of boring. The installed pipe was met accuracy specifications. Also, no damage to the overlying pavement or railroad track was observed. The product pipe was then placed in the casing and final connections were made.

Figure 4.61. View of the project looking from the launching pit at point A across Grand Avenue and the railroad tracks to the receiving pit at point B

Figure 4.62. A potholing pit covered and marked
Figure 4.63. A backhoe lowers a new 20 foot section of steel pipe into the launching pit

Figure 4.64. A close-up view of the pipe and the auger boring machine as a new pipe segment is being fitted to the machine and welded to the pipeline

Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. It was not possible on this project to recover soil samples.
Key Findings

This project provided a good example of the use of horizontal auger boring to install steel casing pipe. A 16 inch diameter steel casing pipe was installed at a depth of 12 feet under an overlying street and railroad track. The bore was over a distance of 140 feet. The casing pipe was installed to shield a new water main connecting a newly built retail store with the city’s water distribution network.

The soil was observed to be a clay, which is ideal for auger boring. The soil conditions were the primary reason that the boring was completed in only two days. The finished bore met accuracy specifications and no damage was observed to the overlying pavement and railroad tracks.

The installation was completed successfully using auger boring technology. This project serves as an example of an appropriate use of auger boring technology.

Auger Boring at Osborn Drive, Iowa State University, Ames, IA.

Project Information

This project was located along Osborn Drive, on the campus of Iowa State University from late July through early August 2007. The auger boring technique was used to install two 24 inch diameter steel pipe casings for ductile iron pressurized waterlines. The casings were necessary to protect the joints in the ductile iron pipe. The two casings were each bored for a length of 80 feet at a depth of 10 feet to the top of the casing pipe. The pipes run parallel to each other with a center-to-center distance of 5 feet and are parallel to Osborne Drive. The auger bore was set up at point A and bored east to point B in Figure 4.65. No plan view was available from the bid documents because the work was part of a change order.
Trenchless Method Selection
Trenchless construction was selected to avoid damaging an existing electrical vault and to save historical trees located in the bore path. Additionally, landscaping and sidewalks were allowed to remain intact despite the installation. Auger boring was the trenchless method chosen because the steel casings were larger than ideal for horizontal directional drilling (HDD), and because auger boring was found to be economical.

Soil Conditions
No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to an extensive familiarity with soil in the area. According to the contractor, this soil included a 20 foot layer of clay that is underlain by a deep stiff clay layer. Dewatering was not necessary as the boring was performed above the water table and no sand seams, boulders, or pieces of debris were expected. Such soil conditions are considered appropriate for auger boring and problems related to soil conditions were not anticipated.
Trenchless Installation

- After nearby existing utilities were located, the launching pit at point A and the receiving pit at point B were dug. Next, steel shoring was installed and track for the boring machine was laid at the proper line and grade in the launching pit on a crushed rock base (see Figure 4.66). The auger boring machine was then lowered into the launching pit and placed on the tracks.

- The auger boring machine had limited steering capability with no steering rod attached to the top of the pipe. Additionally, no waterline system was used to measure grade. Also, no fluid supply line pipe was attached to the outside of the casing, which would have allowed drilling fluid or water to be delivered to the cutting face. However, the limited steering capability was deemed acceptable because a pilot bore was used. This involved jacking a 4 inch diameter pilot bore through the expected bore path until it reached the receiving pit (see Figure 4.67). The pilot bore drill string and the thin borehole that it created was then used to guide the auger boring of the larger casing pipe. Also, the specifications for line and grade were flexible because the product pipe was a pressure main, and therefore didn’t rely on gravity for flow.

- Because local soil conditions were expected to be uniform lean clay, a fluid supply line to the cutting face was not considered necessary. Issues with soil conditions did arise, however, as the soil was discovered to be very dry and very hard, due partially to a lack of rain. The hardness of the soil caused the fins to break off an auger, and necessitate replacement of the auger piece. This delayed boring.

- Four connected 20 foot sections of steel pipe were jacked into the soil to advance the cutting head over the 80 foot total distance. The cutting head excavated a 28 inch diameter hole into which the 24 inch diameter casing was jacked. The annular space lowered friction on the pipe and decreased jacking resistance. As each new pipe section was placed in the launching pit, it was aligned and welded to the previous piece (see Figure 4.68). Soil cuttings were transported by the auger back through the pipe to the
launching pit. A backhoe was used to remove the cuttings to the surface. After first casing pipe emerged into the receiving pit, the track was moved to the other side of the launching pit where an identical pilot bore was created and the second casing was jacked through the soil using the same process as for the first.

- After the last section of steel casing pipe was placed, the ductile iron carrier pipe was installed. Casing spacers were fastened to the ductile iron pipe to allow it to “float” in the casing to protect the joints in the ductile iron pipe. The ductile iron pipe was then placed in the casing using a backhoe, as shown in Figure 4.69. The two installed casings with the first ductile iron pipe installed is shown in Figure 4.70. A link seal was attached to the open end. The final pipe section was then lowered into the driving pit and welded to the augered pipe sections.

- The project was then completed by connecting the new pipes to two existing capped pipes that had been installed one year earlier. The final connections are shown in Figure 4.71. This completed the pipeline. No additional problems were encountered on this project and no surface heave or settlement was observed.

Figure 4.66. Launching pit with shoring and a gravel base
Figure 4.67. First pilot bore emerging in the receiving pit

Figure 4.68. First casing being bored from the launching pit
Figure 4.69. The first ductile iron water pipe with casing spacers attached is placed by backhoe into the steel casing.

Figure 4.70. Both casing pipes after boring are seen from the launching pit, and the carrier pipe can be seen in the casing pipe on the left.

Figure 4.71. The final connection of pipe sections in the launching pit.
Research Team Actions

The research team recovered disturbed samples were taken from the middle of the thick, uniform clayey layer. The samples were removed to the lab for testing.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.7. The gradation curves are shown in Figure 4.72. The soil profile between the ground surface and the pipe locations consists of two soil layers. The first layer, between the ground surface and 1.5 foot depth, consists of clay with organics topsoil. The second layer, from 1.5 feet to at least 10 feet was found to be sandy lean clay. This second layer, in which the pipe was installed, has an average moisture content of 9.8%, a liquid limit of 19.1%, and a plasticity index of 5.7%. The water table was located below the depth of installation. This classification matches the soil description that the contractor expected.

Table 4.7. Ames Osborn Drive auger boring project soil parameters.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.5</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.5 – 10*</td>
<td>Sandy Lean Clay (CL)</td>
<td>9.8</td>
<td>19.1</td>
<td>13.4</td>
<td>5.7</td>
</tr>
</tbody>
</table>

*Top of the 24 inch pipe at 10 foot depth, Location of water table is below this level,
Key Findings

This project provided a good example of the use of horizontal auger boring to install steel casing pipe. Two 24 inch diameter steel casing pipes were installed in parallel at a depth of 10 feet and over a distance of 80 feet. The casing pipes were installed to shield ductile iron water pipes.

Two 4 inch diameter pilot bores were first jacked into place to create an initial borehole that the auger bore could follow. Both auger borings were completed successfully, however the fins of one auger were broken off due to excessively hard soil.

The soil was classified as sandy lean clay, which is an appropriate soil type for auger boring, although the lack of rain recently may have contributed to the soil hardening and causing equipment damage.
The installation was completed successfully using auger boring technology. This project serves as an example of an appropriate use of auger boring technology, while also demonstrating one type of problem that may arise.

*First Horizontal Directional Drilling at Osborn Drive, Iowa State University, Ames, IA.*

**Project Information**

This project was located along Osborn Drive, on the campus of Iowa State University during August, 2007. The horizontal directional drilling technique was used to install an 8 inch diameter HDPE chilled water pipe. The installed pipe has a length of 330 feet, and was bored at a depth to the top of the pipe, which varied between 6 and 9 feet.

The pipe was installed by completing a single bore, as shown in Figure 4.73. The boring runs east-west, parallel to Osborn Drive. The research team observed the installation and interviewed crew members. The plan view of the project is shown in Figure 4.74.

![Figure 4.73. The location of the HDD along Osborn Drive in Ames, IA. (Bore path in red)](image-url)
Trenchless Method Selection
Trenchless construction was selected by the owner to avoid damaging existing trees that had a protected status. Additionally, sidewalks and driveways in the bore path were not only saved from destruction, which would have been caused open-cut pipe installation, but also were allowed to remain in operation during the construction.

Soil Conditions
No soil testing was conducted by the contractor or the owner before starting the HDD installation. Soil testing was not considered necessary mainly due to an extensive familiarity with soil in the area. The soil was expected to be a relatively homogenous, low plasticity clay. The contractor considered these clays to be excellent for construction.

Trenchless Installation
- First, all nearby utilities were located and marked. Potholing was done in several places to confirm the locations of existing utilities that were considered dangerously close to the
bore. Then, an exit pit was excavated at point B where the bore was to end 330 feet from the drill rig. This pit would be used to retrieve drilling tools, pull back reamers, and insert product pipe. Additionally, the exit pit would be used to evacuate drilling fluid with soil cuttings from the borehole. The HDD drill rig was then set up at the entry point at point A.

- A drilling fluid consisting of BOREGEL™ mix and water was used during the boring. This mix contains sodium bentonite, a clay-inhibiting polymer, and soda ash. This product is advertised to improve borehole stability in sandy soils. The fluid was mixed in a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 4 inch drill bit to the directional drilling machine and then pushing it into the ground (see Figure 4.75). The pilot bore depth was 6 feet below the surface except when an existing underground steam tunnel had to be avoided. The bore was steered down to 9 feet to avoid this obstacle, and then steered back up to 6 feet after it was past. The drilling proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The pilot bore proceeded without any difficulties, and the drill bit emerged in the exit pit at point B.

- The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 14 inch reamer (see Figure 4.76) for the prereaming stage. The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 4 inches to 14 inches in diameter. The reamer cut away soil as it spun, and injected drilling fluid into the borehole from perforations in the reamer. Periodically, the pull back was stopped and the reamer was pushed toward the exit pit in order to remove cuttings. After several of these pauses, the reamer was successfully pulled through the length of the borehole and emerged at point A.
The drill string was then pushed back to the exit pit at B with the 14 inch reamer still attached. The 8 inch HDPE pipe (see Figure 4.77) was then attached to the reamer, and the reamer and pipe were pulled back from B to A. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 14 inches. The pipe was installed successfully and without any problems. A pit was then dug at point A to allow better access to the end of the pipe. The pipe was then capped until it later would be connected to the rest of the pipeline. The installation was completed in four days, two of which were spent on the actual boring.

Figure 4.75. Directional drilling machine

Figure 4.76. 14 inch diameter reamer
Research Team Actions

The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Disturbed soil samples were recovered at a depth of 6 feet, which was near the depth of the installation. These samples were removed to the laboratory in sealed plastic bags. Additional soil data was gained from talking to the contractor.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.8. The 8 inch pipe was installed between the depths of 6 feet and 9 feet. The samples recovered by the research team confirmed that sandy lean clay was found at and above the installation, down to at least 9 feet. The gradation curve is shown in Figure 4.78. The water table was located deeper than 9 feet.
Table 4.8. Ames Osborn Drive first HDD project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Strength (psf)</th>
</tr>
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<td>-</td>
</tr>
<tr>
<td>1.5 – 9*</td>
<td>Sandy Lean Clay (CL)</td>
<td>14.8</td>
<td>27.8</td>
<td>14.9</td>
<td>12.9</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 8 inch pipe at 6 foot depth,
Location of water table is below this level

Figure 4.78. Soil gradation curve for depth = 3 to 5 feet

Key Findings
This project involved the installation by horizontal directional drilling of a single 330 feet long, 8 inch diameter HDPE chilled water pipe. The depth to the top of pipe varied from 6 to 9 feet.

The pipe was installed by first drilling a 4 inch pilot bore, followed by prereaming using a 14 inch reamer, before finally pulling the 14 inch reamer attached to the product pipe through the hole.
The soil was tested in the lab and classified as sandy lean clay, which is common to the area. This soil is considered an appropriate soil type for horizontal directional drilling.

This bore was successful in avoiding heave and settlement of the ground surface and overlying pavement for several reasons. First, the contractor drilled at a moderate speed, which prevented outrunning the drilling fluid, which can happen if higher speed is used. Secondly, the contractor considered the drill rig used to have ample power to handle the pullback forces of this bore. Thirdly, the correct selection of a drilling fluid is very significant. The contractor’s experience is very important in having a successful HDD operation.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling.

**Auger Boring under railroad tracks near M Avenue in Tama, IA.**

**Project Information**

This project was located at Chicago & North Western Railroad tracks near M Avenue in Tama, Iowa during late August, 2007. The auger boring technique was used to install three parallel 60 inch diameter steel drainage pipes under the railroad track. These pipes were each bored for a length of 80 feet at a depth of 5 feet from the track to the top of the casing pipes. The auger bore was set up at point A and bored northeast to point B in Figure 4.79 for each of three bores. No plan view from the bid documents was available.
Trenchless Method Selection
Trenchless installation was chosen to allow the installation of the drainage pipes without having to close or detour the railroad traffic. Auger boring was the trenchless method chosen because of the large size of pipes, and because auger boring was found to be economical.

Soil Conditions
The soil to be bored through was backfill underneath the railroad tracks. It was observed to be dark, gravelly organic soil. No additional soil testing was carried out as it was considered unnecessary.

Trenchless Installation
- Several existing utilities lay in the vicinity of the bore. Water jetting was used to make the potholes that were used to identify the exact positions of these pipes. Then, track for the boring machine was laid on a gravel base at the proper line and grade in the launching pit at point A. The auger boring machine was then moved into the launching pit and placed on its track (see Figure 4.80).
The initial placement of the tracks was very important, because the bore possessed no steering mechanism. The accuracy of the bore was important, because of the potential damage to the overhead train tracks, and because of the need for the drainage pipe to be gravity flow. Careful construction was important, because of the railroad track’s low tolerance for deflection. The large diameter of the pipes increased the risk of heave.

The first 20 foot section of pipe was lowered into the launching pit and connected to the machine. The boring then began. After each 20 foot section was installed, the boring was paused while the next pipe section was mounted on the tracks and welded to the pipeline (see Figure 4.81).

The bore was kept on course and the first pipe installation was completed successfully. The track was then shifted, and the second 60 inch pipe was installed in the same manner as the first. This procedure was next followed for the third pipe (see Figure 4.82). The steel pipes each emerged at the receiving pits at point B within accuracy tolerance. The three bores each took about one day to complete. No unusual problems were encountered and no damage to the overlying railroad track was observed.

Figure 4.80. Auger boring machine on its track against the sheet pile thrust block
A new 20 feet long pipe section is welded to the pipeline.

The third pipe being installed.

Research Team Actions
The research team observed the installation and interviewed crew members. Soil samples were not recovered from this project.

Key Findings
This project provided a good example of the use of horizontal auger boring to install large diameter steel drainage pipe. Three parallel 60 inch diameter steel pipes were installed at a
depth of 5 feet below overlying railroad track and for a distance of 80 feet through gravelly organic soil.

Heave was considered a risk due to the large diameter of the bore and the small clearance between the top of the pipes and the railroad track. The low tolerance for deflection of the railroad track made it important for the contractor to take extra care for the accuracy of the bore.

The soil at the level of the pipe in the launching pit was observed to be dark organic clay with gravel, which is an appropriate soil type for auger boring.

The installation was completed successfully using the auger boring method. This project serves as an example of an appropriate use of auger boring technology.

*Horizontal Directional Drilling at Beach Road north of the Forker Building, Iowa State University, Ames, IA.*

**Project Information**

This project was located along Beach Road north of the Forker Building, on the campus of Iowa State University in April, 2008. The horizontal directional drilling technique was used to install two parallel 4 inch HDPE chilled water pipes along the road. The installed pipes have a length of 180 feet, and were bored at a depth to the top of the pipe of about 6 feet.

The pipe was installed by completing a single bore, as shown in Figure 4.83. The boring runs east-west, parallel to Beach Road (see Figure 4.84Figure 4.83). The research team observed the installation and interviewed crew members.
Figure 4.83. The location of the HDD across 210th Street in Boone, IA. (Bore path in red)

Figure 4.84. Plan view of the project site (Bore path in yellow)
Trenchless Method Selection
Trenchless construction was selected by the owner to avoid damaging protected trees in the path of the installation. This requirement made open-cutting unacceptable. Additionally, the presence of several existing utilities in the vicinity of the installation made open-cutting potentially a high risk than HDD.

Soil Conditions
No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to the relatively low risk nature of the bore. The soil uncovered during potholing and the digging of exit and entrance pits was observed by the contractor to gain an understanding of the soil conditions. According to the contractor, this soil consisted of a sand with some clay. This soil may have been fill from earlier construction. The boring was performed above the water table. This soil condition was considered appropriate for horizontal directional drilling construction.

Trenchless Installation
- The contractor made preparations for boring by setting up the HDD machine (see Figure 4.85) on the east side of the site (point A in Figure 4.83) for the first 180 feet bore. An exit pit was excavated using a backhoe at the planned termination of the run at point B to allow the pipe to emerge at the proper depth. The minimum depth of cover was specified to be 6 feet, which was followed by the contractor.

- A drilling fluid consisting of BOREGEL™ mix and water was used during the boring. This mix contains sodium bentonite, a clay-inhibiting polymer, and soda ash. This product is advertised to improve borehole stability in sandy soils. The fluid was mixed in a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 3 inch drill bit to the directional drilling machine and then pushing it into the ground. The pilot bore proceeded with periodic adjustments
being made to the depth and direction to keep the bore on-line. The borehole was drilled to a depth of about 6 feet to the expected top of pipe. The pilot bore was conducted successfully.

- The 3 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 5 inch reamer for the prereaming stage. The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, from the exit pit at point B in Figure 1 toward the directional drilling machine at point A. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 3 inches to 5 inches in diameter. The reamer cut away soil as it spun, and injected drilling fluid into the borehole from perforations in the reamer. The reamer successfully emerged by the drill rig at point A. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point B.

- Next, the 5 inch reamer was reattached to the drill string and the 4 inch diameter HDPE pipe was attached behind the reamer. The pipe and reamer were then pulled into the borehole at point B. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 18 inches. As the installation progressed, pauses were periodically made so an additional 20 foot section of HDPE pipe could be added to the pipeline by heat welding the pipe ends. Each heat welding process took about 15 minutes. The heat-welded joints were almost flush with the outside of the pipe and so added little to the drag of the pipe as it was pulled through the borehole. The pipe was installed successfully and without any problems. A pit was then dug at point A to allow better access to the end of the pipe. Finally, he pipe was capped until it later would be connected to the rest of the pipeline.

- The drill rig was then shifted 3 feet to bore the second hole. This bore was conducted using the same procedure that was used for the first. The second installation was also
completed successfully and without unusual difficulty. The installed parallel pipes are shown at point B in Figure 4.86.

![Directional drilling machine](image1)

**Figure 4.85. Directional drilling machine**

![HDPE pipes](image2)

**Figure 4.86. The 4 inch diameter HDPE pipes connected to the building near point B**

**Research Team Actions**

The research team recovered disturbed samples, which were taken from the middle of the thick sandy layer. The samples were removed to the lab for testing.
Soil Characterization

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.9. The 4 inch pipes were installed at a depth of 6 feet. The samples recovered by the research team confirmed that clayey sand was found at and above the installation. The gradation curve is shown in Figure 4.87. The water table was located deeper than 7 feet.

Table 4.9. Ames Beach Road project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 1.0</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.0 – 7.0*</td>
<td>Clayey sand (SC)</td>
<td>15.9</td>
<td>27.3</td>
<td>16.4</td>
<td>10.9</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 4 inch pipes at 6 foot depth, Location of water table is below this level

Figure 4.87. Soil gradation curves for depth = 5 to 7 feet.
Key Findings
This project involved the installation by horizontal directional drilling of two parallel 180 feet long, 4 inch diameter heat welded HDPE chilled water pipes. The depth to the top of pipes was about 6 feet.

The pipe was installed by first drilling a 3 inch pilot bore, followed by prereaming using a 5 inch reamer, before finally pulling the 5 inch reamer attached to the 4 inch product pipe through the hole. An identical procedure was used for both pipes.

The soil was tested in the lab and classified as clayey sand, which may have originated as fill for earlier construction. This soil is considered to be an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using the HDD method. This project serves as an example of an appropriate use of horizontal directional drilling.

Horizontal Directional Drilling at 210th Street and Quartz Avenue, Boone, IA.

Project Information
This project was located at the intersection of 210th Street (also known as Mamie Eisenhower Avenue) and Quartz Avenue in Boone, Iowa during early March, 2008. The horizontal directional drilling (HDD) technique was used to install a 12 inch diameter polyvinyl chloride (PVC) water pipe underneath 210th Avenue. The installed pipe has a length of 100 feet, and was bored at a depth to the top of the pipe of about 8 feet.

The pipe was installed by completing a single bore, as shown in Figure 4.88. The boring runs north-south, parallel to Quartz Avenue (Figure 4.89). The research team observed the installation and interviewed crew members.
Figure 4.88. The location of the HDD project across 210th Street in Boone, IA. (Bore path in red)

Figure 4.89. Plan view of the project site (Bore path in yellow)
Trenchless Method Selection

Trenchless construction was selected by the owner to avoid open-cutting 210th street, which is an important road in Boone. Trenchless installation therefore had a much lower social cost than did conventional open-cutting. The owner considered the project appropriate for either HDD or auger boring, but HDD was bid cheaper.

Soil Conditions

No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to an extensive familiarity with soil in the area and the relatively low risk nature of the bore. According to the contractor, this soil consisted of an average plasticity clay beneath the organic topsoil. Dewatering was not necessary as the boring was performed above the water table. This soil condition was considered appropriate for horizontal directional drilling construction.

Trenchless Installation

- The contractor made preparations for boring by setting up the HDD machine on the north side of the site (point A in Figure 4.88) for the 100 foot bore (see Figure 4.90). An exit pit was excavated at the planned termination of the run at point B to allow the pipe to emerge at the proper depth. The minimum depth of cover was specified to be 5 feet, but that minimum requirement resulted in most of the length of the installation being bored at a depth of about 8 feet. The boring crossed a abandoned water main, but no other utilities were present in the area.

- A drilling fluid consisting of MAXBORE HDD™ sodium bentonite, and a clay-inhibiting polymer, was used during the boring (see Figure 4.91). The bentonite was advertised to provide suspension, bore stability, filtration control, and help reducing drag. The fluid was mixed in a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.
• The pilot bore began by attaching a 5 inch drill bit to the directional drilling machine and then pushing it into the ground (see Figure 4.92). The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The borehole was drilled to a depth of about 8 feet to the expected top of pipe. The pilot bore was conducted successfully.

• The 5 inch drill bit from the pilot bore was then removed from the drill string and replaced by an 18 inch reamer for the prereaming stage. The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 5 inches to 18 inches in diameter. The contractor’s decision to substantial increase in borehole diameter without using incrementally larger reamers was due to the relatively short run (100 feet) and using a powerful drill rig. The reaming step was completed successfully, and the reamer emerged by the drill rig at point A on Figure 37. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point B.

• Next, the 18 inch reamer was reattached to the drill string and the 12 inch diameter PVC pipe was attached behind the reamer. The pipe and reamer were then pulled into the borehole at point B. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 18 inches. The pipe was installed successfully and without any problems. A pit was then dug at point A to allow better access to the end of the pipe. The pipe was capped until it later would be connected to the rest of the pipeline. This boring was completed in less than a day.
Figure 4.90. The directional drilling machine

Figure 4.91. Drilling fluid being vacuumed from the bottom of the exit pit at B

Figure 4.92. 5 inch diameter pilot bore drill bit
Research Team Actions

The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Disturbed soil samples were recovered at a depth of 6 feet, which was above the depth of the installation. These samples were removed to the laboratory in sealed plastic bags.

Soil Characterization

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.10. The 32 inch pipe was installed between the depths of 17 feet and 19.7 feet. The samples recovered by the research team confirmed that low plasticity clay was found directly above the installation, from the ground surface down to 16.0 feet. The gradation curve is shown in Figure 4.93. From 16.0 feet down to 20 feet, sandy clay was observed. The water table was located deeper than 20 feet.

Table 4.10. Boone 210th Street and Quartz Avenue project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.0</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.0 – 10.0*</td>
<td>Sandy lean clay (CL)</td>
<td>15.8</td>
<td>23.8</td>
<td>15.5</td>
<td>8.3</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 12 inch pipe at 8 foot depth, Location of water table is below this level
Key Findings

This project involved the installation by horizontal directional drilling of a single 100 feet long, 12 inch diameter PVC water pipe. The depth to the top of pipe was about 8 feet.

The pipe was installed by first drilling a 5 inch pilot bore, followed by prereaming using a 18 inch reamer, before finally pulling the 18 inch reamer attached to the product pipe through the hole.

The soil was tested in the lab and classified as low plasticity clay, which is common to the area. This soil is considered an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling.
Horizontal Directional Drilling at West 1st Street and Marion Street, Boone, IA

Project Information
This project was located at the intersection of West 1st Street and Marion Street in Boone, Iowa during late March, 2008. The horizontal directional drilling (HDD) technique was used to install a 16 inch diameter polyvinyl chloride (PVC) water pipe alongside Marion Street. The installed pipe has a length of 75 feet, and was bored at a depth to the top of the pipe of about 5.5 feet.

The pipe was installed by completing a single bore, as shown in Figure 4.94. The boring runs north-south, parallel to Marion Street (see Figure 4.95). The research team observed the installation and interviewed crew members.

Figure 4.94. The location of the HDD installation along 210th Street in Boone, IA. (Bore path in red)
Figure 4.95. Plan view of the project site (Bore path in yellow)

Trenchless Method Selection

Trenchless construction was selected by the owner to avoid the removal of two large trees that were blocking the open-cut pipe installation that was been planned in the area. Trenchless installation had the advantage of completing the project without the social costs of open cut installation. This included avoiding open-cutting through a homeowner’s driveway.

Soil Conditions

No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to an extensive familiarity with soil in the area and the relatively low risk nature of the bore. According to the contractor, this soil consisted of a low plasticity, silty clay beneath the organic topsoil. Dewatering would not necessary as the boring was
performed above the water table. This soil condition was considered appropriate for HDD construction.

Trenchless Installation

- The contractor made preparations for boring by setting up the HDD machine on the south side of the site (point A in Figure 4.94) for the 75 foot bore (see Figure 4.96). An exit pit was excavated at the planned termination of the run at point B to allow the pipe to emerge at the proper depth. Potholing was conducted to assure that adequate distance existed between existing utilities and the planned bore path. The potholing also allowed further observation of the subsurface soils. The minimum depth of cover was specified to be 5.5 feet, and that depth was followed along the entire bore path.

- A drilling fluid consisting of MAXBORE HDD™ sodium bentonite, and a clay-inhibiting polymer, was used during the boring (see Figure 4.97). The bentonite was advertised to provide suspension, bore stability, filtration control, and to help reduce drag (see Figure 4.98). The fluid was mixed in a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 5 inch drill bit to the directional drilling machine and then pushing it into the ground. The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The borehole was drilled to a depth of about 5.5 feet to the expected top of pipe. The pilot bore was conducted successfully.

- The 5 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 24 inch wagon wheel reamer for the prereaming stage (see Figure 4.99). The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing
pilot bore for guidance, and enlarged the hole from 5 inches to 24 inches in diameter. This substantial increase in borehole diameter was possible due to the relatively short run (75 feet) and a powerful drill rig. The reamer successfully emerged by the drill rig at point A on Figure 4.94. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point B.

- Next, the 24 inch reamer was reattached to the drill string and the 16 inch diameter PVC pipe was attached behind the reamer (see Figure 4.100). The pipe and reamer were then pulled into the borehole at point B. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 24 inches. The pipe was installed successfully and without any problems. The pipe was then capped until it later would be connected to the rest of the pipeline. The boring was completed in less than a day.

Figure 4.96. The directional drilling machine
Figure 4.97. Drilling fluid at the bottom of the exit pit at B

Figure 4.98. Drilling fluid close-up showing texture

Figure 4.99. 24 inch wagon wheel reamer
Figure 4.100. 16 inch pipe being lowered into the exit pit at point B, before it is pulled into place by the drill rig

Research Team Actions
The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Soil samples were not recovered for this project.

Key Findings
This project involved the installation by horizontal directional drilling of a single 75 feet long, 16 inch diameter PVC water pipe. The depth to the top of pipe was about 5.5 feet.

The pipe was installed by first drilling a 5 inch pilot bore, followed by prereaming using a 24 inch reamer, before finally pulling the 24 inch reamer attached to the product pipe through the hole.

The soil was observed to be a low plasticity silty clay, which is common to the area. This soil is considered an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling.
4.3 Field Monitoring

The field monitoring portion of the field work involved observing projects and measuring soil stresses during construction. This made the investigations of these projects more thorough than those in the “Site Visits” chapter. Undisturbed soil samples were recovered and tested in the laboratory. Samples were classified, and unconfined compression tests, consolidation tests, and multi-stage consolidated-undrained triaxial tests were performed when appropriate samples and testing equipment were available.

To measure the soil pressure changes during construction, a push-in pressure cell was installed near the bore path. The instrument selected was the Geokon model 4830 push-in pressure cell, which uses vibrating wire technology to measure total stress, pore water pressure, and temperature (see Figure 4.101). Two of these models were purchased, each with a range of 25 psi. This range was exceeded on several occasions, however the manufacturer informed us that the effect would be a shift in baseline readings, which could be corrected by simply taking new zero readings after the pressure cells were removed. The pressure cells are designed to be installed by a conventional drill rig.

Figure 4.101. Geokon model 4830 push-in pressure cell (from Geokon)
A Campbell Scientific CR5000 datalogger was used to record the pressure cells’ readings. However, a Geokon model GK-404 hand held vibrating wire reader was used on the first two “Field Monitoring” projects while several software and hardware problems with the datalogger were resolved. The use of the hand held reader had the effect that only discontinuous readings had to be taken. When problems with the datalogger were settled, it was used. The datalogger was programmed to take readings every 10 seconds, which was considered often enough to obtain a nearly complete record of pressure variations.

The two pressure cells were identical except for their calibrations, so it was important to be able to differentiate between them. The pressure cell with the total pressure sensor label #07-10022 and pore water pressure sensor label #07-10023 was named “A”. The pressure cell with the total pressure sensor label #07-10024 and pore water pressure sensor label #07-10025 was named “B”. These designations are used in the text.

The pressure cells were tested at the Spangler Geotechnical Lab Experimentation Site in Ames, Iowa, during April, 2008 in order to collect sample data (see Figure 4.102). An open area was selected where soil data existed from previous research (see Table 4.11). Iowa One Call facilitated the necessary utility locates, and a conventional drill rig was brought on-site for the installation of the instruments. The drill rig first drilled a borehole through the clayey soil to a depth of 5.0 feet, at which point the drill bit and auger were removed from the drill rig and the push-in pressure cell denoted “A” was attached to a drill rod. The pressure cell was then vertically pushed into the ground by the drill rig from the depth of 5.0 feet to 7.0 feet. Due to the configuration of the instrument, the sensors in the pressure cell were therefore centered at a depth of 6.5 feet, which is a common depth for water main installations. A similar process was then used to install the second pressure cell, denoted “B”. The pressure cells were allowed to continue taking readings for 27 days. The readings are shown in Figure 4.103.
Figure 4.102. The location of the pressure cell testing at Spangler Geotechnical Lab in Ames, IA.

Table 4.11. Ames, Spangler Lab testing soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 1.0</td>
<td>Dark gray silty sandy clay with organics*</td>
<td>12.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.0 – 5.0</td>
<td>Brown silty sandy clay #</td>
<td>12.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5.0 – 25*</td>
<td>Orange-brown silty sandy clay with gravel*</td>
<td>13.6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* Location of water table is below this level
# Reported by prior research
Figure 4.103. Total lateral earth pressure, pore water pressure, and temperature measured at a depth of 6.5 feet

Total pressure readings are observed to fluctuate by about 3 psi for both instruments. The gaps in the data occurred due to problems with the datalogger. It was observed that the pressure cells recorded a higher total pressure directly after installation, and stabilized after about 3 days. After that, the pressure cells exhibited some minor drift over the long period of time. This experimental data was used as a reference when examining the data that was recorded on actual job sites.
The research team installed the pressure cells on all six “Field Monitoring” projects. These projects included five HDD installations, and one impact moling installation. All six projects were located in central Iowa. The projects are next discussed in detail.

Second Horizontal Directional Drilling at Osborn Drive, Iowa State University, Ames, IA.

Project Information

This project was located along Osborn Drive, on the Iowa State University campus from late October through early November 2007. The horizontal directional drilling technique was used to install an 8 inch diameter HDPE domestic water pipe. The installed pipe has a length of 920 feet, and was located at a depth to the top of the pipe of 6 feet except when the bore had to be steered underneath an underground obstacle.

The project was completed using a series of three bores (see Figure 4.104), one for each straight-line portion of the pipeline. The boring runs generally north of MacKay Hall and east of Palmer Hall on the Iowa State University Campus, as shown in Figure 4.105. This includes a north-south bore approximately 85 feet long crossing Osborn Drive (between A and B on Figure 4.104), and east-west bore 495 feet long along the north side of Osborn Drive (between B and C on Figure 4.104), and a north-south bore 325 feet long crossing under Osborn Drive and the parking lot between Palmer HDFA Building and Bessey Hall (between C and D on Figure 4.104). The research team installed a vibrating wire push-in pressure cell at point E to observe changes in lateral earth pressure during installation.
Figure 4.104. The location of the HDD project at Osborn Drive in Ames, IA. (Bore path in red)

Figure 4.105. Plan view of the project site (Bore path in yellow)
Trenchless Method Selection

Trenchless construction was selected by the owner to avoid open-cutting across and along Osborn Drive, which is one of the main streets within the ISU campus. Trenchless construction also allowed landscaping and sidewalks to remain intact during installation. Horizontal directional drilling allowed the pipe to be installed underneath sidewalks and several large trees. Additionally, although University classes were in session during construction, the installation proceeded with minimal disturbance to vehicles and pedestrians in the area.

Soil Conditions

No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to an extensive familiarity with soil in the area. According to the contractor, this soil included a 20 foot layer of clay below the ground surface that is underlain by a deep stiff clay layer. Dewatering was not necessary as the boring was performed above the water table. This soil was considered appropriate for horizontal directional drilling construction.

Trenchless Installation

- The contractor made preparations for boring by setting up the HDD machine on the west side of the site (point B) for the 495 foot east-west portion of the bore (from B to C). An exit pit was excavated using a backhoe at the planned termination of the run (point C) to allow the pipe to emerge at the proper 6 foot depth. All existing utilities near the bore path, including a 6 foot high steam tunnel, were manually located by potholing (see Figure 4.106). Potholing was done at the contractor’s discretion as they were required to repair any existing utilities damaged during the installation, unless the damaged utility was unmarked.

- A drilling fluid consisting of BOREGEL™ mix and water was used during the boring. This mix contains sodium bentonite, a clay-inhibiting polymer, and soda ash. This product is advertised to improve borehole stability in sandy soils. The fluid was mixed in
a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a four inch drill bit to the directional drilling machine and then pushing it into the ground. The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line and to avoid nearby utilities. The borehole was drilled to a depth of about 6 feet to the expected top of pipe, except where steering vertically was necessary to avoid existing pipe. The position of the drill rig at point B while conducting the pilot bore for the run from B to C is shown in Figure 4.107. The pilot bore was conducted successfully and the drill bit emerged in the exit pit after about two hours.

- The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 14 inch reamer for the prereaming stage (see Figure 4.108). The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 4 inches to 14 inches in diameter.

- The prereaming stage continued until frac-out of the drilling fluid was observed directly above the bore path at point F on Figure 45, at a distance of 30 feet east from the pressure cell located at point E. Drilling fluid under pressure had ruptured the outer walls of the borehole and flowed to the ground surface, where it was observed as a puddle of mud. Drilling stopped while the excess drilling fluid was vacuumed. The prereaming process then resumed, and the reamer emerged near the drilling machine, where the pilot bore had originally started.

- The next day, the 8 inch HDPE pipe installation began. This process started by pushing the drill string from the drilling machine to the exit pit. This was quickly accomplished, as the 2 inch diameter drill rods were easily pushed through the 14 inch diameter
borehole that had been prereamed the day before. Once the drill string had emerged at the exit pit, the same 14 inch reamer used previously was reattached, as was the 8 inch HDPE pipe. The presence of the reamer would allow the borehole to be cleaned out and would ensure that the borehole diameter was 14 inches. This would remove any closure of the borehole that may have occurred overnight.

- The pipe installation then commenced as the drilling machine pulled the reamer and pipe back through the hole. As the installation progressed, pauses were periodically made so an additional 20 foot section of HDPE pipe could be added by heat welding the pipe ends. Each heat welding process took about 15 minutes. The heat-welded joints were almost flush with the outside of the pipe, so they added little drag to the pipe as it was pulled through the borehole. Figure 4.109 shows one of the heat-welded joints, and Figure 4.110 shows the pipe segments being pulled into the borehole.

- The pipe installation continued as expected until another frac-out occurred at point G, about 30 foot east of the pressure cell at point E. This point was about 100 foot west of the drilling machine at point B. Fluid pressures in the borehole had again built up and created a fissure in the borehole wall which propagated to the ground surface. The installation was again halted and a vacuum was used to remove the drilling fluid from the ground surface (Figure 4.111).

- The contractor tamped down the ground surface at the location of the frac-out in order to close the fissure, and then restarted the boring. More drilling fluid seeped through the crack, however, so the rate of machine pull-back was decreased to prevent drilling fluid pressures from elevating. The contractor was then able to successfully finish the 495 foot installation.

- After this pipe had been installed, the contractor prepared to begin the next section of boring by moving the directional drilling machine to point D at the southern end of the 325 foot north-south proposed pipe section from point C to D. Point D is located in an
asphalt parking lot between Palmer HDFA Building and Bessey Hall. Potholing was also necessary here in several spots, so small cuts were made through the asphalt.

- Once it was assured that no existing utilities lay near the path of the planned bore, the contractor installed the similar 8 inch HDPE pipe with the same procedure as the first bore. This installation was conducted successfully, and no instances of surface heave, frac-out, or settlement were observed.

- The contractor then moved the drilling machine to a location near where the machine had been set up at point B for the first boring, and this time aimed it south to complete the last 85 foot run of boring. The same procedure was used as the two earlier bores, and this installation was completed successfully and without any observed surface heave, frac-out, or settlement.

Figure 4.106. Potholing to verify the position of nearby utilities
Figure 4.107. The directional drilling machine

Figure 4.108. The 14 inch reamer

Figure 4.109. Heat-welded HDPE pipe joint
Figure 4.110. HDPE pipe (8 inch diameter) being pulled from exit pit at C (pictured) to the launching pit at A (not visible)

Figure 4.111. Drilling fluid at ground surface resulting from frac-out

Lateral Earth Pressure Monitoring
To characterize the soil at this site, the research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig. Disturbed samples were taken at various depths, and a 3 inch diameter thin-walled Shelby tube was pushed through the bottom of the first borehole to collect a sample from a depth of 5 to 7 feet (a depth similar to the depth of HDD installation). The soil samples from the Shelby tube and the disturbed bag samples were tested in the laboratory.
Additionally, a third vertical borehole was drilled at point E at a distance of 1 foot from the centerline of the planned bore path to a depth of 5 feet. Once this depth was reached, the drill bit and auger were removed from the drill rig and the push-in pressure cell was attached to a hollow rod (see Figure 4.112). This pressure cell was then vertically pushed into the ground by the drill rig from the depth of 5 feet to 7 feet (see Figure 4.113). This was done with the flat side of the pressure cell parallel to the planned horizontal borehole. The sensors in the pressure cell were centered at a depth of 6.5 feet, matching the intended depth of the HDD installation. The lateral distance between the face of the pressure cell and the center of the bore path was measured to be 1.3 feet (see Figure 4.114). The contractor’s pilot bore commenced one hour after the research team finished installing the pressure cell.

Figure 4.112. Push-in pressure cell before installation
Using the push-in pressure cell, the total lateral earth pressure, piezometric pressure, and temperature were recorded during the initial 495 foot east-west boring process from B to C. Due to a programming problem with the datalogger, a hand-held vibrating wire reader was
used to take the readings, which were then manually recorded. For this reason, fewer readings were recorded than would have been possible with a datalogger.

It can be observed from Figure 4.115 that the lateral earth pressure changes recorded when the boring implement passed showed a decrease in pressure instead of the expected increase. The pressure cells registered changes of -3.0 psi for the pilot bore, -6.3 psi for the prereaming, and -4.3 psi for the pipe installation. This at first counter-intuitive observation is discussed later in the report.

Additionally, the two incidences of frac-out of the drilling fluid were not accompanied by a measured pressure increase in the surrounding soil. This indicates that the fluid must have followed pre-existing cracks in the soil in order to reach the surface, rather than requiring a larger pressure buildup to fracture the borehole walls.
Figure 4.115. Total lateral earth pressure, pore water pressure, and temperature measured 1.3 feet from the centerline of the bore

**Key to Numbered Construction Events**

1) Pressure cell A is installed to read at a depth of 6.5 feet.
2) 4 inch pilot bore passes the pressure cell with its center at a depth of 6.0 feet and a lateral distance to the pressure cell of 1.3 feet.
3) 14 inch reamer pullback is begun and then paused before reaching the pressure cell due to frac-out.
4) 14 inch reamer passes the pressure cell with its center at a depth of 6.0 feet and a lateral distance to the pressure cell of 1.3 feet.
5) 14 inch reamer with 8 inch product pipe passes the pressure cell with its center at a depth of 6.0 feet and a lateral distance to the pressure cell of 1.3 feet.

6) Pressure cell A is removed.

**Figure 4.115. (continued)**

The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they relate to the HDD process.

**Soil Characterization**

Soil samples recovered included one 3 inch diameter and one 2 inch diameter Shelby tube samples from a depth of 5 to 7 feet, which matched the depth of pipe installation. Disturbed soil samples from various depths were also recovered and removed to the laboratory in sealed plastic bags.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.12. The gradation curves are shown in Figure 4.116. The soil profile between the ground surface and the pipe locations consists of two soil layers. The first layer, between the ground surface and 1.5 feet depth, consists of clay with organics topsoil. The second layer, from 1.5 feet to below the depth of pipe installation was found to be sandy lean clay. This second layer, in which the pipe was installed, has an average moisture content of 16.5%, a liquid limit of 28.3%, and a plasticity index of 11.9%. This classification matches the soil description that the contractor expected.
Table 4.12. Ames Osborn Drive second HDD project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Class.</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconf. Compr. Strength (psf)</th>
<th>Dry Density (pcf)</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.5</td>
<td>Peat (Pt)</td>
<td>20.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.5 - 2.5</td>
<td>Clayey Sand (SC)</td>
<td>18.7</td>
<td>38.0</td>
<td>21.1</td>
<td>16.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.5 - 3.0</td>
<td>Clayey Sand (SC)</td>
<td>17.7</td>
<td>38.0</td>
<td>21.1</td>
<td>16.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3.0 - 4.0</td>
<td>Sandy Lean Clay (CL)</td>
<td>15.8</td>
<td>28.3</td>
<td>16.4</td>
<td>11.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4.0 - 5.0</td>
<td>Sandy Lean Clay (CL)</td>
<td>14.9</td>
<td>28.3</td>
<td>16.4</td>
<td>11.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5.0 - 7.0*</td>
<td>Sandy Lean Clay (CL)</td>
<td>14.3</td>
<td></td>
<td></td>
<td></td>
<td>968.0</td>
<td>103.0</td>
<td>0.61</td>
</tr>
</tbody>
</table>

*Top of the 8 inch pipe at 6 foot depth,
Location of water table is below this level

Figure 4.116. Soil gradation curves for depth = 1.5 to 3 feet and depth = 3 to 5 feet
Relatively undisturbed soil from the 3 inch diameter Shelby tube was used for conducting unconfined compressive strength, consolidation, and consolidated-undrained multistage triaxial tests for the soil at the depth of the HDD installation. The measured unconfined compressive strength of the soil was 968 psf.

The consolidation test revealed an overconsolidation ratio of 3.2, a compression index equal to 0.17, and a preconsolidation pressure of 1.04 ton/ft² (see Figure 4.117). The average coefficient of consolidation is 0.70.

![Figure 4.117. Consolidation test graph showing vertical effective stress vs. void ratio](image)

The multi-stage consolidated undrained test was conducted at confining pressures of roughly 2, 4, 10, and 18 psi, representing initial lateral earth pressure and the range of expected lateral earth pressures during HDD installation. The stress-strain curves resulting from the test are provided in Figure 4.118. These results indicate a friction angle of 26.8° and a cohesion of 5.0 psi. The test picked up very small pore water pressure readings, so the effective friction angle was calculated to be 26.9° and the effective cohesion was calculated to be 4.8 psi. The initial modulus of the soil was 40, 150, 310, and 280 tsf at confining pressures of 2.2, 4.0, 9.9, and 18.0 psi, respectively.
Results given by comparing the q’ vs. p’ relationship is given in Figure 4.119. The graph indicates that α equals 29.3°, and a equals 1.0 psi.

Figure 4.118. Stress vs. strain curves from multistage CU test
Figure 4.119. $q'$ vs. $p'$ curves from multistage CU test

Key Findings
This project provided a good example of the use of horizontal directional drilling to install HDPE water pipe. 920 feet of pipe was installed at a depth to top of pipe of 6 feet through sandy lean clay soil. The 8 inch diameter pipe was installed by first drilling a pilot bore, followed by prereaming using a 14 inch reamer, before finally pulling the reamer attached to the product pipe through the hole. The bore was completed successfully, however, two instances of frac-out forced drilling fluid through the walls of the borehole and up to the ground surface.

The research team installed a vibrating wire push-in pressure cell to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear changes in the pressure readings as the boring proceeded past the pressure cell. This indicates that the boring is having an effect on the surrounding soil.
The soil was classified as sandy lean clay, which is an appropriate soil type for horizontal directional drilling. Frac-out could have been caused by one or a combination of the following factors: 1) a lack of soil cohesion may have enabled the drilling fluid to more easily breach the borehole walls. 2) a non-optimized drilling fluid mix may have contributed. 3) a lack of stability of the borehole walls contributed to the problem, and 4) the speed with which the reamer was pulled through the borehole could have also had an effect. It is impossible to conclude positively the cause of the frac-out, however.

The installation was completed successfully using horizontal directional drilling technology. The only significant problems encountered were due to frac-out, however, because this occurred in grass, the frac-out had no important negative effect on the installation other than delaying the construction. This project serves as an example of an appropriate use of horizontal directional drilling technology, while also demonstrating problems that may arise.

*Horizontal Directional Drilling Across Wallace Road, Iowa State University, Ames, IA.*

**Project Information**

This project was located on the campus of Iowa State University during November 2007. The horizontal directional drilling technique was used to install two parallel 6 inch diameter HDPE chilled water pipes. Each installed pipe has a length of 240 feet, and a depth to the top of the pipe of 6 feet except when the bore had to be steered underneath an underground obstacle.

The pipes were installed by completing two parallel bores (see Figure 4.120), one for each pipeline. The boring runs generally from north of the Seed Science Building across Wallace Road to the east, as shown in Figure 4.121. The research team installed two vibrating wire push-in pressure cells at this project at point B on Figure 4.120 to observe changes in lateral earth pressure during installation.
Figure 4.120. The location of the HDD project across Wallace Road in Ames, IA. (Bore path in red)

Figure 4.121. Plan view of the project site (Bore paths in yellow)
Trenchless Method Selection
Trenchless construction was selected by the owner to allow Wallace Road and the sidewalks along it to remain open. Open-cut trenching would have forced the closure of the road and sidewalks and forced motorists and pedestrians to detour around the area. Also, numerous existing utility pipes crossed the proposed bore path, and HDD was considered an appropriate way to avoid these obstacles.

Soil Conditions
The project specifications gave the contractor the option to conduct soil testing. The contractor considered soil testing unnecessary, though, due to the general familiarity with the sandy lean clays common to the area. An exit pit dug at point C in Figure 1 before boring began also provided the contractor with information regarding soil conditions (see Figure 4.122).

Dewatering was not necessary as the boring was performed above the water table. These soil conditions were considered appropriate for horizontal directional drilling.

Trenchless Installation
- The contractor made preparations for boring by first removing the asphalt surface in the driveway near where the boring would begin (point A in Figure 4.121). All nearby utilities were located to confirm that a safe distance existed between them and the intended bore path. Potholing was done at the contractor’s discretion as they were required to repair any existing utilities damaged during the installation, unless the damaged utility was unmarked. An exit pit was excavated at the planned termination of the run (point C), to allow the pipe to emerge at the proper 6 foot depth (see Figure 4.122). The HDD rig was then set up at point A facing east toward point C.

- A drilling fluid consisting of BOREGEL™ mix and water was used during the boring. This mix contains sodium bentonite, a clay-inhibiting polymer, and soda ash. This product is advertised to improve borehole stability in sandy soils. The fluid was mixed in
a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 4 inch drill bit to the directional drilling machine (see Figure 4.123) and then using the machine to push the drill bit into the ground. The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line (see Figure 4.124). The borehole was drilled to a depth of about 6 feet to the expected top of pipe. The pilot bore was conducted successfully and the drill bit emerged in the exit pit at point D.

- The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 10 inch reamer for the prereaming stage (see Figure 4.125). The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 4 inches to 10 inches in diameter. The pullback was paused after every 80 feet (five 16 feet long drill rods) and the reamer was pushed back toward point C to clean out the hole and help prevent frac-out. The pullback was then resumed. The prereaming was completed when the reamer emerged near the HDD rig at point A. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point C.

- Next, the 10 inch reamer was reattached to the drill string and the 6 inch diameter HDPE pipe was attached behind the reamer. 20 foot sections of pipe were butt-fused together to reach the total 280 foot length. The pipe and reamer were then pulled into the borehole at point C. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 10 inches. The pipe was installed successfully and without any problems. The pipe was then capped until it later would be connected to the rest of the pipeline.
After the first pipe had been installed, the contractor prepared to use the same procedure to install the second which was to be located parallel to the first pipe at a distance of 6 feet center-to-center. The drill rig was shifted south 6.0 feet, and the same 4 inch diameter pilot bore drill bit and 10 inch diameter reamer were used. This bore was also completed successfully with no problems (see Figure 4.126).

The contractor successfully installed both 280 feet long pipe sections of 6 inch diameter HDPE pipe. No significant problems were encountered.

Figure 4.122. The view looking east across Wallace Road toward the exit pit at point C. The 20 foot pipe sections and the pipe welding machine are visible

Figure 4.123. The 4 inch pilot bore drill bit attached to the drill rig
Figure 4.124. The directional drilling machine

Figure 4.125. The 10 inch reamer

Figure 4.126. The second pipe emerges at point A and the reamer is visible in front of the pipe
Lateral Earth Pressure Monitoring

The research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig in order to characterize soil at this site. Disturbed samples were collected at various depths, and a 3 inch diameter thin-walled Shelby tube was pushed through the bottom of both boreholes to collect a sample from a depth of 5 to 7 feet (a depth similar to the depth of HDD installation). The Shelby tube sampling was unsuccessful, however, due to the large percentage of sand, which lacked the cohesion necessary for the sample to remain in the tube during extraction. The soil samples from the disturbed bag samples, however, were tested in the laboratory.

The two vibrating wire push-in pressure cells were then installed at point B. To do this, a borehole was first drilled at a lateral distance of 3.0 feet from the planned bore path (see Figure 4.127). This borehole was drilled to a depth of 5.0 feet, at which point the drill bit and auger were removed from the drill rig and the push-in pressure cell denoted “B” was attached to a drill rod. The pressure cell was then vertically pushed into the ground by the drill rig from the depth of 5.0 feet to 7.0 feet. This was done with the flat, pressure sensing side of the pressure cell parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were therefore centered at a depth of 6.5 feet, matching the intended depth of the HDD installation. A similar process was then used to install the second pressure cell, denoted “A”. Pressure cell A was installed in a new borehole to take readings also at a depth of 6.5 feet, but at a lateral distance of 4.6 feet from the planned bore path. Pressure cell B was located 3.0 feet away from pressure cell A so that they were located as close as possible while still leaving a buffer (see Figure 4.129). This 3.0 foot buffer was to ensure that the soil deformations induced by the installation processes and the boring would not affect the other pressure cell’s readings.
Figure 4.127. Drilling of borehole through asphalt

Figure 4.128. Profile showing the pressure cells and the borehole (looking west)
The two vibrating wire push-in pressure cells recorded the total lateral earth pressure, piezometric pressure, and temperature during the 280 foot boring processes from point A to C. Due to a programming problem with the datalogger, a hand-held vibrating wire reader was used to take the readings which were then manually recorded. For this reason, fewer readings were recorded than would have been possible with a datalogger. However, enough readings were taken during each phase of the process so that a clear picture emerged on the effects of the directional drilling process on the lateral earth pressure of the surrounding soil.

The pressure cells were installed 4 days before the HDD work began. This was considered an adequate amount of time to allow the localized pressure increases due to soil deformations induced by the installation of the pressure cell to dissipate.

The data recorded from the push-in pressure cells made it possible to compile a record of lateral earth pressures at the locations of pressure cells A and B during the boring (see Figure 4.130). It is observed that the total pressure changes recorded by both pressure cells A and B were relatively similar during each of pass of the pilot drill and reamer. For the first pipe installation, pressure cell B, reading at a depth of 6.5 feet and at a measured distance to the center of bore of 9.9 feet, recorded no pressure change when the 4 inch pilot bore passed.
Pressure cell A, reading at a depth of 6.5 feet and a distance to the center of bore of 11.5 feet, also recorded no pressure increase. The 10 inch reamer for the first pipe caused a 0.1 psi pressure increase at pressure cell B and no pressure increase at pressure cell A. The final step in which the 10 inch reamer and the 6 inch HDPE pipe were pulled through the borehole created a 0.7 psi pressure increase at pressure cell B and no pressure increase at pressure cell A.

The installation of the second pipe produced higher readings because it was much closer to the pressure cells. During the second installation, pressure cell B, reading at a depth of 6.5 feet and a distance to the center of bore of 3.8 feet, recorded a pressure increase of 2.4 psi when the 4 inch pilot bore passed. Pressure cell A, reading at a depth of 6.5 feet and a distance to the center of bore of 5.4 feet, recorded a slight pressure increase of 0.1 psi. The 10 inch reamer for the first pipe caused a 2.5 psi pressure increase at pressure cell B and an increase of 0.2 psi at pressure cell A. The final step in which the 10 inch reamer and the 6 inch HDPE pipe were pulled through the borehole created a 2.1 psi pressure increase at pressure cell B and a small increase of 0.1 psi at pressure cell A.
Figure 4.130. Total lateral earth pressure, pore water pressure, and temperature measured 3.8, 5.4, 9.5, and 11.5 feet from the centerlines of the bores

**Key to Numbered Construction Events**

1) Pressure cells A and B are installed to read at a depth of 6.5 feet.
2) 4 inch pilot bore for the first pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B.
3) 12 inch reamer for the first pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B.
4) 12 inch reamer with the first 6 inch product pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B. This
installs the pipe at depth to top of pipe 6.0 feet.

5) 4 inch pilot bore for the second pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B

6) 12 inch reamer for the second pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B

7) 12 inch reamer with the second 6 inch product pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B. This installs the pipe at depth to top of pipe of 6.0 feet

8) Pressure cells A and B are removed

Figure 4.130. (continued)

The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they relate to the HDD process.

Soil Characterization
Disturbed soil samples from various depths were recovered and removed to the laboratory in sealed plastic bags.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.13. The 6 inch pipes were installed at a depth of 6 feet. The samples recovered by the research team confirmed that medium plasticity clayey sand was found between a depth of 2 feet and 7 feet. The gradation curve is shown in Figure 4.131. The water table was located deeper than 7 feet.

Table 4.13. Ames Wallace Road project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.0</td>
<td>Asphalt and Subbase</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.0 – 7.0*</td>
<td>Well Graded Sand with Silt and Gravel (SW-SM)</td>
<td>7.8</td>
<td>37.9</td>
<td>19.2</td>
<td>18.6</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 6 inch pipe at 6 foot depth, Location of water table is below this level
Key Findings

Horizontal directional drilling was used in this project to install two parallel 6 inch diameter HDPE chilled water pipes. Each pipe was installed 280 feet at a depth to the top of the pipe of 6 feet. The two pipes were offset 6.1 feet center-to-center.

Both pipes were installed using an identical procedure, in which a 4 inch pilot bore was first drilled, followed by prereaming using a 10 inch reamer, before finally pulling the 10 inch reamer attached to the product pipe through the hole.

The research team installed two vibrating wire push-in pressure cells to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear changes in the pressure readings as the boring proceeded past the pressure cells, especially during the prereaming stage. This indicates that the boring is having an effect on the surrounding soil.

Figure 4.131. Soil gradation curves for depth = 4 to 6 feet
The soil was tested in the lab and classified as clayey sand, which is probably present due to past construction activity. This soil is considered an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling.

*Horizontal Directional Drilling at the Hub, Iowa State University, Ames, IA.*

**Project Information**

This project was located on the campus of Iowa State University during March 2008. The horizontal directional drilling technique was used to install two parallel 3 inch diameter HDPE chilled water pipes. The installed pipes each have a length of 200 feet, and a depth to the top of the pipe of 6 feet except when the bore had to be steered underneath an underground obstacle.

The pipes were installed by completing two parallel bores (see Figure 4.132), one for each pipeline. The boring runs generally northeast of the Hub, which is a building housing vending machines in the middle of the Iowa State University Campus, as shown in Figure 4.133. The research team installed two vibrating wire push-in pressure cells at this project at point B on Figure 4.132 to observe changes in lateral earth pressure during installation.
Figure 4.132. The location of the HDD project at the Hub in Ames, IA. (Bore path in red)

Figure 4.133. Plan view of the project site (Bore paths in yellow)
Trenchless Method Selection
Trenchless construction was selected by the owner to avoid the numerous existing utilities that crossed the planned bore path. Additionally, the avoidance of open-cutting allowed several sidewalks to remain intact. Because the construction proceeded while university classes were in session, the continued use of these sidewalks was valuable in preventing the disruption of the flow of pedestrian traffic through the middle of campus.

Soil Conditions
No soil testing was conducted by the contractor or the owner before starting the HDD installation, mainly due to an extensive familiarity with soil in the area. According to the contractor, this soil included a layer of yellow clay at the depth of the bore. Dewatering was not necessary as the boring was performed above the water table. This soil condition was considered appropriate for horizontal directional drilling construction.

Trenchless Installation
- The contractor made preparations for boring by setting up the HDD machine on the north side of the site (point A in Figure 61) for the first of the two 200 feet northwest to southeast bores (from A to C). An exit pit was excavated using a backhoe at the planned termination of the run to allow the pipe to emerge at the proper 6 foot depth. All existing utilities near the bore path were manually located by potholing. Potholing was done at the contractor’s discretion as they were required to repair any existing utilities damaged during the installation, unless the damaged utility was unmarked. Potholing was therefore used as confirmation of the position of existing utilities located dangerously close to the proposed bore path.

- A drilling fluid consisting of sodium bentonite, a clay-inhibiting polymer, and a detergent was used during all stages of the boring. The polymer-based additive was WYO-VIS™, which is advertised to build viscosity, increase flowability, and stabilize the borehole in clay formations. The soil detergent was DRIL-SOL™, which is advertised to increase hole stability. The fluid was mixed in a separate tank and pumped through the hollow
drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 4 inch drill bit to the directional drilling machine and then pushing it into the ground. The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line and to avoid nearby utilities. The borehole was drilled to a depth of about 6 feet to the expected top of pipe, except where steering vertically was necessary to avoid existing pipe. It was necessary to steer the bore down to 8 feet deep at a couple locations. The position of the drill rig at point A while conducting the pilot bore for the run from A to C is shown in Figure 4.134. The pilot bore was conducted successfully.

- The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 6 inch reamer for the prereaming stage. The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 4 inches to 6 inches in diameter. The reamer successfully emerged by the drill rig at point A. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point C.

- Next, the 6 inch reamer was reattached to the drill string and the 3 inch diameter HDPE pipe was attached behind the reamer. 20 feet sections of pipe were butt-fused together to reach the total 200 feet length. The pipe and reamer were then pulled into the borehole at point C. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 6 inches. A pit was then dug at point A to allow better access to the end of the pipe. The installation was completed successfully and the pipe was then capped until it would later be connected to the rest of the pipeline.
After the first pipe had been installed, the contractor prepared to use the same procedure to install the second which was to be located parallel to the first pipe at a distance of 3.5 feet center-to-center. The drill rig was shifted west 3.5 feet, and the same 4 inch diameter pilot bore drill bit and 6 inch diameter reamer were used. This bore was also completed successfully with no problems.

The contractor successfully installed both 200 feet long pipe sections of 3 inch diameter HDPE pipe. No significant problems were encountered.

Figure 4.134. The directional drilling machine

Lateral Earth Pressure Monitoring
The research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig in order to characterize soil at this site. Disturbed samples were taken at various depths, and a 3 inch diameter thin-walled Shelby tube was pushed through the bottom of the first borehole to collect a sample from a depth of 5 to 7 feet (a depth similar to the depth of HDD installation). The soil samples from the Shelby tube and the disturbed bag samples were tested in the laboratory.

The two vibrating wire push-in pressure cells were then installed at point B. To do this, a borehole was first drilled at a lateral distance of 2.0 feet from the planned bore path. This borehole was drilled to a depth of 5.0 feet, at which point the drill bit and auger were
removed from the drill rig and the push-in pressure cell denoted “A” was attached to a drill rod. The pressure cell was then vertically pushed into the ground by the drill rig from the depth of 5.0 feet to 7.0 feet. This was done with the flat, pressure sensing side of the pressure cell parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were therefore centered at a depth of 6.5 feet, matching the intended depth of the HDD installation. A similar process was then used to install the second pressure cell, denoted “B”. Pressure cell B was installed in a new borehole to take readings also at a depth of 6.5 feet, but at a lateral distance of 3.0 feet from the planned bore path (see Figure 4.135). Pressure cell B was located 3.0 feet away from pressure cell A so that they were located as close as possible while still leaving a buffer. This 3.0 foot buffer was to ensure that the soil deformations induced by the installation processes and the boring would not affect the other pressure cell’s readings (see Figure 4.136).

Figure 4.135. Push-in pressure cell after installation
A problem arose before boring commenced when it was discovered that a misunderstanding about the location of an existing utility pipe necessitated shifting the planned bore paths of both new pipes 7 feet west. This occurred after the pressure cells had been installed, and meant that the borings would occur on their west sides rather than the east. This resulted in the centerline of the first bore being located 3.5 feet from pressure cell A and 4.5 feet from pressure cell B, and the centerline of the second bore being located 7.0 feet from pressure cell A and 8.0 feet from pressure cell B (see Figure 4.137). Additionally, the bore nearest to the pressure cells was conducted before the farther away bore, which would suggest that the readings for any lateral earth pressure increases caused by the second bore might be blocked by the already installed near bore.
Figure 4.137. Profile showing the pressure cells and the borehole (looking southwest)

The two vibrating wire push-in pressure cells recorded the total lateral earth pressure, piezometric pressure, and temperature during the 200 foot boring process from point B to D. The pressure cells were connected to a datalogger that was stored in a pick-up truck. The datalogger recorded readings once every 10 seconds. This frequency of readings was considered sufficient to create a clear picture of the effects of the directional drilling process on the lateral earth pressure of the surrounding soil.

The pressure cells were installed 1 day before the HDD work began. This was considered enough time for lateral earth pressure readings to mostly stabilize. The pressure cells were removed 2 days after boring finished (see Figure 4.138).
The data recorded from the push-in pressure cells made it possible to compile a complete record of lateral earth pressures at the locations of pressure cells A and B during the boring (see Figure 4.139). It is observed that the passage of the reamer during the installation of the first pipe created the largest total pressure changes recorded by both pressure cells A and B. Pressure cell A, reading at a depth of 6.5 feet and a distance to the center of bore of 3.5 feet, recorded a pressure increase of 0.9 psi when the pilot bore passed. Pressure cell B, reading at a depth of 6.5 feet and a distance to the center of bore of 4.5 feet, also recorded a pressure increase of 0.9 psi. The 4 inch pilot bore for the first pipe caused a 0.5 psi pressure increase at pressure cell A and a 0.3 psi pressure increase at pressure cell B. The final step in which the 6 inch reamer and the 3 inch HDPE pipe were pulled through the borehole created a 0.5 psi pressure increase at pressure cell A and a -0.5 psi pressure increase at pressure cell B.

The installation of the second pipe produced no readings of total lateral earth pressure change. This is likely due to the presence of the first pipe between the second borehole and the pressure cells, blocking the soil deformation. Additionally, the distance (7 feet and 8 feet) between the second borehole and the pressure cells might also be expected to cause the lack of pressure increase.
Figure 4.139. Total lateral earth pressure, pore water pressure, and temperature measured 3.5, 4.5, 7.0, and 8.0 feet from the centerlines of the bores

Key to Numbered Construction Events

1) Pressure cells A and B are installed to read at a depth of 6.5 feet.
2) 4 inch pilot bore for the first pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B.
3) 6 inch reamer for the first pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B.
4) 6 inch reamer with the first 3 inch product pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B.
to B. This installs the pipe at depth to top of pipe 6.0 feet
5) 4 inch pilot bore for the second pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 7.0 feet to A and 8.0 feet to B
6) 6 inch reamer for the second pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 7.0 feet to A and 8.0 feet to B
7) 6 inch reamer with the second 3 inch product pipe passes pressure cells with its center at a depth of 6.0 feet and a lateral distance of 7.0 feet to A and 8.0 feet to B. This installs the pipe at depth to top of pipe of 6.0 feet
8) Pressure cells A and B are removed

Figure 4.139. (continued)

The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they relate to the HDD process.

Soil Characterization
Disturbed soil samples from various depths were recovered and removed to the laboratory in sealed plastic bags.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.14. The 3 inch pipes were installed at a depth of 6 feet. The samples recovered by the research team confirmed that low plasticity clay was found between 5 feet and 7 feet deep. The gradation curve is shown in Figure 4.140. The water table was located deeper than 7 feet.
Table 4.14. Ames The Hub project soil parameters

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
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<tr>
<td>0 – 1.0</td>
<td>Peat (Pt)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.0 – 2.5</td>
<td>Fat clay (CH)</td>
<td>28.1</td>
<td>52.3</td>
<td>18.3</td>
<td>34.0</td>
<td>-</td>
</tr>
<tr>
<td>2.5 – 5.0</td>
<td>Lean clay with sand (CL)</td>
<td>22.0</td>
<td>36.3</td>
<td>15.6</td>
<td>20.7</td>
<td>-</td>
</tr>
<tr>
<td>5.0 – 7.0*</td>
<td>Sandy lean clay (CL)</td>
<td>15.2</td>
<td>26.5</td>
<td>13.4</td>
<td>13.0</td>
<td>-</td>
</tr>
</tbody>
</table>

*Top of the 3 inch pipes at 6 foot depth,
Location of water table is below this level

Figure 4.140. Soil gradation curves

Key Findings
This project involved the installation by horizontal directional drilling of two parallel 3 inch diameter HDPE water pipes. Each pipe was installed 200 feet at a depth to the top of the pipe of 6 feet.
The pipe was installed by first drilling a 4 inch pilot bore, followed by prereaming using a 6 inch reamer, before finally pulling the 6 inch reamer attached to the product pipe through the hole. Identical procedures were used for the two pipes, which were offset 3.5 feet center-to-center.

The research team installed two vibrating wire push-in pressure cells to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear changes in the pressure readings as the boring proceeded past the pressure cells, especially during the prereaming stage. This indicates that the boring is having an effect on the surrounding soil.

The soil was tested in the lab and classified as sandy lean clay, which is common to the area. This soil is considered an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling.

*Horizontal Directional Drilling at Pammel Drive, Ames, IA.*

**Project Information**

This project occurred along Pammel Drive in Ames, Iowa during June 2008. Horizontal directional drilling was used to install an 8 inch diameter HDPE water main pipe. The purpose of the project was to expand Iowa State University’s water distribution system to accommodate a new building scheduled to be constructed. The pipe section discussed in this report was installed using HDD for a distance of 480 feet at a depth to the top of the pipe of 6 feet.

The pipe was installed by drilling and then enlarging a two straight runs. A north-south run between points, “C” and “D”, and also an east-west bore between points “A” and “C” were completed (see Figure 4.141). The research team installed two vibrating wire push-in pressure
cells at this project at point B on Figure 71 to observe changes in lateral earth pressure during installation.

![Map of HDD project on Pammel Drive in Ames, IA.](image)

**Figure 4.141. The location of the HDD project on Pammel Drive in Ames, IA. (Bore path in red)**

**Trenchless Method Selection**
Trenchless construction was selected by the owner to allow Pammel Drive and the sidewalks along it to remain open. Open-cut trenching would have forced the closure of the road and sidewalks and forced motorists and pedestrians to detour around the area. Landscaping was also saved. Trenchless construction was called for in the plan set, and the two planned borings can be seen as part of the larger building construction project in Figure 4.142.
Soil Conditions
The project specifications gave the contractor the option to conduct soil testing. The contractor considered soil testing unnecessary, though, due to the general familiarity with the sandy lean clays common to the area. An exit pit dug at point D before boring began also provided the contractor with information regarding soil conditions. Dewatering was not necessary as the boring was performed above the water table. These soil conditions were considered appropriate for horizontal directional drilling.

Trenchless Installation
- The contractor decided to first attempt the 80 foot boring between points C and D. Preparations were made by first using a backhoe to dig an exit pit at point D. This pit would be used to visually confirm the locations of 3 existing pipes running parallel with Pammel Drive on its south side. Additionally, the pit at point D would allow the recovery of boring equipment at the proper depth. The HDD rig was then set up at point C, facing south toward point D. No additional utilities were located in the vicinity of the bore, so no additional potholing was necessary to confirm existing utility locations.

- A drilling fluid consisting of sodium bentonite and a clay buster detergent was used during all stages of the boring. The bentonite used was ASTEC® High Yield Bentonite. The drilling fluid was mixed in a separate tank and pumped through the hollow drill rods
to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 4 inch drill bit to the directional drilling machine (see Figure 4.143) and then using the machine to push the drill bit into the ground. The drill bit was advanced by pushing and spinning the drill rod using the hydraulic machinery of the drill rig. A sonde attached to the drill bit allowed a handheld locator at the surface to monitor the position of the drill bit in the ground (see Figure 4.144). The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The borehole was drilled to a depth of about 6 feet to the expected top of pipe. The pilot bore was conducted successfully and the drill bit emerged in the exit pit at point D.

- The 4 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 12 inch reamer for the prereaming stage. The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 4 inches to 12 inches in diameter. This prereaming process was executed successfully, but some difficulties were encountered due to the large amount of unexpected debris in the soil, such as cobble sized rocks, discarded rebar, additional relics from old construction. This slowed progress, but the pullback was completed when the reamer emerged near the HDD rig at point C. The reamer was removed, and the drill string was capped and pushed back through the borehole to the exit pit at point D.

- Next, the 12 inch reamer was reattached to the drill string and the 8 inch diameter HDPE pipe was attached behind the reamer. The 20 foot sections of pipe were butt-fused together to reach the total 80 feet length. The pipe and reamer were then pulled into the borehole at point D. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 12 inches. The pipe was installed successfully and
without any problems. A pit was then dug at point C to allow better access to the end of the pipe. The pipe was then capped until it later would be connected to the rest of the pipeline (see Figure 4.145).

- The contractor then followed a similar procedure for the 400 foot boring from point C to A. A larger, more powerful drill rig was brought in to handle the increased pullback resistance of the longer bore (see Figure 4.146). For this bore a 5 inch pilot drill bit was used instead of the 4 inch drill bit used for the bore from C to D. The larger drill bit was used because of the contractor’s intention to create a larger borehole for this longer installation, to decrease pullback pressures when the product pipe would be pulled through.

- Due to a potential conflict with additional existing utilities that was not discovered until several days before boring began, the owner and the contractor were forced to shift the planned bore path south 3 feet. This was due to an existing pipeline being located 3 feet away from the location stated on existing plans. This problem illustrated the importance of the standard practice of using a walk-over locator to determine exact locations of existing utilities, rather than completely trusting old plans.

- The pilot bore was then begun. Progress at first was somewhat slower than usual, with the underground debris encountered earlier as one cause. Also, stiff clay was encountered at a depth of about 8 feet, which was shallower than the 20 foot depth the contractor had expected from talking to the owner. An additional problem arose when, after 150 feet of boring, the sonde transmitter attached to the pilot bore became dislodged. This meant it would be impossible to accurately locate the pilot bore in the ground for the remaining 250 feet to be bored, so the contractor was forced to stop the bore and pull the drill bit back to the machine at point C. The sonde was then replaced, and the pilot bore was begun again. Soil conditions caused the rest of the bore to proceed slowly, but the run was eventually finished successfully. The drill bit eventually reached the surface at point A, as shown in Figure 4.147.
• Then, the 5 inch drill bit from the pilot bore was removed from the drill string at point A and replaced by a 14 inch reamer for the prereaming stage. The reamer was pulled back through the pilot borehole, and enlarged the hole from 5 inches to 14 inches in diameter (see Figure 4.148). This prereaming process was also executed successfully, although with some delay due to soil conditions. The reamer emerged near the HDD rig at point C, where it was removed. The drill string was capped and pushed back through the borehole to the exit point at A.

• Next, the 14 inch reamer was reattached to the drill string and the 8 inch diameter HDPE pipe was attached behind the reamer. The reamer and pipe were then pulled into the borehole at point A (see Figure 4.149). The pipe was installed successfully using the same procedure used for the bore from C to D, and no new problems were encountered. The pipe was capped and the trenchless installation was complete.

Figure 4.143. The directional drilling machine drilling from point C to D
Figure 4.144. The pilot bore is advanced underneath Pammel Drive. The photograph is looking north from point D to point C

Figure 4.145. HDPE pipe (8 inch diameter) after installation from point C to D. It is shown capped at point C. Locate wire is visible

Figure 4.146. HDD rigs set up for 80 foot C to D bore (left machine) and 400 foot C to A bore (right machine)
Figure 4.147. Drill bit (5 inch diameter) emerging at point A

Figure 4.148. Reamer (14 inch diameter) is pulled into the borehole at A (clockwise from upper left)
Lateral Earth Pressure Monitoring

The research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig in order to characterize soil at the site. Disturbed samples were taken at various depths, and a 3 inch diameter thin-walled Shelby tube was pushed through the bottom of the first borehole to collect a sample from a depth of 5 to 7 feet (a depth similar to the depth of HDD installation). The soil samples from the Shelby tube and the disturbed bag samples were tested in the laboratory.

The two vibrating wire push-in pressure cells were then installed at point B. To do this, a borehole was first drilled at a lateral distance of 3.0 feet from the planned bore path. This borehole was drilled to a depth of 5.0 feet, at which point the drill bit and auger were removed from the drill rig and the push-in pressure cell denoted “A” was attached to a drill rod. The pressure cell was then vertically pushed into the ground by the drill rig from the depth of 5.0 feet to 7.0 feet (see Figure 4.150). This was done with the flat, pressure sensing side of the pressure cell parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were therefore centered at a depth of 6.5 feet, matching the intended depth of the HDD installation. A similar process was then used to install the second pressure cell, denoted “B”. Pressure cell B was installed in a new borehole to take readings also at a depth of 6.5 feet, but at a lateral distance of 3.8 feet from the planned bore path. Pressure cell B was located 4.7 feet away from pressure cell A so that...
they were located as close as possible while still leaving a buffer. This 4.7 foot buffer was to ensure that the soil deformations induced by the installation processes and the boring would not affect the other pressure cell’s readings.

![Image](http://example.com/image.jpg)

**Figure 4.150. Push-in pressure cells being installed**

The two vibrating wire push-in pressure cells recorded the total lateral earth pressure, piezometric pressure, and temperature during the 400 foot boring process from point B to D. The pressure cells were connected to a datalogger that was stored in a pick-up truck. The datalogger recorded readings once every 10 seconds. This frequency of readings was considered sufficient to create a clear picture of the effects of the directional drilling process on the lateral earth pressure of the surrounding soil.

The pressure cells were installed 4 days before the HDD work began. This was considered an appropriate amount of time to allow the localized pressure increases due to soil deformations induced by the installation of the pressure cell to dissipate.

After the pilot bore had passed the pressure cells it was possible to measure the true lateral distance between the pressure cells and the actual center of the bore path. The walkover locator marked the position of the of the pilot bore as it passed, which was measured to be 3.0 feet laterally from the face of pressure cell A and 3.8 feet from pressure cell B (see Figure 4.151). These measured distances matched the target distances. The depth of the pilot bore
also followed expectations, as it was measured to be about 6.5 feet deep when it passed the pressure cells (see Figure 4.152).

Figure 4.151. The bore is seen approaching the two pressure cells (left)

Figure 4.152. Profile showing the pressure cells and the borehole (looking west)
The data recorded from the push-in pressure cells made it possible to compile a complete record of lateral earth pressures at the locations of pressure cells A and B during the boring (see Figure 4.153). It is observed that the passage of the product pipe attached to the reamer created the largest total pressure changes recorded by both pressure cells A and B. Pressure cell A, reading at a depth of 6.5 feet and a distance to the center of bore of 3.0 feet, recorded a pressure increase of 1.5 psi when the pilot bore passed. Pressure cell B, reading at a depth of 6.5 feet and a distance to the center of bore of 3.8 feet, recorded a pressure increase of 0.7 psi. The 14 inch reamer in the prereaming phase caused a 1.7 psi pressure increase at pressure cell A and a 0.5 psi pressure increase at pressure cell B. The final step in which the 14 inch reamer and the 8 inch HDPE pipe were pulled through the borehole created a 3.6 psi pressure increase at pressure cell A and a 1.0 psi pressure increase at pressure cell B. Additionally, two periods of severe weather that included strong rains and high winds caused pore water pressure increases through the permeable sandy soil, which led to accompanying total pressure increases.
Figure 4.153. Total lateral earth pressure, pore water pressure, and temperature measured 2.1 and 3.4 feet from the centerline of the bore from C to A

Key to Numbered Construction Events

1) Pressure cells A and B are installed to read at a depth of 6.5 feet.
2) Weather-related overnight pressure increase.
3) 5 inch pilot bore passes pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3.0 feet to A and 3.8 feet to B. This run was aborted due to the sonde becoming dislodged.
4) 5 inch pilot bore passes pressure cells with its center at a depth of 6.5 feet
and a lateral distance of 3.0 feet to A and 3.8 feet to B. This run was completed.

5) 14 inch reamer passes pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3.0 feet to A and 3.8 feet to B.

6) 14 inch reamer with the 8 inch product pipe passes pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3.0 feet to A and 3.8 feet to B. This installs the pipe at depth to top of pipe of 6.0 feet.

7) Weather-related overnight pressure increase.

8) Pressure cells A and B are removed

Figure 4.153. (continued)

Soil Characterization

The owner supplied a geotechnical report for the project, which made additional soil testing by the research team unnecessary (see Appendix D). Values for that report were used to estimate soil properties at the project site (see Table 4.15). The water table was located deeper than 10 feet.

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS Classification</th>
<th>Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Unconfined Compressive Strength (psf)</th>
<th>Dry Density (pcf)</th>
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<td>-</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>8000</td>
<td>125</td>
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</table>

*Top of the 8 inch pipe at 6 foot depth, Location of water table is below this level

Key Findings

The two separate horizontal direction drilling bores to install HDPE water pipe provided examples of effective use of HDD. During the first bore, the contractor installed 80 feet of 8 inch diameter pipe at a depth to top of pipe of 6 feet. The pipe was installed by first drilling a 4 inch pilot bore, followed by prereaming using a 12 inch reamer, before finally pulling the 12 inch reamer attached to the product pipe through the hole. The bore was completed successfully although with some delay due to unexpected debris being encountered
underground. The second bore involved installing 400 feet of 8 inch diameter pipe also at a depth of 6 feet. The contractor approached this bore slightly different from the first, however, due to the increased length. The contractor used a more powerful HDD rig than before, and substituted a 5 inch diameter pilot bore and a 14 reamer. In this way, the contractor drilled a wider borehole, which allowed the product pipe to be installed with less resistance.

The research team installed two vibrating wire push-in pressure cells to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear changes in the pressure readings as the boring proceeded past the pressure cells, especially during the pilot bore. This indicates that the boring is having an effect on the surrounding soil.

A geotechnical report supplied by the owner revealed that stiff sandy lean clay was located just above the depth of pipe installation.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling technology and demonstrates the potential effectiveness of the method.

**Impact Moling at IAMU Safety and Training Field, Ankeny, IA**

**Project Information**

This project occurred at the Iowa Association of Municipal Utilities’ Safety and Training Field in Ankeny, Iowa during late August 2008. Impact moling, a horizontal boring method, was used to install a length of 48 feet of ¾ inch copper gas pipe at a depth of about 4 feet. The project was completed as part of a training workshop for public gas employees.

The pipe was installed by first using a pneumatic piercing tool, often called a “mole”, to create a 2.5 inch diameter borehole. The copper pipe was then pulled through the hole and into position. The borehole was drilled between points A and C on Figure 4.154.
research team installed two vibrating wire push-in pressure cells at this project at point B to observe changes in lateral earth pressure during installation.

Figure 4.154 The location of the impact moling project at IAMU Safety and Training Field, 1735 NE 70th Avenue in Ankeny, IA. (Bore path in red)

Trenchless Method Selection
Impact moling was selected by the Iowa Association of Municipal Utilities (IAMU) as a topic to address in their workshop on the municipal gas industry. The project was intended to simulate the trenchless installation of new gas pipe in a residential area. Impact moling can be an effective method for short, shallow, and small diameter projects such as in this scenario because of impact moling’s fast speed, compact equipment, low cost, and simplicity of operation. Additionally, moling is especially suited for gas pipe applications as the air compressor can also be used for pressure testing. No plan set was created for this demonstration workshop.

Soil Conditions
The bore was conducted through native soils comprised of mainly black, fat clay. Impact moling is considered appropriate in these soil conditions, and the installation was considered low risk, so no soil testing was considered necessary. Also, the digging of entry and exit pits
on either side of the bore path in preparation for the bore allowed confirmation of the soil type at the depth of boring. Formal soil classification would have been encouraged, however, for a normal project. Some dewatering was necessary because heavy rain and topography of the area meant the water table was within 3 feet of the ground surface.

Trenchless Installation

- The contractor made preparations by first using a backhoe to dig entry and exit pits at points A and C, respectively. Both pits were excavated to dimensions of about 2 feet by 5 feet. The pits were dug to a depth of about 4 feet, as it would be necessary to launch the mole near the bottom of the entry pit. The bottoms of the pits were filled with about 8 inches of water, necessitating dewatering. The contractor equipment included a vacuum connected to a tank (see Figure 4.155), which was used to dewater the entry pit and allow the mole to be launched.

- This demonstration used a 2.5 inch diameter Grundomat impact mole, the head of which is shown in Figure 4.156. This mole head differs from models following an older, conventional design, in that the Grundomat mole contains a stepped head and concave tip, rather than a smooth conical head. This design alteration is intended to help the mole to continue its course when encountering an underground obstruction, such as a rock. No drilling fluid is necessary for impact moling and the equipment is not designed to deliver it.

- The next step was to position the 2.5 inch diameter mole at the bottom of the entry pit. The mole was laid in a starting cradle 1 to 2 inches above the bottom of the pit and slowly eased into the ground while using a telescopic aiming frame to monitor line and grade (see Figure 4.157). Compressed air from the trailer was turned on and the mole began piercing the soil. Line and grade were continuously monitored and adjusted until the mole had fully entered the soil, at which point further steering was impossible. The mole continued its progress into the soil without the need for further adjustments.
The mole continued to advance without pausing until it successfully reached the exit pit at point C (see Figure 4.158). The pit had become flooded with 8 inches of water because the tank of the vacuum was filled, so the mole was underwater when it reached the exit pit at point C. Dewatering was necessary so the mole did not “swim”, which could occur if the soil was sufficiently saturated and soft so that the mole could no longer advance. The vacuum tank was then emptied and the exit pit was dewatered allowing the mole to be retrieved (see Figure 4.159).

Next, the ¾ inch copper pipe was installed. This was done by disconnecting the pneumatic hose from the air compressor at point A, and then reconnecting it to the end of the copper pipe. The pneumatic hose was then used like a rope and pulled by hand out of the borehole at C while the copper pipe was pulled in at A (See Figure 4.160). This completed the installation.

The installation was finished with no problems encountered. The wet soil condition was the only challenge faced, and dewatering proved to be a sufficient measure to counteract it.

![Vacuum tank and Air compressor](image)

Figure 4.155. Contractor equipment including air compressor and vacuum tank used in dewatering
Figure 4.156. Grundomat reciprocating stepped-cone chisel head

Figure 4.157. Launching the mole and adjusting line and grade using a telescopic aiming frame.

Figure 4.158. Locating the mole after it emerged into the exit pit at point C. The pit is flooded due to the full vacuum tank preventing dewatering
Figure 4.159. The exit pit at point C is dewatered to allow the retrieval of the mole

Figure 4.160. The entry pit at point A as the 3/4 inch copper pipe is being pulled into the 2.5 inch diameter hole created by the mole

Lateral Earth Pressure Monitoring

The research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig in order to characterize soil at the site. Disturbed samples were taken at various depths. The research team attempted to recover undisturbed samples in 3 inch diameter thin-walled Shelby tubes, but the saturated state of the soil made it impossible to retain a sample in the tubes. The soil samples from the disturbed bag samples were tested in the laboratory.
The two vibrating wire push-in pressure cells were then installed at point B two days before the boring started. To do this, a borehole was first drilled at a lateral distance of 1.5 feet from the planned bore path. The drill bit was removed from the auger and a drill rod was attached to pressure cell “A,” which was then pushed into the ground by the drill rig to record pressure readings at a depth of 3.3 feet, matching closely the expected depth of the upcoming boring. The pressure cell installation was done with the flat, pressure sensing side of the pressure cell parallel to the planned horizontal borehole. A similar process was then used to install the second pressure cell, denoted “B”. Pressure cell B was installed in a new borehole to take readings also at a depth of 3.3 feet, but at a lateral distance of 2.5 feet from the planned bore path. Pressure cells A and B were located 3.0 apart in the direction parallel to the planned bore path, so that they were located as close as possible while still leaving a buffer. This 3.0 foot distance was to ensure that the soil deformations induced by the installation processes and the boring would not affect the other pressure cell’s readings.

The two vibrating wire push-in pressure cells recorded the total lateral earth pressure, piezometric pressure, and temperature during the 48 foot boring process from point A to C. The pressure cells were connected to a datalogger that was stored in a pick-up truck. The datalogger recorded readings once every 10 seconds. This frequency of readings was considered sufficient to create a clear picture of the effect that the impact moling process had on lateral earth pressure of the surrounding soil. The bore path and the location of the pressure cells can be seen in Figure 4.161.
Figure 4.161. The exit pit at C is seen in the foreground and the entry pit at A is seen in the background. The pressure cells at B are visible near the middle of the photograph.

After the mole had passed the pressure cell it was possible to measure more accurately the lateral distance between the pressure cells and the actual center of the bore path. The walkover locator marked the position of the of the mole as it passed, which was measured to be 1.6 feet laterally from the face of pressure cell A and 1.6 feet from pressure cell B. This was considered very close to the expected distances. The depth of the moling was slightly deeper than expected, however, as it was measured to be 3.8 feet instead of the 3.3 feet that was expected (see Figure 4.162). This slight depth discrepancy was expected to lower the pressure readings.
The data recorded from the push-in pressure cells made it possible to compile a complete record of lateral earth pressures at the locations of pressure cells A and B during the boring (See Figure 4.163). A problem with the datalogger prevented more readings from being taken after the moling. Pressure cell A, reading at a depth of 3.8 feet and a distance to the edge of the borehole of 1.6 foot, recorded a pressure decrease of 0.9 psi when the mole passed. Pressure cell B, reading at a depth of 3.8 feet and a distance to the edge of the borehole of 2.6 feet, recorded a pressure decrease of 0.5 psi. These decreases are likely related to the mole displacing soil radially as it moved through the ground.
Figure 4.163. Total lateral earth pressure, pore water pressure, and temperature measured 1.7 and 3.1 feet from the centerline of the bore

**Key to Numbered Construction Events**

1) Pressure cells A and B are installed to read at a depth of 3.3 feet.
2) 2.5 inch pneumatic mole passes pressure cells with its center at a depth of 3.8 feet and a lateral distance of 1.7 feet to A and 2.7 feet to B.
3) No more data is recorded. Pressure cells are removed 1 day later.
The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they relate to the HDD process.

**Soil Characterization**

Soil samples were not taken for this project, but the soil was observed to be black, fat clay.

**Key Findings**

This project demonstrated an effective use of impact moling to install copper gas pipe. The contractor installed 48 feet of 2.5 inch diameter pipe at a depth to top of pipe of 3.3 feet. The pipe was installed by first forming a borehole using an impact mole, and then pulling the pipe into the hole afterward. The bore was completed successfully with no significant problems encountered.

The research team installed two vibrating wire push-in pressure cells to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear reductions in the pressure readings as the boring proceeded past the pressure cells, however the changes indicated a negative pressure change. It is clear, however, that the boring is having an effect on surrounding soil pressures.

The soil observed to be fat clay down to the depth of installation. The high water table was considered not ideal for moling, but the installation was completed, nevertheless. Dewatering measures employed by the contractor proved to be effective in preventing the mole from becoming stuck in the middle of the bore.

The installation was completed successfully using impact moling. This project serves as an example of an appropriate use of impact moling technology and demonstrates the potential effectiveness of the method.
**Horizontal Directional Drilling at State Avenue, Ames, IA.**

**Project Information**
This project occurred along State Avenue in Ames, Iowa during September 2008. Horizontal directional drilling was used to install a 12 inch PVC water main pipe. The purpose of the project was to connect additional pipe to the nearby water tower and create an additional redundancy in the water distribution system. The pipe was installed using open-cut methods for part of the project, but HDD was used at strategic locations. The pipe section discussed in this report was installed using HDD for a distance of 30 feet at a depth to the top of the pipe of 8 feet.

The pipe was installed by drilling and then enlarging a single borehole running north-south parallel to State Avenue (see Figure 4.164). The borehole was drilled between points B and D. The research team installed two vibrating wire push-in pressure cells at this project at point C on Figure 4.164 to observe changes in lateral earth pressure during installation.

![Figure 4.164. The location of the HDD project along State Avenue in Ames, IA. (Bore path in red)](image-url)
Trenchless Method Selection

Trenchless construction was selected by the owner to allow the entrance to an ISU research farm field to remain open. Open-cut trenching would have cut off the field from vehicular access until the entrance could be rebuilt. Trenchless construction was called for in the plan set, and is stated as only being necessary under the field entrance (see Figure 4.165).

![Figure 4.165. Plan view of the project site taken from the plan set (Bore path in yellow)](image)

Soil Conditions

As a city of Ames project, the option of soil testing was the responsibility of the contractor as stated in the project specifications. In this case, the contractor elected not to conduct any soil testing. This was due mainly to a familiarity with the clayey soils common to the area. The contractor also stated that they normally talk to other people who drilled in the area earlier. Additionally, the digging of pits on either side of the field entrance, for the purpose of accessing the ends of the pipe after installation, also allowed visual confirmation of the soil types at the depth of boring. According to the contractor, this soil included a layer of clay located 3.5 feet below the field entrance surface that extended to a depth below the planned boring. Dewatering was not necessary as the boring was performed above the water table.
This soil condition was considered appropriate for horizontal directional drilling construction.

**Trenchless Installation**

- The contractor made preparations for boring by first using a backhoe to dig pits on both sides of the field entrance, at points B and D. These were the points where the 30 foot trenchlessly installed pipe would eventually be connected to the rest of the pipeline, which would be installed by open cutting later. The pit dug at point B is shown in Figure 4.166. The pit at point D would also serve as the exit pit where the bore would emerge to the surface. The contractor then set up the HDD machine on the north side of the site, at point A. No utilities were located in the vicinity of the bore, so no additional potholing was necessary to confirm existing utility locations.

- A drilling fluid consisting of sodium bentonite and a clay-inhibiting polymer was used during all stages of the boring. The bentonite used was TRU-BORE®, and the polymer was UNI-DRILL®. An additional substance, CON DET®, was added to help prevent the clay from sticking. The drilling fluid was mixed in a separate tank and pumped through the hollow drill rods to the cutting face where it was introduced to the borehole through perforations in the drill bit and the reamer.

- The pilot bore began by attaching a 5.5 inch drill bit to the directional drilling machine (see Figure 4.167) and then using the machine to push the drill bit into the ground. The drill bit was advanced by pushing and spinning the drill rod using the hydraulic machinery of the drill rig. A sonde attached to the drill bit allowed a handheld locator at the surface to monitor the position of the drill bit in the ground (see Figure 4.168).

- The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore on-line. The borehole was drilled to a depth of about 8 feet to the expected top of pipe, measured from the field entrance. This depth was used to reflect minimum depth requirements at low points in the ground surface, as stated in the
specifications. The pilot bore was conducted successfully and the drill bit emerged in the exit pit at point D after about 30 minutes of boring (see Figure 4.169).

- The 5.5 inch drill bit from the pilot bore was then removed from the drill string and replaced by a 16 inch reamer for the prereaming stage (see Figure 4.170). The directional drilling machine was switched from push mode to pull mode and the drill string with the reamer attached was then pulled back through the pilot bore, toward the directional drilling machine. The reamer was pulled back using the existing pilot bore for guidance, and enlarged the hole from 5.5 inches to 16 inches in diameter. This prereaming process was executed without problems, and the pullback was finished when the reamer emerged in the pit that was dug at point B. The machine was then switched to pushing mode and the reamer was pushed back into the borehole it had just carved, until it re-emerged at point D.

- Next, the 12 inch diameter PVC pipe was attached behind the reamer. A skid loader was used to move the pipe into place. The pipe was a combination of two 15 foot sections connected by a 16 inch diameter Certa-Lok® bell (See Figure 4.171). The pipe and reamer are shown being pulled into the borehole at point D in Figure 4.172. The reamer in front of the pipe served to clean out the borehole and ensure that the diameter was a full 16 inches. The pullback was subject to additional resistance due to the borehole and the bell diameters both being 16 inches, but this was deemed acceptable because the pullback was only 30 feet. The pipe was installed successfully and without any problems. Water was jetted around the opening of the pipe in the receiving end to allow the pulling head to be removed. The pipe was then capped until it later would be connected to the open-cut portion of the pipeline installation.

- The contractor also followed the same procedure for two similar bores in the vicinity. These were both also conducted successfully, with no problems encountered.
Figure 4.166. Pit for access to Potholing to verify position of nearby utilities

Figure 4.167. The directional drilling machine

Figure 4.168. The pilot bore is advanced as a crewman with a handheld locator (center) monitors depth and position. Also visible (top right) is a truck with the drilling fluid mixing tank
Figure 4.169. Pilot bore emerging from exit pit at point D

Figure 4.170. The 16 inch diameter reamer attached to the drill string

Figure 4.171. A 16 inch diameter Certa-Lok® bell connecting 12 inch diameter pipe sections
The research team drilled two vertical test boreholes near the bore path using a conventional augered drill rig in order to characterize soil at the site. Disturbed samples were taken at various depths, and a 3 inch diameter thin-walled Shelby tube was pushed through the bottom of the first borehole to collect a sample from a depth of 6 to 8 feet (a depth similar to the depth of HDD installation). The soil samples from the Shelby tube and the disturbed bag samples were tested in the laboratory.

The two vibrating wire push-in pressure cells were then installed at point C. To do this, a borehole was first drilled at a lateral distance of 2.0 feet from the planned bore path. This borehole was drilled to a depth of 6.0 feet, at which point the drill bit and auger were removed from the drill rig and the push-in pressure cell denoted “A” was attached to a drill rod. The pressure cell was then vertically pushed into the ground by the drill rig from the depth of 6.0 feet to 8.0 feet (see Figure 4.173). This was done with the flat, pressure sensing side of the pressure cell parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were therefore centered at a depth of 7.5 feet, matching the expected depth of the HDD installation. A similar process was then used to install the second pressure cell, denoted “B”. Pressure cell B was installed in a new
borehole to take readings also at a depth of 7.5 feet, but at a lateral distance of 3.5 feet from the planned bore path. Pressure cell B was located 3.0 feet away from pressure cell A so that they were located as close as possible while still leaving a buffer (see Figure 4.174). This 3.0 foot buffer was to ensure that the soil deformations induced by the installation processes and the boring would not affect the other pressure cell’s readings.

Figure 4.173. Push-in pressure cells being installed
The two vibrating wire push-in pressure cells recorded the total lateral earth pressure, piezometric pressure, and temperature during the 30 foot boring process from point B to D. The pressure cells were connected to a datalogger that was stored in a pick-up truck. The datalogger recorded readings once every 10 seconds. This frequency of readings was considered sufficient to create a clear picture of the effect that the directional drilling process had on lateral earth pressure of the surrounding soil.

The pressure cells were installed 5 weeks before the HDD work began. This time was longer than was necessary for the initial readings to stabilize, but contractor scheduling caused the HDD work to be delayed.

The pressure readings had reached a fairly stable, constant value in the time between their installation and the start of the HDD project. The approach of the pilot bore to the pressure...
cells’ location is shown in Figure 4.175. After the pilot bore had passed the pressure cells it was possible to measure the true lateral distance between the pressure cells and the actual center of the bore path. The walkover locator marked the position of the of the pilot bore as it passed, which was measured to be 2.1 feet laterally from the face of pressure cell A and 3.4 feet from pressure cell B. This was considered very close to the expected distances. The depth of the pilot bore was slightly deeper than expected, however, as it was measured to be 8.0 feet instead of the 7.5 feet that was expected. This slight depth discrepancy was expected to lower the pressure readings.

Figure 4.175. The bore is seen approaching the two pressure cells (left)

The data recorded from the push-in pressure cells made it possible to compile a complete record of lateral earth pressures at the locations of pressure cells A and B during the boring. It can be observed from Figure 4.176 that the pilot bore created the largest total pressure increase recorded by both pressure cells A and B. Pressure cell A, reading at a depth of 7.5 feet and a distance to the center of bore of 2.1 feet, recorded a pressure increase of 5.6 psi when the pilot bore passed. Pressure cell B, reading at a depth of 7.5 feet and a distance to the center of bore of 3.4 feet, recorded a pressure increase of 3.7 psi. The prereaming and pipe installation phases created much smaller pressure increases. The 16 inch reamer in the prereaming phase caused a 1.0 psi pressure increase at pressure cell A and a 0.5 psi pressure increase at pressure cell B. The final step in which the 16 inch reamer and the 12 inch PVC
pipe with a 16 inch bell were pulled through the borehole created a 1.6 psi pressure increase at pressure cell A and a 0.5 psi pressure increase at pressure cell B. It is difficult to determine the exact time the bell passed the pressure cells, as a period of small pressure fluctuations was recorded in the time after the reamer and the start of the pipe had passed the pressure cells. It is estimated that about 5 minutes may have elapsed although no single pressure spike was noticed. Small additional fluctuations after the installation was completed may be due to a backhoe being used very near the pressure cells.
Figure 4.176. Total lateral earth pressure, pore water pressure, and temperature measured 2.1 and 3.4 feet from the centerline of the bore

Key to Numbered Construction Events

1) Pressure cells A and B are installed to read at a depth of 7.5 feet.
2) 5.5 inch pilot bore passes pressure cells with its center at a depth of 8.0 feet and a lateral distance of 2.1 feet to A and 3.4 feet to B.
3) 16 inch reamer passes pressure cells with its center at a depth of 8.0 feet and a lateral distance of 2.1 feet to A and 3.4 feet to B.
4) 16 inch reamer with the 12 inch product pipe passes the pressure cells with its
center at a depth of 8.0 feet and at a lateral distance of 2.1 feet to A and 3.4 feet to B. The 16 inch diameter bell passes about 5 minutes later, causing a slight fluctuation in the pressure readings. The pipe was successfully installed.

5) Pressure cells A and B are removed.

Figure 4.176. (continued)

The soil samples that had been recovered during pressure cell installation were then analyzed to gain a better understanding of the soil’s properties and how they relate to the HDD process.

Soil Characterization
Soil on site was observed to be sandy lean clay. Scheduling allowed no time for soil testing.

Key Findings
This project demonstrated an effective use of horizontal directional drilling to install PVC water pipe. The contractor installed 30 feet of 12 inch diameter pipe at a depth to top of pipe of 8 feet. The pipe was installed by first drilling a pilot bore, followed by prereaming using a 16 inch reamer, before finally pulling the 16 inch reamer attached to the product pipe through the hole. The bore was completed successfully with no significant problems encountered.

The research team installed two vibrating wire push-in pressure cells to measure changes in lateral earth pressure during the boring. Results of this instrumentation show clear changes in the pressure readings as the boring proceeded past the pressure cells, especially during the pilot bore. This indicates that the boring is having an effect on the surrounding soil.

The soil was tested in the lab and classified as sandy lean clay, which is common to the area. This soil is considered an appropriate soil type for horizontal directional drilling.

The installation was completed successfully using horizontal directional drilling. This project serves as an example of an appropriate use of horizontal directional drilling technology and demonstrates the potential effectiveness of the method.
4.4 Discussion

The research team visited trenchless jobsites in Iowa with the goal of gaining a better understanding of how this work is performed, and to better identify the risks involved in trenchless technologies and how these risks can be minimized. A total of 19 projects were visited. The trenchless methods used on these projects included 1 pipe jacking, 1 tunneling, 1 impact moling, 5 auger boring, and 11 HDD.

The “Site Visits” portion of the field work involved observing 13 trenchless construction projects. These projects involved the research team making visits to the job site and observing and documenting construction practices, and the successes and failures experienced. Lab testing was also done to evaluate soil properties. Pipe sizes installed ranged from 0.75 inches in diameter up to a 10 by 5 foot box culvert. Installation lengths ranged from 24 up to 495 feet. These projects were all completed successfully, but on one project, the HDD caused frac-out and surface heave.

This project was undertaken at Johnny Majors practice field in Ames, Iowa. The project involved using HDD to install two 4 inch HDPE pipes in a single borehole over a length of 400 feet and at a depth of 17 feet. After successfully creating the 4 inch pilot hole, problems were experienced during the prereaming stage, when the hole was enlarged to 12 inches in diameter. Stiff soil cause boring to proceed very slowly, and while drilling fluid was being pumped out of the reamer and into the borehole, less volume of drilling fluid than expected was observed to be emerging at the launching and retrieval pits. This led to a pressure buildup causing the borehole walls to rupture and frac-out to appear on the ground surface. This occurred while boring through gravelly clay soil. The prereaming process was continued and several hours later, surface heave of about 10 inches appeared directly over a portion of the bore path. This occurred about 10 feet after switching from gravelly soil to clay.

Ultimately, the cause of this failure is probably due to an incorrect drilling fluid mixture. A drilling fluid high in bentonite was used due to the contractor’s uncertainty of the subsurface
conditions. When clay was encountered, the lack of a clay inhibiting polymer in the drilling fluid cause the drilling fluid to stick to the gravelly clay and led to the borehole sealing shut. The reamer itself was also becoming clogged with clay solids, preventing the drilling fluid from flowing out past the reamer and through the smaller pilot borehole toward the entry pit. This led to fluid pressures rising and the soil eventually fracturing along planes of greatest weakness and the evacuation of fluid to the surface. This hypothesis is supported by the research team’s later subsurface soil investigation, in which permeable sandy soil was observed around the area of frac-out and less permeable clay around the area of heave. A better knowledge of subsurface conditions would have caused the contractor to use a different drilling fluid mix which and probably would have avoided this problem. This suggests that additional field testing is advisable on HDD projects when the contractor has significant uncertainty of the subsurface conditions.

The second project that encountered frac-out was studied as one of the “Field Monitoring” projects, in which the research team recovered undisturbed soil samples for laboratory testing, measured soil stresses during the construction, and carried out more in-depth investigations of project. This project installed an 8 inch pipe at a depth of 6 feet for a distance of 485 feet. Frac-out first occurred while prereaming a 4 inch pilot hole to 14 inches in diameter. The contractor vacuumed the spilled drilling fluid, and continued the bore. A second instance of frac-out occurred while the pipe was being installed behind the 14 inch reamer. A push-in pressure cell was being used to monitor soil pressure increase caused by the bore, but no increases were measured directly before either frac-out. This suggests that drilling fluid pressures never became extremely high, and the fluid may have followed existing fractures in the soil rather than requiring a large pressure buildup to fracture the borehole walls.

This frac-out could have been caused by one or a combination of the following factors: 1) a lack of soil cohesion may have enabled the drilling fluid to more easily breach the borehole walls. 2) a non-optimized drilling fluid mix may have contributed. 3) a lack of stability of the borehole walls contributed to the problem, and 4) the speed with which the reamer was
pulled through the borehole could have also had an effect. It is impossible to conclude positively the cause of the frac-out, however.

The research team noted that the majority of projects that were observed did not utilize a soil testing program. It was generally felt by the contractors that experience in an area made soil testing an unnecessary expense.

The pressure cell results of the “Field Monitoring” project are shown in Table 4.16, displaying the soil pressure increases measured during passes of the boring equipment. Readings were taken in sandy lean clay, well graded sand with silt and gravel, and fat clay. The pressure cells recorded readings between 1.3 feet and 11.5 feet from the edge of the borehole. Readings as high as 5.6 psi were recorded. During the Osborn Drive project, negative pressure increases were recorded during the passage of the HDD pilot bore, prereamer, and pipe pull-in. These results cannot yet be explained.
Table 4.16. Field monitoring results

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<th>Project</th>
<th>Trenchless Method</th>
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<th>Distance from Bore (ft)</th>
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Total pressure readings from HDD installations of two 3 inch diameter pipes installed in one borehole through well graded sand with silt and gravel during the passage of the 4 inch pilot bore are shown in Figure 4.177. It is observed that the pressure readings consistently decrease with distance from the borehole. This expected result is partially because the all
four sets of readings came from a single project, in which two separate pipes were installed. Both pressure cells were installed at identical depths, so the readings allow a valid comparison to be made. A similar decrease of pressure increase with depth is observed during the 10 inch reamer pullback in Error! Reference source not found.. The pressure variation with distance measured during the pipe pull-in stage is not consistent with what is expected, however (Figure 4.179).

Figure 4.177. Total pressures induced by the pilot bore for projects installing two 6 inch pipes in well graded sand with silt and gravel
Figure 4.178. Total pressures induced by the prereaming step for projects installing two 6 inch pipes in well graded sand with silt and gravel

Figure 4.179. Total pressures induced by the reamer and pipe pull-in step for projects installing two 6 inch pipes in well graded sand with silt and gravel

Cavity expansion theory was used to compare predicted soil stress increases with measured stress increases at various distances from the borehole. The equation below from Yu (2000)
was used for the calculation of radial pressure ($\sigma_r$) at selected distances from the center of the borehole.

$$\sigma_r = p_0 + (p - p_0)\left(\frac{a}{r}\right)^3$$

In the equation, $p_0$ is the external pressure, taken to be zero, $p$ is the internal pressure, which was assumed to be less than 400 psi, $a$ is the inner radius of the borehole, and $r$ is the distance from the center at which radial pressure is being calculated.

Calculations were also made for the tangent pressure ($\sigma_\theta$) at selected distances from the center of the borehole. The equation below also comes from Yu (2000).

$$\sigma_\theta = p_0 - 0.5(p - p_0)\left(\frac{a}{r}\right)^3$$

These calculations were first made assuming a 4 inch pilot bore was being expanded. This allowed direct comparison with measured pressure results from projects that involved expanding a 4 inch pilot bore. The HDD projects at the Hub, Wallace Road, and Pammel Drive all fit this criterion. The results of the calculations with the measured readings overlayed are given in Figure 4.180.
Figure 4.180. Comparison of stress calculations and stress measurements made during 4 inch pilot bores

A similar comparison was also made for the HDD project that began with a 5.5 inch pilot bore. These results are shown in Figure 4.181.
Figure 4.181. Comparison of stress calculations and stress measurements made during a 5.5 inch pilot bores

Calculated values were also established that measure cavity expansion in terms of the change in area divided by area. An equation from Hunter (2004) was used to approximate the values. This equation solves for the radial pressure at the borehole to soil interface.

\[ p_r = p_0 + c_u \left[ 1 + \ln \left( \frac{G}{c_u} \right) + \ln \left( \frac{\delta A}{A} \right) \right] \]

The undrained shear strength is \( c_u \), the shear modulus is \( G \), and the change in area divided by area is \( \frac{\delta A}{A} \).

The graph showing the relationship between the radial pressure and the change in area of the cavity over area of the cavity using soil parameters from the Osborn Drive HDD project is shown below Figure 4.182.
Figure 4.182. Calculation of radial pressure for a given change in area over area and the external pressure
5 SUMMARY AND CONCLUSIONS

This research was undertaken with two primary objectives, as listed below.

- To document the current practices and applications of trenchless technology in the United States and particularly in Iowa.
- To evaluate the effects of trenchless construction on surrounding soil and adjacent structures.

These objectives were studied by first performing a literature review to assemble information on the current practice of trenchless technologies. The literature review examines the rationale for trenchless technology and introduces the major trenchless construction and rehabilitation methods. Also discussed are soil investigation methods for trenchless projects, quality control/quality assurance, the effects of trenchless technologies on surrounding soil, and design processes.

A program of surveying and interviewing trenchless practitioners was then undertaken to gain additional insights into experiences in the field, focusing mainly on practices in Iowa. Three separate surveys were sent to professionals, with each survey targeting a different geographic region. The surveys targeted Iowa, the Midwest, and the entire United States. These surveys and interviews focused on the four major topics listed below.

- Method familiarity
- Observed pavement distress
- Reliability of methods
- Future improvements

The Iowa survey gained 34 respondents, 60% of whom were public employees while 40% were contractors and consultants. The survey revealed that HDD, auger boring, pipe jacking,
and cured-in-place pipe are thought to be the most common methods used in Iowa. The respondents also reported that pipe ramming and pipe bursting are viewed the least favorably of the common methods, due to their perceived risks. Respondents were asked if they had seen pavement distress or other problems occur as a result of trenchless technologies, and 47% reported that they had.

A shorter survey was sent to professionals around the Midwest, and it gained 32 respondents. 22% of these professionals were public employees, while 78% were contractors and consultants. These respondents reported that the most common trenchless methods used in the Midwest are HDD, cured-in-place pipe, pipe jacking, and localized repairs. Of these respondents, 29% reported seeing pavement distress or other problems occur due to trenchless technologies.

Questions from the Midwest survey were included in a larger survey by Dr. Mohammed Najafi, of the University of Texas at Arlington, and sent to state employees across the country. The 12 respondents reported that sliplining, HDD, pipe jacking, cured-in-place pipe, and localized repairs were the methods that they had encountered most.

Many additional comments were collected related to these surveys, and a program of interviews was undertaken which resulted in the research team collecting many comments related to trenchless technology. Professionals commonly expressed several general comments.

- There is a desire for cost effective QC/QA standards to reduce risk.
- Encountering unmarked utilities is a major problem.
- More soil testing could be useful, as many projects currently use no soil testing.
- Heave or subsidence due to trenchless construction can cause ground movements of up to about 2 feet.
Survey and interview results indicate that the frequency of pavement distress or other trenchless related problems is an ongoing problem in the industry. Inadequate soils information and QC/QA are partially to blame.

A field investigation was performed which involved observing 19 trenchless construction projects, documenting procedural successes and failures, interviewing personnel, recovering soil samples for laboratory testing, and measuring stress changes in the soil surrounding the borehole during construction. Laboratory testing was carried out to better understand the interactions between the trenchless construction processes and the soil. Lastly, the results were analyzed and discussed.

The trenchless construction projects that were studied in the field work were classified as “Site Visits” and “Field Monitoring.” The “Site Visits” portion of the field work involved observing 13 trenchless construction projects. These projects involved the research team making visits to the job site and observing and documenting construction practices, and the successes and failures experienced. Lab testing was also done to evaluate soil properties. “Field Monitoring” involved observing projects and measuring soil stresses during construction. This made the investigations of these projects more thorough than those in the “Site Visits” chapter. Undisturbed soil samples were recovered and tested in the laboratory. Samples were classified and unconfined compression tests, consolidation tests, and multi-stage consolidated-undrained triaxial tests were performed when appropriate samples and testing equipment were available.

Soil stresses in the field were measure during the 6 “Field Monitoring” projects by installing push-in-pressure cells in the ground near the bore path before boring began. This provided readings of soil pressure increase experienced during the passage of the boring equipment near the instruments. Soil samples were analyzed in the lab to correlate observations and pressure readings to soil properties.
Pipe sizes installed during the observed projects ranged from 0.75 inches in diameter up to a 10 by 5 foot box culvert. Installation lengths ranged from 24 feet up to 495 feet. These projects were all completed successfully, but in two projects, the HDD caused frac-out and surface heave. The trenchless methods used on the 19 total projects included 1 pipe jacking, 1 tunneling, 1 impact moling, 5 auger boring, and 11 HDD projects.

The first project to experience surface heave and frac-out was an HDD installation of two 4 inch diameter HDPE pipes in one borehole. The depth of installation was 17 feet, and the length of bore was 400 feet. The cause of this failure was probably an incorrect drilling fluid mixture. A drilling fluid high in bentonite was used due to the contractor’s uncertainty of the subsurface conditions. When clay was encountered, the lack of a clay inhibiting polymer in the drilling fluid caused the drill fluid to stick to the gravelly clay soil and led to the borehole sealing shut. Fluid pressures rose and the borehole eventually fractured along planes of greatest weakness and drilling fluid evacuated to the surface. This hypothesis is supported by the research team’s later subsurface soil investigation, in which permeable sandy soil was observed around the area of frac-out and less permeable clay around the area of heave. A better knowledge of subsurface conditions would have caused the contractor to use a different drilling fluid mix, and probably avoid this problem. This suggests that additional field testing is advisable on HDD projects in which the contractor has significant uncertainty of the subsurface conditions.

The second project to experience frac-out was an HDD installation of 8 inch HDPE pipe at a depth of 6 feet and over a length of 495 feet. The frac-out could have been caused by one or a combination of the following factors: 1) a lack of soil cohesion may have enabled the drilling fluid to more easily breach the borehole walls. 2) a non-optimized drilling fluid mix may have contributed. 3) a lack of stability of the borehole walls contributed to the problem, and 4) the speed with which the reamer was pulled through the borehole could have also had an effect. It is impossible to conclude positively the cause of the frac-out, however.
The research team noted that the majority of projects that were observed did not utilize a soil testing program. It was generally felt by the contractors that experience in an area made soil testing an unnecessary expense.

Calculations were made using cavity expansion theory to predict the pressure increases that would have been expected. These results are shown in the “Discussions” section after the field work.

Future research could allow a better understanding of several trenchless construction questions. Soil pressure monitoring of trenchless methods additional to HDD and impact moling could allow a better understanding of how the different methods interact with soil. Also, additional finite element modeling of the different installation procedures could improve understanding of the conditions which increase the risk of pavement deformations and other problems. Additionally, an improved knowledge of the physical causes of HDD drilling fluid pressure build-ups which lead to heave and frac-out could lead to a decreased risk of these problems.

Because the projects observed by the research team were successful overall, trenchless technologies appear to be effective methods for utility pipe installation in areas where open-trenching is undesirable. The experience level of the contractor is very important, however, and it is also important to conduct soil testing in areas of uncertain subsurface conditions. It is expected that technological improvements and growing experience will result in trenchless technologies becoming more popular in Iowa and worldwide.


APPENDIX A: DETAIL FROM SURVEY AND INTERVIEW RESPONSES

Midwest, Iowa, and national survey respondents were asked to elaborate on why they felt current levels of QC/QA associated with trenchless projects were or were not appropriate. Responses were mixed, but it was generally felt that current methods are not always adequate. Individual comments are listed below.

- Problems experienced with CIPP, including sags in the felt liner, poor adhesion, and difficulty telling if appropriate temperatures for curing are maintained uniformly.
- Current lack of real-time monitoring of ground movements
- Lack of good understanding by local authorities
- Short warranty on completed project
- Overall inexperience of personnel involved
- Enforcement of QC/QA in specifications
- Lack of well trained inspectors. Not enough are allowed to attend the many conferences/training sessions which are available
- Contractor short-cuts
- In some cases companies reduce the amount of resin in CIPP liners to reduce cost
- Sometimes adequate, it depends on how much knowledge of the area is available
- Need more vertical soil borings along route of bore
- Soil borings can miss localized problem areas
- Ok for water mains
- Testing doesn’t consider soil stability

Iowa survey respondents were asked about the currently used methods of soil investigation prior to trenchless construction. Their responses are summarized below.

- Vertical soil borings (many responses). The presence of sand and the depth of the water table are important.
• No soil explorations before auger boring (from one respondent)
• Geophones
• Soil strength
• Rely on experience of engineer, client, and local contractors
• No one does soil investigations on rural roads.
• Soil borings at both ends of trenchless construction work
• Soil classifications and water table depth.
• Important to know locations of sand to decide if auger bore requires a casing.
• Finding the location of rock
• Test holes dug using Hydrovac excavation equipment

The Iowa survey asked respondents what lessons could be learned from failures experienced. Summarized responses are given below.

• Geotechnical exploration is critical before starting trenchless construction.
• Experienced contractors are very important.
• Respondent thought that a casing should be used for any installation larger than 6 inches in diameter.
• Jacking a pipe and grouting is not good practice on pipes with bells.
• Overlying street should be monitored daily with chain drags and the street should be core drilled to check for voids.
• More oversight needed by the community of the work performed by utility companies
• The contractor should monitor the amount of material removed from the casing during auger boring as the casing is advanced to minimize the amount of over-excavation.
• HDD boring should be deeper under sidewalks, or you should just open-cut and replace the sidewalks.
• Use a high quality closed circuit television before placing a liner.
Installation process must move beyond art and develop more parameters to ensure reliability.

Accurately locating existing utilities is very important.

Cobbles and boulders in glacial till can alter alignment, slow advancement, and break pipe.

Space limitations can exist within right of way that make it difficult to properly shore and brace an excavation.

Problems can occur when pulling CIPP through pipes that are separated or out of alignment.

Boring contractor can’t always tell when they hit existing utilities.

Additional thoughts from the Midwest survey on the topic of reliability of trenchless technologies are given below:

- Qualifications of geotechnical engineers and contractors are very important
- Low bid process can be risky procurement method
- One city employee said he had experienced only one failure in 340,000 feet of small and medium diameter and about 5,000 feet of large diameter CIPP, HDD, and pipe bursting. Also reported about 33% cost savings on HDD water main replacement projects as compared with open-cut
- Dewatering can be a challenge during tunneling and microtunneling
- Trenchless methods avoid disrupting the public and business owners and reduces the carbon feet print of construction
- Infiltration can interfere with the curing process of CIPP liner
- Potential for mistakes in trenchless that can cause big problems

National survey respondents had several additional comments. A respondent from Texas expressed the opinion that engineers need more training in trenchless technologies. A respondent from Alaska said that they have recently started using pipe ramming as an
installation option. They said that embankments there frequently include cobbles and boulders, and pipe ramming has been an adequate solution.

Respondents to the Iowa survey were asked to list research and improvements that could be made to trenchless technologies to make them more feasible.

- More requirements on exact final location of the piping after installation
- Improved grade control for HDD
- Improved machine control and monitoring systems
- More certification programs for contractors
- Tightening specified tolerances
- Increase pipe types per application
- Provide the Road Agencies with a document showing possible problems with trenchless technology methods and how they can be solved.
- Develop a cost effective QC/QA to reduce risk

An interview with an Iowa HDD and auger boring contractor yielded a large amount of information about the trenchless construction industry and the problems they have commonly encountered in the field. The comments are organized in a bullet list.

- Experienced contractors need to be given the flexibility to use whatever methods they deem necessary to complete a job given the necessary specifications. Contractors’ practical experience can be a valuable design resource and engineers’ designs shouldn’t be too inflexible.
- Projects with large pipes are very difficult because large pipes displace so much soil.
- A hazard when auger boring in clay is the possibility of the drill teeth bunching up and moving a larger volume of clay. This affects the soil properties in a larger vicinity around the drill head.
- Projects in close proximity to creeks with rock and sand beds can be dangerous because of the possibility of encountering water.
The contractor receives engineers’ soils reports for most projects. They look for blow counts, water content, and location of water table. They interpret soil with blow counts of 1 or 2 to indicate a soil of “toothpaste” like consistency, blow counts of 10-25 to indicate reasonable soil, and blow counts of >50 to indicate rock.

The contractor stressed that the uncertainties in trenchless construction and the many variables make best practice design guidelines unreliable. Additionally, best practice guidelines add liability.

The contractor pointed out that simple human errors can cause problems in an otherwise well handled project.

Combination pilot tube and pipe ramming methods are expected to become more popular.

Ground penetrating radar has been used in projects, but not to locate boulders. Soils in Iowa are too dense to “see” very deeply with radar. In many locations in Florida, it is possible to “see” down to bedrock.

The contractor thought that the problem with horizontal soil test borings is the small borehole width.

The contractor stressed the difficulty in predicting heave and settlement. This organization has tunneled 1 foot deep with no settlement while at 20 feet deep they have gotten settlement.

The contractor pointed out that, “no two tunnels are alike”, stressing the design challenges.

Railroad companies do the most on-job testing. They have the most concern for the methods used, and they closely monitor railroad track elevations for heave and settlement. Railroad companies have opposed HDD because of bad experiences in which bentonite pressure built up during pull back and cause surface heave.

One city designer told researchers that a franchised utility company installed conduits approximately 4 inches to 6 inches in diameter at shallow depths. These were the biggest threat of surface heave and overlying pavement cracking.
Utility potholing is generally done at the contractor’s discretion, and the contractor is required to repair damage that results if something is hit, unless the obstacle was completely unmarked. The owner will occasionally direct the contractor to pothole if there are critical utilities in the area. Many times it is obvious that there is no conflict between the old and new utility pipe.

Many additional comments were collected from contractors on job sites, designers, and equipment vendors. These comments are provided below.

- During HDD work on the Iowa State University campus, the contractor told the research team that blue and gray clay leads to hard, slow drilling. Yellow clay and black soil would provide easier drilling and reaming.
- An HDD equipment vendor explained that “people can do everything right and still get heave”. The vendor also said that asphalt pavement will heave more easily than PCC pavement.
- An HDD sonde, or “beacon”, is located near the head of the drill string. An epoxy strip covers it to let the signal escape. Steel cannot be used because it would block the signal. The vendor said that clear transmission of the signal can become an issue when a lot of metal is present in the vicinity or if power lines are nearby.
- An HDD vendor said that the rule of thumb for the ratio of borehole diameter to product pipe outside diameter is 1.5 to 1. This is less than the 2 to 1 ratio that one contractor informed us that they use. Even smaller ratios are used for grade boring because of the importance of the pipe’s exact location in the borehole.
- The necessary drilling fluid requirements are estimated by doubling the annular space volume of the borehole. The formula for this becomes reamer diameter squared / 25 = gal/ft. Usually, a 1:1 bentonite to water ratio is used in sand because it is a challenge to stabilize a borehole in sandy material. This results in a viscosity of 65. For clayey soils, a 2 to 1 ratio of polymer to water is used to lubricate the equipment and to emulsify and suspend the clay cuttings.
An HDD vendor mentioned that there may be several causes for surface heave. The operator could be in a hurry and back ream too fast. Too much drilling fluid could be used. Or, the wrong drilling fluid mix could be used.

An HDD vendor also claimed that drill rigs are capable of grade accuracy to 0.1%. This makes them suitable for installing gravity flow pipe.

An HDD contractor said that the three most common causes of frac-out and heave were excessive speed, which could cause outrunning the drill fluid, using a machine that is too small to execute the pullback process correctly, and incorrect drill fluid. Lack of experience is a common cause for many errors, but even experienced contractors will make mistakes from time to time. Additionally, if fluid pressures are building up underground, they may be released by digging a pressure relief hole vertically into the ground. Common problems for contractors include neglect in locating existing utilities. Also of note is that sometimes contractors will hire independent soil testing laboratories to conduct investigations when owners have not done so. The need for doing this can be a factor in bidding.

An auger boring contractor stated that soil is unlikely to heave when boring through clay. Heave would occur only if the contractor pushed the casing too fast, and it compressed soil faster than it could be augered out. Except for the previously described circumstance, the bore would need to be very shallow before most experts would have a concern about heaving.

Additional soil testing could be useful in rocky and sandy soils.

HDD speed is about the same through clay and sand, but contractors must be more careful when working in sand. Rock drilling takes about 3 times longer in comparison to working in clay.

Often, one hour is required to haul spent drilling fluid to a disposal site.

There are no specific standards for potholing.

There is little benefit for conducting extra soil testing for typical HDD installations in familiar areas.

HDD installations through clay soils are twice as fast as those in sand. Steering is harder in sand as compared to clay.
Most common problems encountered: 1) Existing utilities, 2) Rock, 3) Water.
If borehole seals up, it is because there is not enough drilling fluid.
Drilling fluid selections errors are the most common source of frac-out/heave.
Drilling fluid selection is an imprecise science. In the field, a driller must judge it by its consistency to see if it appears to be correct.
Usually, the most important challenge is avoiding obstacles.
Additional soil testing is not necessary if potholing is conducted. During potholing the contractor can make an assessment of the soil.
Relief holes can be excavated to sandy bores to alleviate soil pressure.
In sands, contractors do not need clay inhibitor. Instead, more bentonite is required.
Frac-out can occur when a bore is too long.
Inappropriate drilling fluid mixtures are the biggest cause of frac-out and heave problems.
A second important cause of frac-out and heave is drilling too fast.
There should be at least one experienced member on each drill crew.
Soil testing efforts are useful to locate sand. Little testing is required for a bore through clay.
Boring through sand is slower as compared to clay.
Rule of thumb: stay 2 feet from any other utility.
Contractors should use their judgment as they decide whether or not to preream.
An HDD operator can tell when the machine is boring through sand because the drill head gets bound up when trying to turn in sand. Also, he can feel the grittiness. Gravel can be felt. Tree roots can be felt because the pressure builds up and releases as they are cut through.
Frac-out cannot occur in saturated ground. It usually occurs in August and September when the ground is dry. Using a reamer that is too small can cause frac-out. Drill fluid needs to have a thick consistency.
Increased soil testing in HDD is usually not necessary or economical.
Clay bores much faster than sand.
- Being stopped by existing utilities is the most common problem that impedes progress.

A vendor involved with sales and training for contractors in impact moling and pipe bursting provided the following comments on those methods.

- In moling, for each 1 inch of borehole diameter, 10 inches to 1 foot of soil cover is required.
- In pipe bursting, you need 2 feet of clearance between the pipe to be burst and the nearest other pipe.
- Not much pipe bursting is done in the Midwest, except in Minnesota.
- Moling is a method that is mostly used for gas.
- When moling is used for installations, it is easier to conduct pressure testing because an air compressor is already on-site.
- In saturated conditions, the mole can get stuck and “swim”.
- Moling requires a smaller pit than in comparison to HDD
- 2’x2’ pothole window can be dug in the middle of longer installations to verify line and grade.
- When moling is used for an installation, it is useful to know the water table elevations and the standard penetration test blow count.
- Studies in California have shown that moling creates soil pressure changes that are similar to those created by a passing semi-truck.
Do you work for a contractor, city, county, DOT, or consulting firm?

- Contractor
- City
- County
- DOT
- Consulting
- Specify your own value:

In what areas of Iowa or the U.S. (if any) have you used trenchless methods of construction?

What types of trenchless techniques have you experienced in practice?

- Horizontal Auger Boring
- Pipe Ramming
- Pipe Jacking
- Horizontal Directional Drilling
- Microtunneling
- Pipe Bursting
- Cured-in-Place Pipe
- Sliplining
- Localized Repairs
- Specify your own value:
Based on your experience, have you encountered any constructability problems with the trenchless construction methods that you checked in the last question? Please explain.

Have you seen pavement distresses or other problems as a result of using trenchless methods

- Yes
- No

If yes, were these pavement distress problems mainly caused by the difficult soil types at the site or by the unreliability of the trenchless technology techniques used?

What lessons can be learned from these observations?

(Please only answer if you have knowledge of trenchless projects in Iowa) Which trenchless techniques do you think are
most commonly used in Iowa?

☐ Horizontal Auger Boring
☐ Pipe Ramming
☐ Pipe Jacking
☐ Horizontal Directional Drilling
☐ Microtunneling
☐ Pipe Bursting
☐ Cured-in-Place Pipe
☐ Sliplining
☐ Localized Repairs
☐ Specify your own value:

(Please only answer if you have knowledge of trenchless projects in Iowa) Why do you think these methods are selected for use in Iowa?

(Please only answer if you have knowledge of trenchless projects in the U.S. excluding Iowa) Which trenchless techniques do you think are most commonly used in the U.S.?
Specify your own value:

(Please only answer if you have knowledge of trenchless projects in the US excluding Iowa) Why do you think these methods are selected for use in the U.S. (excluding Iowa)?

How much soil would you say is vertically displaced by trenchless methods of installation? (Please provide estimate of heave in inches for sands, clays, and gravels for a given trenchless method)

From a practical point of view, which one (or more) of these trenchless methods do you prefer?

- Horizontal Auger Boring
- Pipe Ramming
- Pipe Jacking
- Horizontal Directional Drilling
- Microtunneling
- Pipe Bursting
- Cured-in-Place Pipe
- Sliplining
- Localized Repairs
Specify your own value:

Based on your answer to the last question, please provide an explanation of why you prefer the method(s).

Which trenchless method do you find the least favorable?

- Horizontal Auger Boring
- Pipe Ramming
- Pipe Jacking
- Horizontal Directional Drilling
- Microtunneling
- Pipe Bursting
- Cured-in-Place Pipe
- Sliplining
- Localized Repairs
- Specify your own value:

Based on your answer to the last question, why do you find the method(s) least favorable?
What methods of soil investigation are currently being used prior to trenchless construction projects in clays, sands, rock, etc.? Also, what are the soil properties of interest?

Do you think that these current soil investigation methods are adequate? Why, or why not?

What QC/QA methods are currently being used for trenchless projects?
What trends do you see emerging in QC/QA for trenchless projects?

What research should be done to make trenchless methods more feasible?

If you may be willing to be contacted by the research team to be interviewed, or if you may be willing to participate in the project by allowing the research team to observe your projects' construction practices, please provide your contact information.
APPENDIX C: REGIONAL AND NATIONAL SURVEY

1. In what trenchless related field do you work?
   - City Consulting
   - Contracting
   - County DOT
   - Manufacturing/Sales
   Other (please specify)

2. In what areas of the US have you used trenchless methods?

3. What types of trenchless technologies have you experienced in practice?
   - Horizontal Auger Boring
   - Pipe Ramming
   - Pipe Jacking
   - Horizontal Directional Drilling
   - Microtunneling
   - Tunneling
   - Compaction Tools
   - Pipe Bursting
   - Sliplining
   - Cured-in-Place Pipe
   - Localized Repairs
   Other (please specify)
4. Have you encountered pavement deformations caused by trenchless methods?
- yes
- no

5. If yes, would you be willing to be contacted by the research team to be briefly interviewed by telephone to tell us about the circumstances of this project? If so, please provide the necessary contact information.

6. Do you feel current levels of QC/QA associated with trenchless projects are appropriate?
- yes
- no

7. If you would like to elaborate on your answer to question #6, please do so.

8. Please rate your view of the reliability of trenchless technology as a rehabilitation and construction solution.
- 1 - poor
- 2
- 3
- 4
- 5 – excellent

9. Please explain why you chose your answer to question #8.
10. Please share any additional comments you may have about trenchless technologies.
APPENDIX D: SOIL BORING LOGS FROM PAMMEL DRIVE HDD PROJECT
### LOG OF BORING NO. 1

**Owner:** Iowa State University  
**Site:** Ames, Iowa  
**Project:** Iowa State University Chemistry Building

<table>
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<tbody>
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<td>Number</td>
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<td>SS</td>
<td>HS</td>
</tr>
<tr>
<td>SS</td>
<td>HS</td>
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</table>

**DESCRIPTION:**
- Approx. Surface Elev.: 954.0 ft.
- Topsoil - Organic CLAY, trace sand and gravel, very dark brown
- Sandy loam CLAY, trace gravel and ferruginous laminae, olive brown, very stiff

- Becomes hard at 10.5'
- Ferruginous ends, color change to dark gray and becomes very stiff at 16'

**Water Level Observations:**
- Boring started: 6-5-07
- Boring completed: 6-6-07

**GEO-TECHNICAL INVESTIGATIONS:**
- Approved by TEAM Services, Inc.
- Job #: 1-2005

---

Calibrated Hand Penetrometer*
LOG OF BORING NO. 1

OWNER: IOWA STATE UNIVERSITY

ARCHITECT/ENGINEER: ISU PROJECT NO. CP1207

SITE: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG

DESCRIPTION

Sandy loam CLAY, trace gravel, dark gray, very stiff

---color change to dark olive gray and becomes hard @ 47

50.0 904.0

Bottom of Boring

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES. EDGEWELL, THE TRANSITION MAY BE GRADUAL.

WATER LEVEL OBSERVATIONS

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<th>WD</th>
<th>ADR</th>
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BORING INVESTIGATIONS

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<th>WL</th>
<th>Name</th>
<th>WD</th>
<th>ADR</th>
<th>24 hrs</th>
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BOARING STARTED: 6-8-07
BOARING COMPLETED: 6-10-07
RED: Rig 112
FOREMAN: MG

TEAM Services, Inc.

APPROVED: RED
JOB #: 1-2066
# LOG OF BORING NO. 2

**Owner:** Iowa State University  
**Architect/Engineer:** Iowa State University Chemistry Building  
**Site:** Ames, Iowa  
**Project:** Iowa State University Chemistry Building

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**Water Level Observations**  
**Boring Started:** 6-7-07  
**Boring Completed:** 6-27-07

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**Team Services, Inc.**  
**Geotechnical Investigations**  
**Approved by:** RED  
**Job #:** 1-2066
### LOG OF BORING NO. 2

**Owner:** Iowa State University  
**Site:** Ames, Iowa  
**Project:** Iowa State University Chemistry Building

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<td>16</td>
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<td>--color change to olive and becomes hard @ 47</td>
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<tr>
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<td>HS</td>
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<td>19</td>
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**Calibrated Hard Pendrometer**

**Water Level Observations**

- **Boring Started:** 6-7-07  
- **Boring Completed:** 6-17-07

**Team Services, Inc.**

**Approved:** RED  
**Job #:** 1-2066
### LOG OF BORING NO. 3

**graphic log**

**description**

Approx. Surface Elev.: 964.0 ft.

- Lean CLAY, with sand, trace organic matter, dark grayish brown
- Sandy lean CLAY, trace gravel, olive brown, stiff
- --trace ferruginous staining and become very stiff @ 7
- --color change to light olive brown @ 12
- --ferruginous staining ends and color change to dark gray @ 17

**samples**

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<tr>
<td>5</td>
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<td></td>
<td></td>
<td>12.2</td>
<td>124</td>
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<td>10</td>
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<td></td>
<td>16.0</td>
<td>119</td>
<td>4034 - 7000*</td>
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<tr>
<td>15</td>
<td>HS</td>
<td></td>
<td></td>
<td>18</td>
<td>16.3</td>
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<tr>
<td>20</td>
<td>HS</td>
<td></td>
<td></td>
<td>17</td>
<td>13.3</td>
<td>8000*</td>
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<tr>
<td>25</td>
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<td></td>
<td></td>
<td>16</td>
<td>14.1</td>
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<tr>
<td>30</td>
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<td></td>
<td>15</td>
<td>15.7</td>
<td>7000*</td>
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**water level observations**

<table>
<thead>
<tr>
<th>WL</th>
<th>M</th>
<th>Depth</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>49</td>
<td>M</td>
<td>249 ft</td>
<td>6-6-07</td>
</tr>
</tbody>
</table>

**team services, inc.**

**borings started: 6-6-07**

**approved: 1-2066**
LOG OF BORING NO. 3

OWNER: IOWA STATE UNIVERSITY

Site: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG

DESCRIPTION

Sandy loam CLAY, trace gravel, dark gray, very stiff

Color change to dark olive gray, with gravel and becomes hard @ 42'

Silt, trace sand, dark gray, hard

Bottom of Boring

DEPT (Ft.) | USCS SYMBOL | NUMBER | RECOVERY | BLOWS/FT. | MOISTURE % | DRY DENSITY | HAND PORE PRESSURE
---|---|---|---|---|---|---|---
47.0 | CL | 9 | SS | 15 | 15.1 | 907.0 |
45.0 | CL | 10 | SS | 23 | 9.1 | |
50.0 | ML | 11 | SS | 36 | 10.2 | 904.0 |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES. DIGITAL THE TRANSITION MAY BE GRADUAL.

WATER LEVEL OBSERVATIONS

WL | Depth | Name | 24 hrs
---|---|---|---
WL | | | 49
WL | | | GEOTECHNICAL INVESTIGATIONS

BOARING STARTED 6-8-07
BOARING COMPLETED 6-8-07

Approved RED JOB # 1-2066
LOG OF BORING NO. 4

Approx. Surface Elev.: 964.5 ft.

-2.0 Asphalt
1.0 Concrete
3.0 Fill—Lean CLAY, with sand, trace gravel, olive brown, very dark gray
5.0 Lean CLAY, trace sand and organic matter, very dark brown, stuff (brazed topsoil)
Sandy lean CLAY, trace gravel and ferrous staining, olive gray, stuff
--color change to dark greenish gray and becomes very stiff @ 8
--color change to olive brown @ 10'

-ferrous ends and color change to dark gray @ 17
### LOG OF BORING NO. 4

#### Iowa State University Chemistry Building

**SITE**
Ames, Iowa

**DESCRIPTION**
Sandy loam CLAY, trace gravel, dark gray, very stiff

<table>
<thead>
<tr>
<th>WL</th>
<th>Name</th>
<th>WDL</th>
<th>2M</th>
<th>24 hr</th>
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<tbody>
<tr>
<td>0</td>
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#### SAMPLES

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<tr>
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<th>UCSC SYMBOL</th>
<th>NUMBER</th>
<th>TYPE</th>
<th>RECOVERY</th>
<th>S PL BLOWS/FT</th>
<th>MOISTURE</th>
<th>DRY DENSITY</th>
<th>UNCOND. UNCONS.</th>
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<tbody>
<tr>
<td>11</td>
<td>CL</td>
<td>11</td>
<td>HS</td>
<td>14°</td>
<td>13</td>
<td>13.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>CL</td>
<td>12</td>
<td>HS</td>
<td>17°</td>
<td>17</td>
<td>15.4</td>
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<td></td>
</tr>
<tr>
<td>13</td>
<td>CL</td>
<td>13</td>
<td>HS</td>
<td>19°</td>
<td>21</td>
<td>14.8</td>
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**Bottom of Boring**

904.5

---

**WATER LEVEL OBSERVATIONS**

<table>
<thead>
<tr>
<th>BORING STARTED</th>
<th>6-7-07</th>
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<tbody>
<tr>
<td>BORING COMPLETED</td>
<td>6-12-07</td>
</tr>
<tr>
<td>RED</td>
<td>Rig 112</td>
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<tr>
<td>TEAM Services, Inc.</td>
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**GEO-TECHNICAL INVESTIGATIONS**

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<thead>
<tr>
<th>APPROVED</th>
<th>RED</th>
<th>JOB #</th>
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<tbody>
<tr>
<td></td>
<td>RED</td>
<td>1-2066</td>
</tr>
<tr>
<td>SAMPLE</td>
<td>DESCRIPTION</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>0-1</td>
<td>Crushed limestone</td>
<td></td>
</tr>
<tr>
<td>1-2</td>
<td>Sandy loam CLAY, trace gravel, very dark gray (buried topsoil)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandy loam CLAY, trace gravel and ferrous staining, grayish brown, stiff</td>
<td></td>
</tr>
<tr>
<td></td>
<td>--color change to olive brown and becomes very stiff @ 7</td>
<td></td>
</tr>
<tr>
<td>2-3</td>
<td>Ferrous ends and color change to dark gray @ 17</td>
<td></td>
</tr>
</tbody>
</table>

**Samples**

- **Type**: HS
- **Recovery**: 18 in.
- **SLS**: 800 lb.
- **Density**: 8000 lb/ft³
- **Moisture**: 13.1%
- **Index**: HS

---

**Tests**

- **Calibrated Hand Penetrometer**: 6.2 ft

**Water Level Observations**

- **Team Services, Inc.**
- **Bo. 112**
- **Job #: 1-2066**

**Logs**

- **Bo. 112**
- **Log NO. 5**
- **ISU Project # CP1207**
- **Iowa State University Chemistry Building**
- **Site**: Ames, Iowa
### LOG OF BORING NO. 5

**SITE**

Ames, Iowa

**PROJECT**

Iowa State University Chemistry Building

<table>
<thead>
<tr>
<th>GRAPHIC LOG</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sandy lean CLAY, trace gravel, dark gray, very stiff</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>SAMPLES</th>
<th>TESTS</th>
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</thead>
<tbody>
<tr>
<td>21&quot;</td>
<td>HS</td>
<td></td>
</tr>
<tr>
<td>19&quot;</td>
<td>HS</td>
<td></td>
</tr>
<tr>
<td>19&quot;</td>
<td>HS</td>
<td></td>
</tr>
<tr>
<td>18&quot;</td>
<td>HS</td>
<td></td>
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**WATER LEVEL OBSERVATIONS**

TEAM Services, Inc.

**MATERIALS**

**BORE STARTED** 6-7-07

**BORE COMPLETED** 05-27-07

**RED** Rig 112

504.5

Bottom of Boring

**CALIBRATED Hand Penetrometer**

**24 hrs**

**GEO-TECHNICAL INVESTIGATIONS**

**APPROVED** RED JOB # 1-2066