Identification of Practices, Design, Construction, and Repair Using Trenchless Technology

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Final Report
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16. 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. However, the use of trenchless technologies in Iowa and their effects on surrounding soil and nearby structures has not been adequately documented.

Surveys of and interviews with professionals working in trenchless-related industries in Iowa were conducted, and the results were 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 issues are an ongoing problem in the industry. Inadequate soil information and quality control/quality assurance (QC/QA) are partially to blame.

Fieldwork 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 tests were 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.

Soil testing showed that the installations were made in sandy clay or well-graded sand with silt and gravel. Pipes were installed primarily using horizontal directional drilling with pipe diameters from 3 to 12 inches. Pressure cell monitoring was conducted during the following construction phases: pilot bore, pre-reaming, and combined pipe pulling and reaming. The greatest increase in lateral earth pressure was 5.6 psi and was detected 2.1 feet from the centerline of the bore during a pilot hole operation in sandy lean clay. Measurements from 1.0 to 2.5 psi were common.

Comparisons were made between field measurements and analytical and finite element calculation methods.

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IDENTIFICATION OF PRACTICES, DESIGN, CONSTRUCTION, AND REPAIR USING TRENCHLESS TECHNOLOGY

Final Report
October 2010

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EXECUTIVE SUMMARY

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. However, the use of trenchless technologies in Iowa and their effects on surrounding soil and nearby structures has not been adequately documented.

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Soil testing showed that the installations were made in sandy clay or well-graded sand with silt and gravel. Pipes were installed primarily using horizontal directional drilling with pipe diameters from 3 to 12 inches. Pressure cell monitoring was conducted during the following construction phases: pilot bore, pre-reaming, and combined pipe pulling and reaming. The greatest increase in lateral earth pressure was 5.6 psi and was detected 2.1 feet from the centerline of the bore during a pilot hole operation in sandy lean clay. Measurements from 1.0 to 2.5 psi were common.

Comparisons were made between field measurements and analytical and finite element calculation methods.

CHAPTER 1. INTRODUCTION

1.1 Problem Statement

Trenchless technologies are a group of methods for constructing and rehabilitating underground utilities without using open-cut excavations. 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 installing new pipes. Many applications exist for trenchless construction methods (TCMs), which take advantage of the ability to install utility lines with minimal disruption of facilities above and next to the alignment of the new utility line. 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 different trenchless methods and to document current construction and quality control/quality assurance (QC/QA) practices used across Iowa and the United States. A testing program was implemented to address these objectives.

1.3 Methodology

This report 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, QC/QA, 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 insight into field experiences, mainly focusing 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 measuring 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 were analyzed and are discussed.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

Trenchless technologies can be defined as a group of methods for constructing and rehabilitating 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, and telecommunications and storm and sanitary sewers multiply beneath roads.

Open-cut methods of utility pipe installation involve excavating a trench along the proposed pipeline path and placing the pipe in the trench. Other names for this method are open-cut, open trench, utility cut, dig-and-install, dig-and-repair, or dig-and-replace. These 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, which can cause construction efforts 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 that 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 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 further pavement deterioration. Arudi et al. (2000) add that such problems directly influence the pavement integrity, life, aesthetic value, and drivers' safety. Bodocsi et al. (1995) quantify this observation by noting that new pavement should last between 15 and 20 years, but 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 that was 0.75 to 1.5 inches thick 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 replacing landscaping and pavement.

Because of the limitations of open-cut methods, the development of trenchless technologies has been encouraged. TCMs 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 (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
- Decreased disturbance to local residents and businesses
- Increased flexibility in alignment selection
- Increased flexibility in choosing depth of new installation, which may allow more favorable soil conditions to be used
- Less relationship between cost and depth of installation
- Reduced number of utility relocations
- Reduction in the amount of spoil that requires disposal
- Reduced 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 jobsite workers
- Ease of renewal of existing pipelines
- Mitigation of air, water, and noise pollution
- Reduced 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: TCMs and trenchless rehabilitation methods (TRMs).

Stidger (2002) states the belief that a lack of understanding regarding TCMs and TRMs and the perception that TCMs and TRMS will have higher costs seem to be the key reasons that open-cut methods are still commonly chosen for many projects. Considering that the focus of utility pipe work is shifting toward established urban areas with aging utility pipes (Thompson 1993), designers should consider TCMs and TRMs. 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 the average annual daily traffic 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 that is beyond the scope of this research.

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

This literature review intends to document the current TCMs and TRMs, 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 (TCMs)

TCMs 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 compared to open-cut methods.

Myers et al. (1999) estimated that 150,000 miles of new conduit is installed each year in North America. The distribution of new conduit construction by end use (see Figure 2.1) shows that the gas and telecommunication industries require the most.

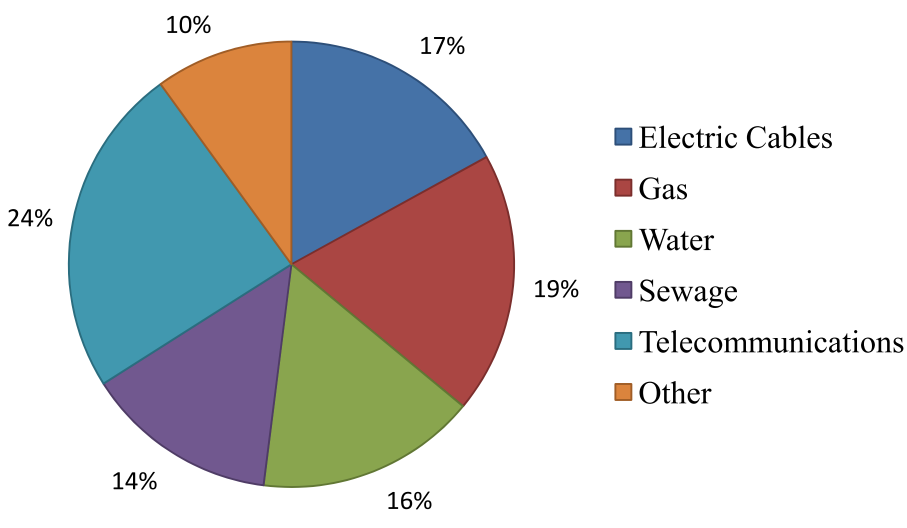


Figure 2.1. New conduit type breakdown (after Stein et al. 1999)

Numerous trenchless construction methods exist, and they are still evolving as experience and technology improve. Jung and Sinha (2007) reported that the following methods are most commonly used:

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

Table 2.1 shows important characteristics of the most common TCMs.

Table 2.1. Trenchless construction methods (after Najafi 2005)

Method	Diameter range (in)	Maximum installation (ft)	Pipe material	Applications	Accuracy of Installation
Pipe jacking and conventional tunneling	≥42	1,500	RCP*, GRP*, steel	Pressure and gravity pipe	± 1 in
Auger boring	4–60	600	Steel	Road and rail crossing	±1% of the bore length
Microtunneling	10–136	500–1,500	RCP, GRP, VCP*, DIP*, Steel, PCP*	Gravity pipe	± 1 in
Mini-HDD	2–12	600	PE, steel, PVC*, clay, FRP*	Pressure pipe/cable	Varies
Midi-HDD	12–24	1,000	PE, steel, ductile iron	Pressure pipe	Varies
Maxi-HDD	24–48	6,000	PE, steel	Pressure pipe	Varies
Pipe ramming	<120	400	Steel	Road and rail crossing	Dependent on setup
Compaction methods	<8	250	Any	Pipe or cable	±1% of the bore length
PTMT	6–10	300	RCP, GRP, VCP, DIP, Steel, PCP	Small diameter gravity pipes	± 1 in

*Abbreviations: PE= Polyethylene pipe, RCP = Reinforced concrete pipe, GRP = Glass fiber reinforced polyester, VCP = Vitrified clay pipe, DIP = Ductile iron pipe, PCP = Polymer concrete pipe, PVC = Polyvinyl chloride pipe, and FRP = Fiberglass reinforced polyester

Pipe diameter, length of bore, pipe material, type of utility, and ground conditions affect the construction method chosen. The location of the bore relative to the water table and the type of soil 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 TCMs and provides general guidelines for the suitability of common trenchless techniques in different soil types. Table 2.2 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 TCMs.

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

Ground Conditions	Guided Boring	Auger Boring	Pipe Ramming	HDD	Pipe Jacking
Soft to very soft clays, silts, and organic deposits	Y	Y	Y	Y	M
Medium to very stiff clays and silts	Y	Y	Y	Y	Y
Hard clays and highly weathered shales	Y	M	Y	Y	Y
Very loose to loose sands above water table	M	Y	M	Y	M
Medium to dense sands below the water table	N	N	Y	Y	N
Medium to dense sands above the water table	Y	Y	Y	Y	Y
Gravels and cobbles less than 50-100 mm diameter	Y	Y	M	M	Y
Soils with significant cobbles, boulders, and obstructions larger than 100-150 mm diameter	M	Y	M	N	M
Weathered rocks, marls, chalks, and firmly cemented soils	Y	M	Y	Y	M
Slightly weathered to unweathered rocks	Y	M	M	M	N
Yes	Generally suitable when installed by an experienced contractor with adequate equipment				
Marginal	Difficulties may occur; some modifications of equipment or procedure may be required				
No	Substantial problems will likely occur if this method is used; generally unsuitable or unintended for these conditions.				

2.2.1 Horizontal Auger Boring and Guided Auger Boring

The American Society of Civil Engineers (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, according to the North American Society for Trenchless Technology (NASTT), is also one of the most cost-effective (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 to 72 inches in diameter for a length averaging between 175 and 225 feet with a maximum distance of 600 feet (ASCE 2004). There is no limit for the potential installation depth. 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).

For highway and railroad crossings, casing will often be installed using trenchless methods, and then the product pipe (the actual utility pipe) will be installed in the casing. This technique prevents leaks in the product pipe from causing damage to the roadbed.

The auger is a flighted tube that 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 that moves forward on a track in the launching pit. 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, moves to the back of the launching pit, and a new segment of auger and pipe is 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 may be grouted. (Kenny et al. 2003)

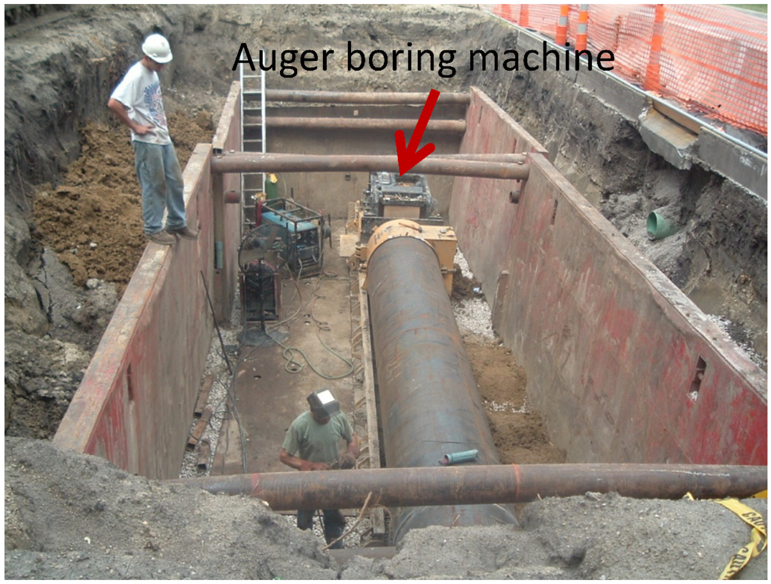


Figure 2.2. Launching pit with auger boring machine installing pipe

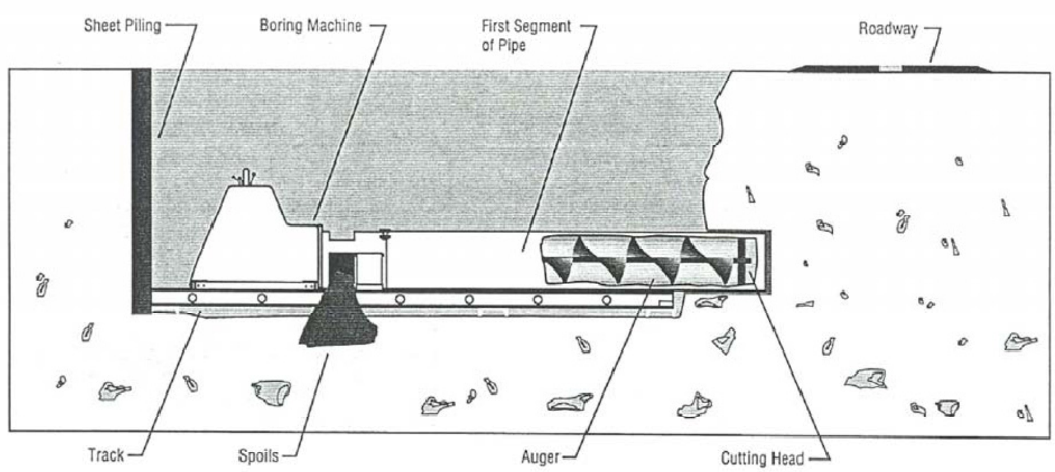


Figure 2.3. Typical auger boring system (Iseley and Gokhale 1997)

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 operation requires constructing 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). The first small pipe is used for a basic waterline system to measure grade. The second small pipe is a fluid supply line that can be used in case it becomes necessary to deliver grout or water to the cutting face. Water may be needed if sticky clay is encountered, and grout may 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 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 top of casing

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, which 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 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 is connected one by one to the casing and jacked into place, while the auger and casing sections are simultaneously removed at the retrieval pit (see Figure 2.5).

Figure 2.5. Vitrified clay pipe being installed using guided auger boring (from Allen Watson, Ltd.)

Guided auger boring is not recommended for use in soils with boulders because the pilot tubes may be deflected, and guided boring is not recommended for 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 and guided auger boring is available in ASCE's *Horizontal Auger Boring Projects* manual of practice (ASCE 2004).

2.2.2 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, though 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 the machine is steadily advanced into the borehole by a hydraulic jack positioned behind the borehole 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, including cutter head rotation, jacking rate, steering, and spoil removal, are controlled by an operator seated inside the machine. 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 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

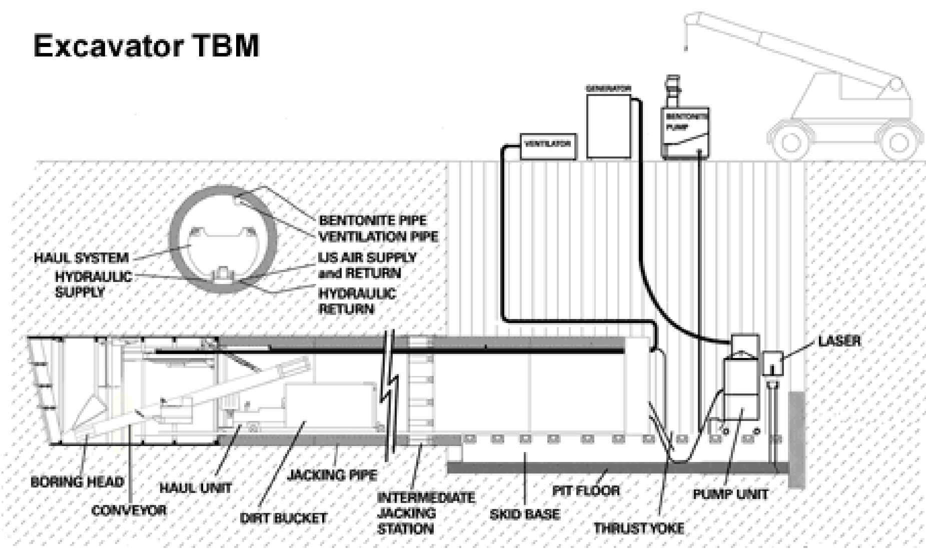


Figure 2.7. Utility tunneling machine diagram (Miller the Driller)

MTBMs are mostly used for installing gravity pipelines, such as for a sanitary or storm sewer. ASCE's *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 to 136 inches or more and have virtually no depth limitation. A cross-section of a microtunneling procedure is shown in Figure 2.9.

Figure 2.8. MTMB with disc cutter head on display (photo by Charles T. Jahren)

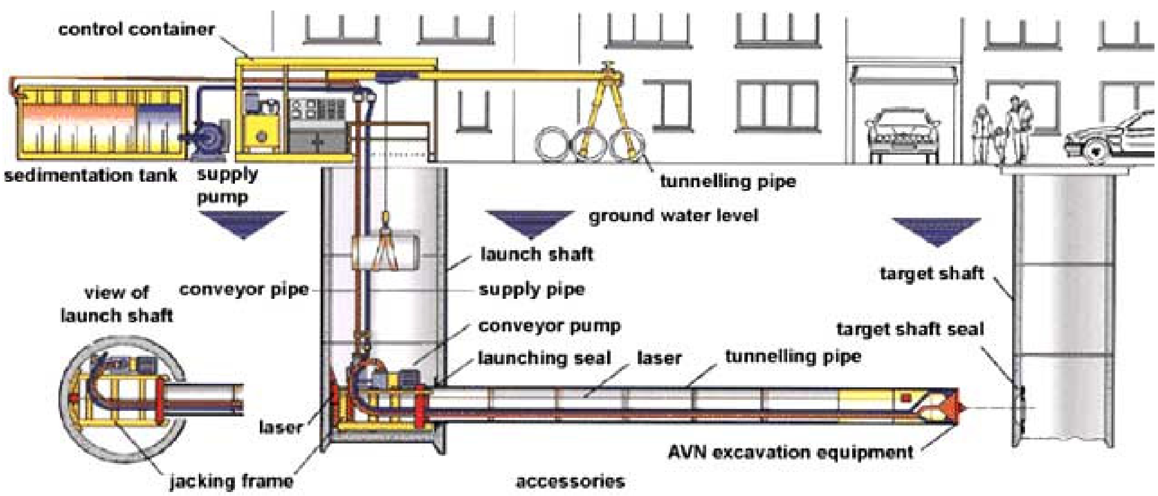


Figure 2.9. Microtunneling in urban setting (Irish Tunneling, Ltd.)

When using either type of tunneling method, the most critical risk 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). For example, the presence of boulders and saturated sandy soils can be particularly 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 that result in surface settlement. The prevention of inflow by dewatering 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 with groundwater and flowing, highly permeable soils. Conversely, the open face of TBMs can make these machines unsuitable for flowing soil and water conditions. TBMs can be fitted with a closed-face shield that allows groundwater inflow to be controlled in certain soils, such as low-permeability clays. Even in these 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 soil hydrofracture. 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 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 them to the relative advantages of both methods before making a selection. Table 2.3 compares relative differences between TBM and MTBM technologies.

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

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

2.2.3 Horizontal Directional Drilling (HDD)

HDD is a TCM that utilizes 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 repairing and replacing 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 to 48 inches in diameter 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. However, HDD is not effective for soils with a significant number of cobbles and boulders 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 being a traditional drill rig for vertical drilling that is turned on its side (Neu 2004). Complete HDD systems usually include 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° 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, which 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. Information was not available about the amount of soil displaced and the amount compacted into the sides of the borehole. 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. Pilot bore being started by pushing drill bit into soil

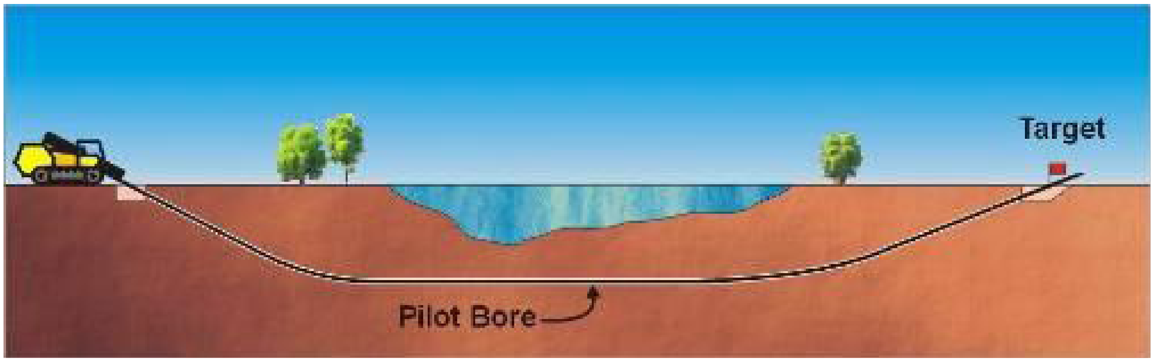


Figure 2.12. Pilot bore diagram (from NASTT)

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 located behind the drill bit. Sometimes, plain water is used for bores of 50 feet or less and in 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 pass or in several passes with reamers of progressively larger diameters. The product pipe will be attached to the reamer before the final pullback to complete the installation of the pipe (see Figure 2.14). Typically, product pipe pullback is the operation that causes the highest pullback stresses, due to the friction between the product pipe and the wall of the borehole. (Najafi, 2005)

Figure 2.13. Reamer with 14-inch-diameter

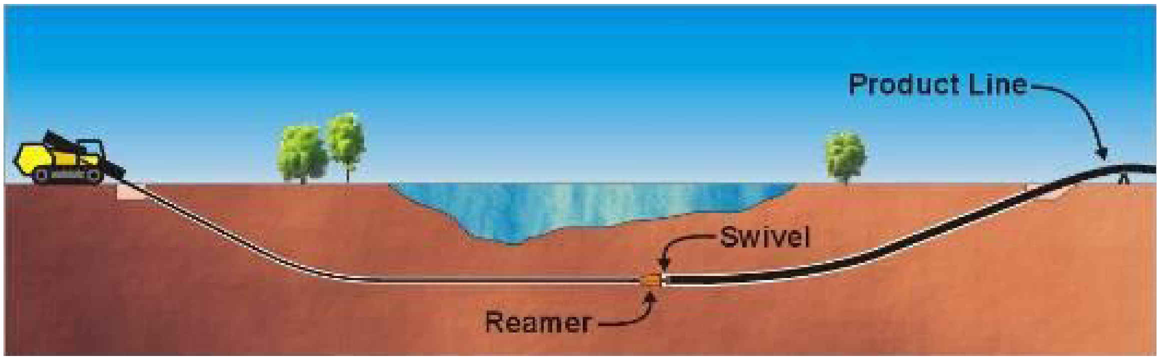


Figure 2.14. Reaming and product line installation (from NASTT)

Project specifications require a weak-link device to be attached between the reamer and product pipe during installation. A weak-link protects the pipe from excessive tensile stresses. Newer equipment is also available, such as the “TensiTrak” device, that measures the pullback forces and reports the results to the operator (see Figure 2.15). American Society for Testing and Materials (ASTM) pipe standards are used to determine the allowable tensile load for setting weak-link devices.

Figure 2.15. Weak-link device (trade name “TensiTrak”) (NASTT 2005)

A slant-head drill bit, such as the one shown 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 because it helps create 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 is optimal for harder soil conditions, such as stiff clays. A fly cutter reamer is used for even 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 if improper drilling fluid is used. A spiral reamer is used for loose and stony soil. Each type of drill bit and reamer contains small nozzles through which a continuous flow of drilling fluid washes 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 used to predict pull loads for large, expensive installations are based on assumptions of ideal borehole conditions. Specifically, these assumptions are a clean, stable borehole filled with low-viscosity drilling mud.

2.2.4 Pipe Jacking

The term "pipe jacking" may be used to describe either a specific TCM 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 different 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 different trenchless methods that use a jacking mechanism to advance the pipe and cutter head.

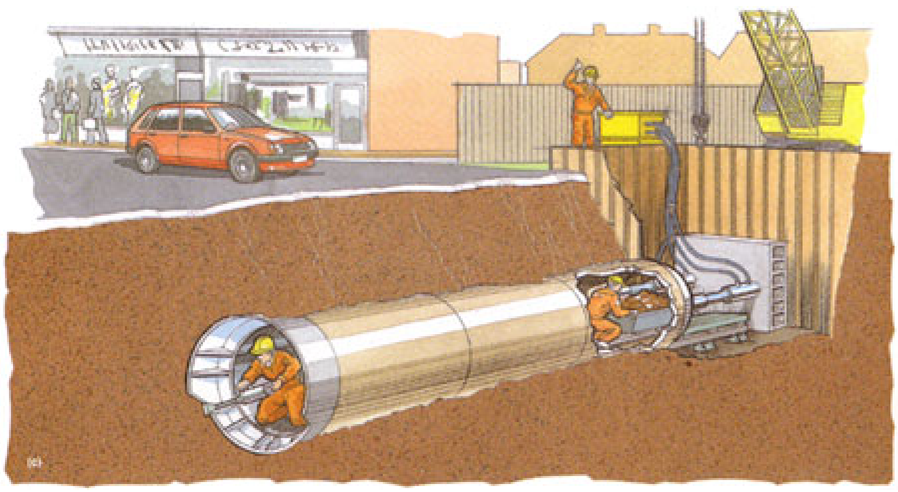


Figure 2.16. Pipe jacking with manual soil excavation (WK Construction)

Pipe jacking is used to install pipe that is greater than 42 inches in diameter and for lengths up to 1,500 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).

2.2.5 Pipe Ramming

Pipe ramming is a trenchless construction procedure that involves pneumatically hammering a steel pipe into the soil formation (see Figure 2.17). The leading edge of the pipe can either be closed with a cone tip or be 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 radially displaced during installation, resulting in significantly increased soil pressures on all sides of the pipe and increased risk of surface heave.

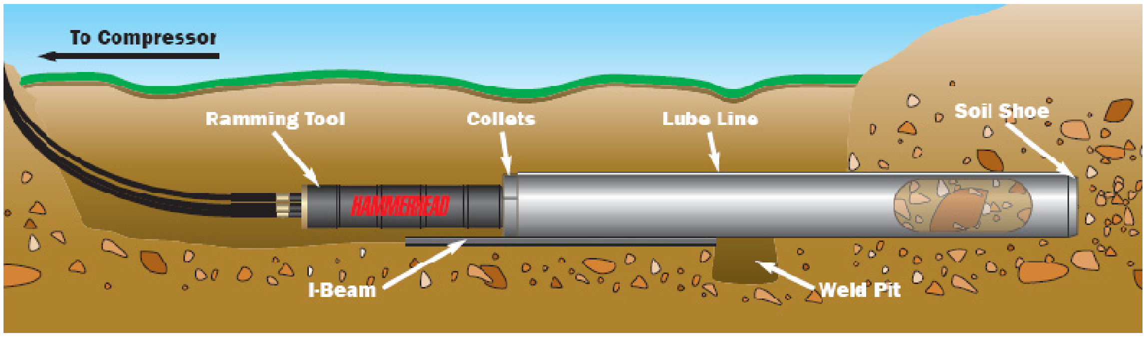


Figure 2.17. Pipe ramming diagram (Earth Tool Company, LLC)

Open-faced pipe ramming is usually used for pipes with diameters up to 55 inches and lengths up to 150 feet long. Pipes should be installation 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. For short bores of under 60 feet, the method can allow more cost savings than auger boring and HDD due to the faster set up times and faster installation times. Pipe ramming can be used in nearly all soil types except for solid rock. However, this method can be unsuitable at depths below the water table, especially in sands, because groundwater can flow through the pipe and enter the insertion pit. A drilling fluid similar to that used for HDD installations is used 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).

2.2.6 Compaction Methods (Impact Moling)

Impact moling is a TCM that uses a pneumatic mole to bore a small diameter hole. Impact moling is used to install pipes with a diameter of up to 10 inches and a length of up to 200 feet. Pipes should be installed at a depth of at least 10 times the diameter of the product pipe or 3 to 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 that resist deformation and unsuitable for loose sands and gravels because of the potential for the borehole to collapse and because rocky soils can cause the mole to deflect from its course (Clarke 2004).

Figure 2.18. 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 an operator slowly eases it 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 proceeds 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.

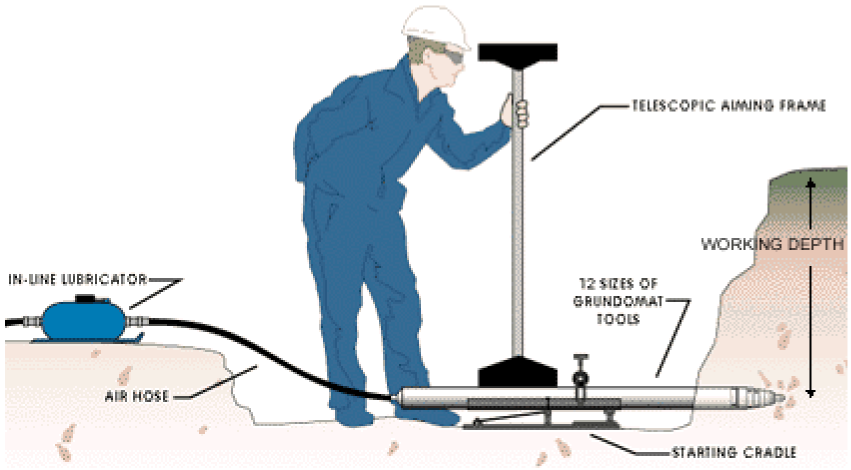


Figure 2.19. Positioning mole before launching (Allen Watson, Ltd.)

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).

2.2.7 Pilot Tube Microtunneling (PTMT)

PTMT was first introduced in the United States in 1995 and has been becoming increasingly popular as an alternative to microtunneling. PTMT is used for installing small diameter pipes that require high accuracy. This method can be considered a hybrid of three existing trenchless boring methods. Similar to HDD, a pilot bore head with a slanted face is used (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).

Figure 2.20. PTMT steering heads (Purdue University)

PTMT has been growing in popularity due to low equipment costs, a small surface footprint, accuracy, and small launching pits. Pipe 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 as additional thrust is available from more powerful hydraulic jacking systems. PTMT is most effective in soft soil conditions and is not considered suitable for soil with significant cobbles and boulders because these obstacles can impact steering. PTMT can be used above or below the water table (Najafi 2005).

A typical PTMT project begins with the excavation of circular jacking and receiving pits, which usually measure 6.5 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 because 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; Force et al. 2003).

Figure 2.21. PTMT reamer (Purdue University)

2.3 Trenchless Rehabilitation Methods (TRM)

TRMs 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 TRMs 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. Inline 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 nonstructural 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 TRMs.

Table 2.4. Trenchless rehabilitation methods (Najafi 2005)

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

Table 2.4. Trenchless rehabilitation methods (continued)

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

Note that method abbreviations are defined above the table.
*Material abbreviations: PE = Polyethylene, PP = Polypropylene, PVC = Polyvinyl chloride, GRP = Glass fiber reinforced polyester, EPDM = Ethylene propylene diene monomer, HDPE = High density polyethylene, MDPE = Medium density polyethylene

2.3.1 Cured-in-Place Pipe (CIPP)

The CIPP renewal procedure involves inserting a resin-impregnated fabric tube into an existing damaged pipe through 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 used to initiate pipe curing (Bonanotte and Kampbell 2004).

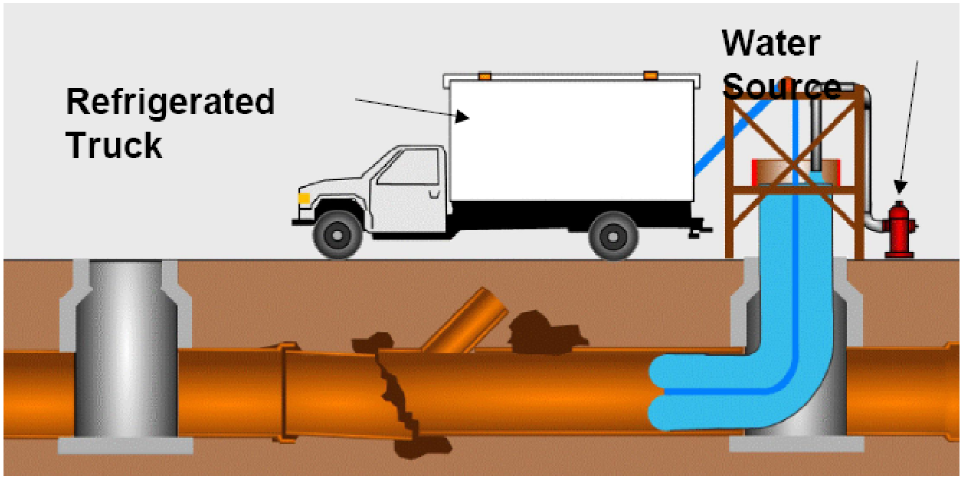


Figure 2.22. Water inversion–installed CIPP (NASTT)

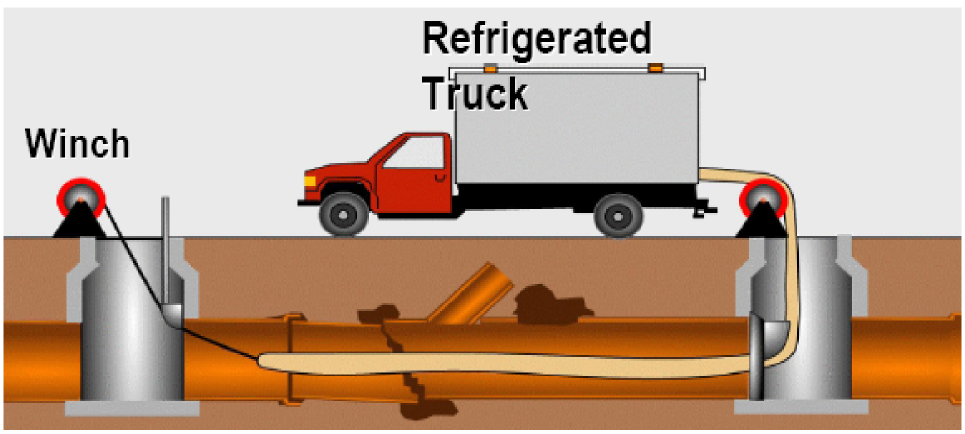


Figure 2.23. Winch-installed CIPP (NASTT)

CIPP can be used for structural or nonstructural 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 be easily 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. CIPP's wall is composed of two to three layers, depending on the installation method. The direct inversion installation method has 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. Pull-in-place installations and projects requiring a pre-liner have an addition outer layer of plastic film. (Kampbell and Whittle 2003)

2.3.2 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 to improve 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 commonly used because 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 because of 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 severely corrosive environments. Polymers possess the flexibility to be formulated for both structural and nonstructural use. Polymer linings are also often used to coat cement mortar linings in chemically severe environments.

Polyvinyl chloride (PVC) or polyethylene (PE) 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 in combination with poured-in-place concrete are now used for new construction. Najafi (2005) states that systems have been developed recently in which these sheet liners are applied to urethane mastic coatings to create a composite polymer coating-sheet lining system.

2.3.3 Sliplining (SL)

SLis a technique where 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 this method is also relatively inexpensive, which makes SL an effective measure when viable.

2.3.4 Modified Sliplining (MSL)

MSL describes 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.

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

Formed-in-place pipe is often used for renewing wastewater and storm water pipes 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 high-density polyethylene (HDPE) are installed against the walls of the pipe, and the annular area between the sheets is grouted, forming a new pipe (Najafi 2005).

2.3.5 In-Line Replacement (ILR)

ILR is an option for pipeline renewal if the current capacity of the pipeline is inadequate. Water, wastewater, and gas pipelines can be replaced using this method. Nearly all types of pipe—including concrete, clay, steel, ductile iron, cast iron, asbestos, PVC, and PE—can be replaced. ILR techniques can be subdivided 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 is also referred to as “pipe eating.”

Pipe bursting is usually carried out in increments of 300 to 400 feet, roughly corresponding to the distance between manholes. Pipes up to 48 inches in diameter have been burst, and method improvements are making it possible to burst larger diameters. Pipe bursting is not appropriate for use in expansive soils, near other underground structures, near 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 that a distance of 2 feet rather than 2.5 feet from the bursting head to other pipe or structures 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 advanced through the old pipe, a new pipe, usually PE, is 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, a 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.

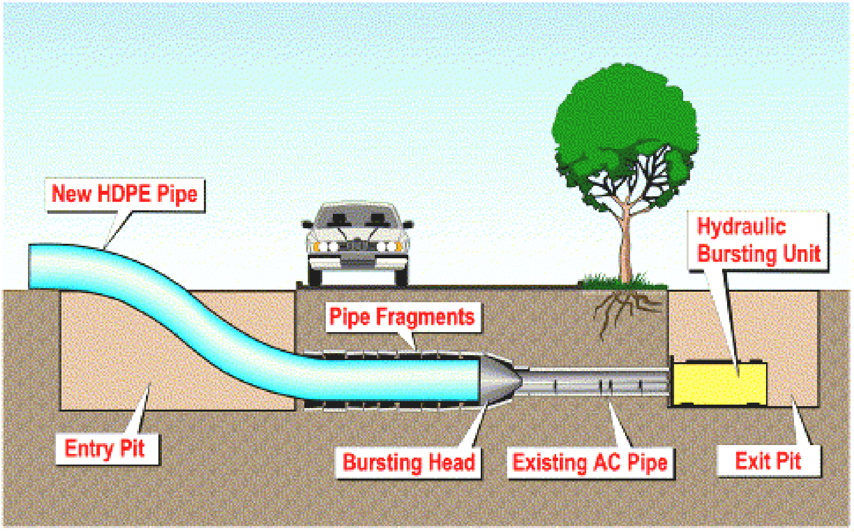


Figure 2.24. Pipe bursting (Dayton & Knight, Ltd.)

Pipe splitting is a variation of pipe bursting 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).

Figure 2.25. Pipe splitting (RJM Company)

Pipe eating, the other branch of ILR, 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 remotely controlled with a laser guidance system.

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

2.3.6 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)

Figure 2.26. Close-fit pipe (Insituform Technologies Inc.)

2.3.7 Localized Repair (LOR)

LOR, 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, normally referred to as grout, into defects ranging from 4 to 24 inches in diameter (Bauhan et al. 1997). Grouting is used to solve the following basic problems (Najafi 2005):

- Connecting fragmentary pieces of unreinforced pipe
- Providing additional localized structural capacity
- Sealing cracks in the pipe to prevent infiltration and exfiltration
- Replacing missing pipe sections

2.3.8 Thermoformed Pipe (ThP)

ThP has been used extensively in the United States since 1988, with over 21 million feet installed by 2003 (Najafi 2005). ThP can be used for sewer systems, water mains, and gas lines.

Structural and nonstructural uses are possible. Advantages of ThP 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 adding heat in order to fit tightly inside the existing pipe. Three methods of thermoforming exist:

- Fold and formed
- Deformed and reformed
- Fused and expanded

The fold and formed method involves flattening PVC pipe in the factory during production and folding it 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 ThP includes HDPE pipe that is 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 it back to its original round shape and pressurized to expand it 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).

2.3.9 Lateral Renewal (LR)

The majority of all wastewater pipe leaks occur from service laterals (Kiest Jr. and Flanery 2003). LR can be used to repair and renew sanitary sewer service laterals with the same methods used for main pipelines, which include CIPP, CFP, 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 and up to a maximum of 100 feet in length. (Najafi 2005)

2.3.10 Sewer Manhole Renewal (SMR)

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

2.4 Soil Investigation Methods

The complexity and limited access to the soil/boring tool interface make TCMs 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 selecting 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 about 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, which are closely coordinated, progress from planning to investigating to reporting. Figure 2.27 shows a proposed iterative approach of the three phases with possible inputs and outputs (Richardson et al. 2003).

Input	Design Phase	Output
project requirements/ preliminary project information; desk study; case history search; existing geotechnical reports/boring logs search/review	→ PLANNING →	preliminary tunnel profile and alignment; anticipated ground conditions; lessons learned (projects in similar geological environments); generalized soil/rock properties; generalized groundwater elevation data; geotechnical investigation planning document preparation
preliminary geotechnical investigation/borings; preliminary laboratory testing (soil/rock interface)	→ PRELIMINARY →	preliminary geological section; preliminary material properties (soil/rock); preliminary design parameters; preliminary tunnel profile and alignment adjustment
additional borings (as required); additional laboratory testing; owners review/comment/risk management	→ FINAL →	final tunnel profile and alignment; preliminary material properties (soil/rock); final design parameters; geological/construction risk evaluation; geotechnical baseline report

Figure 2.27. Suggested iterative approach for trenchless projects (from Richardson et al. 2003)

The planning stage of a trenchless construction project requires developing 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, which could potentially 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 pre-design surface survey should include the following elements:

- Work area requirements
- Existing grade elevation data
- Surface features, such as roadways, sidewalks, and 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 locate the utility. 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, 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:

- Determine the nature of soil at the site and its stratification
- Obtain disturbed and undisturbed soil samples for visual identification and laboratory tests
- Determine the depth and nature of bedrock if encountered
- Perform in situ field tests
- Observe surface drainage conditions from and into the site
- Assess any special construction problems with respect to the nearby existing structures
- Determine 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 following as main methods of geotechnical surveys:

- *Ground-penetrating radar.* Effective in gravels and sands.
- *Acoustic (sonar).* Useful for determining rock depth, 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 water table depth and of the bedrock.

- *Test pits or trenches.* This method is only suitable for shallow depths but allows visual observation over a larger area than is possible with samples from borings.
- *Hand augers.* Only suitable for shallow depths; only disturbed or mixed soil samples can be obtained with this method.
- *Boring test holes and sampling with drill rigs.* This method is the principal method for detailed soil investigations. Sampling intervals and techniques should be set to accurately describe the subsurface material characteristics, accounting for site-specific conditions. Typically, split-spoon samples will be taken in soft soil at depth intervals of 5 feet 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 samples, contact sensing probes, and borehole geophysical tools capable of providing horizontally continuous geotechnical information. These devices are usually advanced into the ground using HDD technology.

Allouche et al. (2001) states that a site characterization project that involves horizontal boring is 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 HDD industry. Allouche et al. (2001) also presents a methodology for selecting and deploying 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)

Name	Description	Comments
Multiple-port soil sampler	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.	Deployed in a pre-drilled continuous borehole. Suitable for cohesive and unconsolidated soils. Capable of collecting six samples and deploying miniature CPT.
Ditch-Witch soil sampler	After the pilot bore is completed, the device is advanced into the borehole, following the original alignment. Soil samples collected are of relatively low quality. Length: 0.5 m; diameter: 62 mm.	Suitable for soft to firm cohesive and unconsolidated soils. In soft ground, sampler may deviate from path of pilot bore.
Punch-Master 2000	After the pilot bore is completed, the device is advanced into the borehole, following the original alignment. Operates similarly to a Shelby tube. Samples collected of high quality. Sampler: length: 7 m; diameter: 120 mm.	Can only be used with medium or large size rigs. Suitable for a wide range of soil conditions.
Devico continuous rock coring method	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.	Involves specialty equipment and thus is a relatively costly method. Suitable only for rock formation.
Microdrilling	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.	Currently under development by the U.S. Department of Energy/oil and gas exploration industry.
Horizontal pre-bore cone penetration	Cone is pulled through a 100 mm diameter pre-bored hole. Data related to tip resistance and sleeve friction are transmitted to surface. Length: 1.0 m; diameter: 150 mm.	Susceptible to the disturbance of the formation caused by the drilling operation and drilling fluid.

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 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 statement 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 project data and GBR and include it in a separate design report that is excluded from the contract 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 conducted because of the difficulty of quantifying the benefits of a given investment level in site characterization, which 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 for trenchless methods. TCMs 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 mainly occurs as a result of soil loss during tunneling and because of dewatering operations that lead to subsidence. During a trenchless technology project, soil loss may be associated with soil squeezing, fluid running or flowing into the heading, 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 operating and steering the machine. If passive earth pressure is exceeded, ground surface heave may occur, causing damage to nearby utilities and other structures.

HDD is particularly susceptible to subsurface deformations because the method uses drilling fluid and some radial soil displacement is present. Researchers consider the primary mitigation tools to be allowable drilling pressures and ground improvement protections. 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, such as those outlined in Marshall and Knight (2003) and Hunter (2005), exist to try to model this action using cavity expansion theory.

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 for determining the proper driving force that will not lead to surface deformations (Shou and Chang 2006).

TRMs are considered to have little to no effect on the existing soil, with the exceptions of pipe bursting and pipe splitting. Both of these methods 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. (2005) show that an elliptical soil expansion 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 compared to the complexity of 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 caused 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 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 QC/QA 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 by adopting 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)

Risk	Cause(s)	Potential consequence(s)	Product type
Permanent deformation	Excessive pulling force during pullback	Reduced mechanical strength, reduced useful life	MDPE, HDPE
Rupture	Excessive pulling force during pullback	Failure of installation, loss of borehole, loss of pipe product	MDPE, HDPE, PVC
Scratching and denting	Sharp stones or other objects projected into borehole	Reduce pressure rating, initiation of crack propagation	MDPE, HDPE, PVC
		Corrosion	Steel
Kinks	Failure to maintain minimum bending radius/presence of large obstacles along bore	Onset of local buckling	Steel
		Reduce ovality	MDPE, HDPE
Failure of joint	Excessive bending	Failure of installation, loss of product	PVC
Damaging an existing utility	Poor as-built information, lack of/unsatisfactory locate, poor drilling practices	Injuries to crew/passersby, cost of repair itself, project downtime, secondary damage, downtime cost for utility provider	All pipes and conduits
Frac-out	Poor soil conditions, poor drill path planning, poor drilling practices	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	All installations
Surface heave	Insufficient burial cover, excessive pilot drilling or reaming rates, failure to use sufficient volume of drilling fluids, borehole enlargement increment too large	Cracked roads and driveways, heaved sidewalks and pedestals, damage to adjacent utilities and foundations	All installations
Failure of installation to meet technical requirements	Poor drilling practices, inadequate soil investigation, improper selection of pipe product, adverse soil conditions	Failure to complete installation, failure to exit borehole within acceptable window, failure to maintain grade and alignment within pre-specified tolerance	All installations

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 because of the lack of availability of monitoring technology.

Quality control in trenchless rehabilitation projects tends to be simpler and mainly consists 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 the widely used close-fit liners.

Because CIPP 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-and-form and deformed/reformed liners, quality control in the field is simplified 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 two years for older pipes and every three to four 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 have been developed to assist asset managers and construction practitioners in assessing the strengths and weaknesses of various 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. Allouche and Parhami (2003) give a brief comparison of various method evaluation models. The successful design of a trenchless construction project requires the following steps (Najafi 2005):

- Identify the requirements for the new pipeline
- Conduct surface and subsurface investigations
- Identify feasible trenchless technology alignments
- Select of an appropriate TCM
- Implement and model

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

- Identify pipe conditions and recognize and classify problems
- Prioritize problem-considering strategies and long-term plans
- Select an appropriate pipeline renewal method
- Design renewal methods based on project-specific conditions
- Implement and model

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 the project is a rehabilitation project. A method can then be selected 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 about 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.

CHAPTER 3. TRENCHLESS TECHNOLOGIES SURVEYS 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 four major topics:

- 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-three responses were received; the respondents' professional backgrounds are shown graphically in Figure 3.1. The majority of respondents were from design fields, with the largest specific groups being city and consulting personnel.

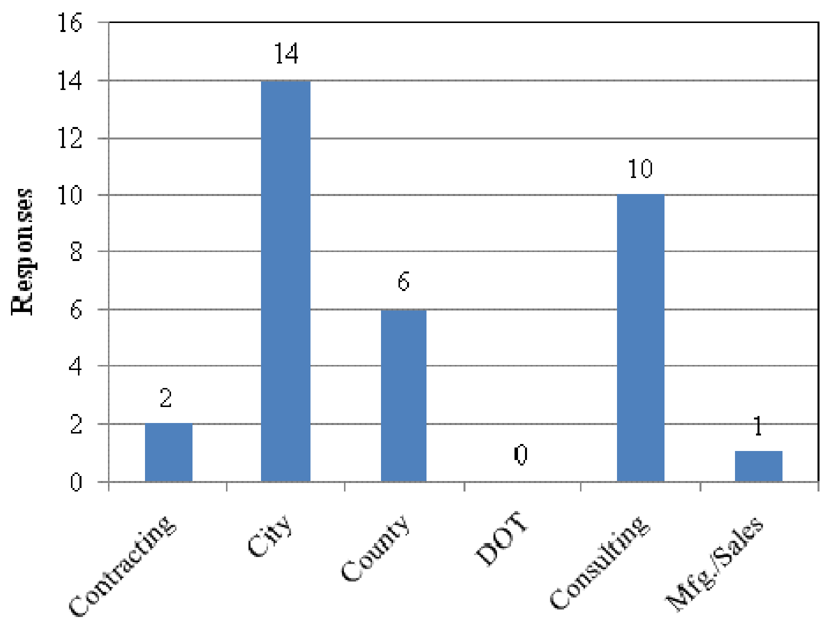


Figure 3.1. Trenchless-related backgrounds of respondents to Iowa survey

A second survey was released to a wider group that included professionals working in the Midwestern 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 (33 and 32, respectively) makes the comparison of results more useful. Observations of the Midwest survey show that a higher number of contractors took part in this survey, while fewer city workers and roughly the same number of consulting workers responded compared to the respondents from the Iowa survey. Overall, it would appear that fewer designers and more contracting and manufacturing/sales personnel responded to the Midwest survey compared to the Iowa survey. This slightly different demographic sample may have resulted in differing responses in the two surveys.

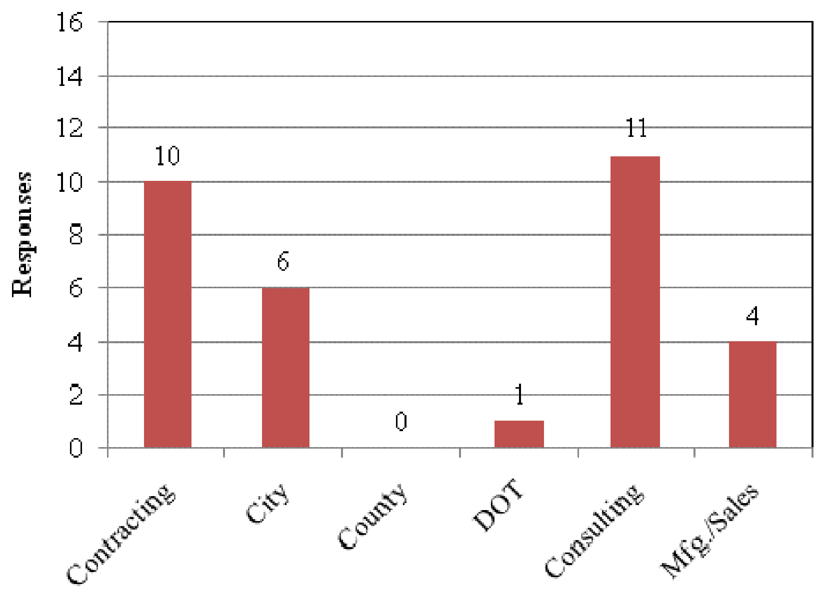


Figure 3.2. Trenchless-related backgrounds of respondents to the Midwest survey

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 (see Figure 3.3). The geographical distribution of the national survey is shown in Figure 3.4.

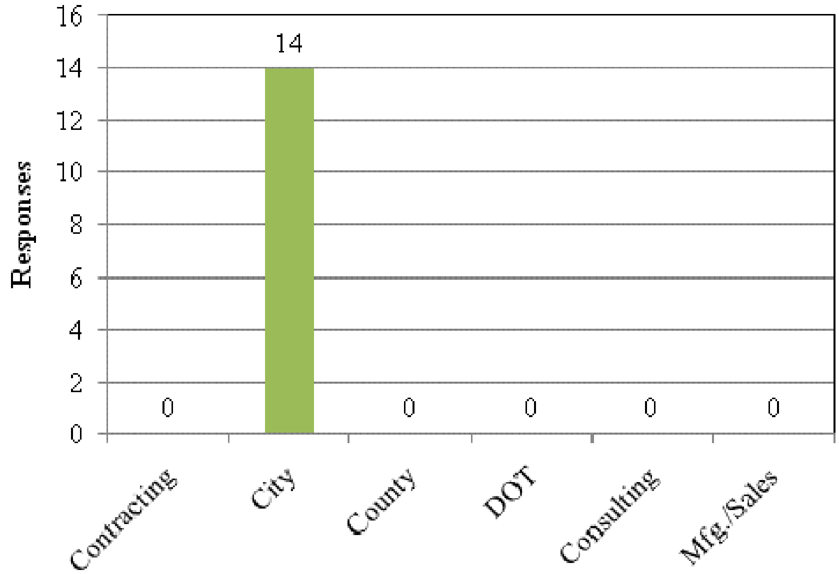


Figure 3.3. Trenchless-related backgrounds of respondents to the national survey

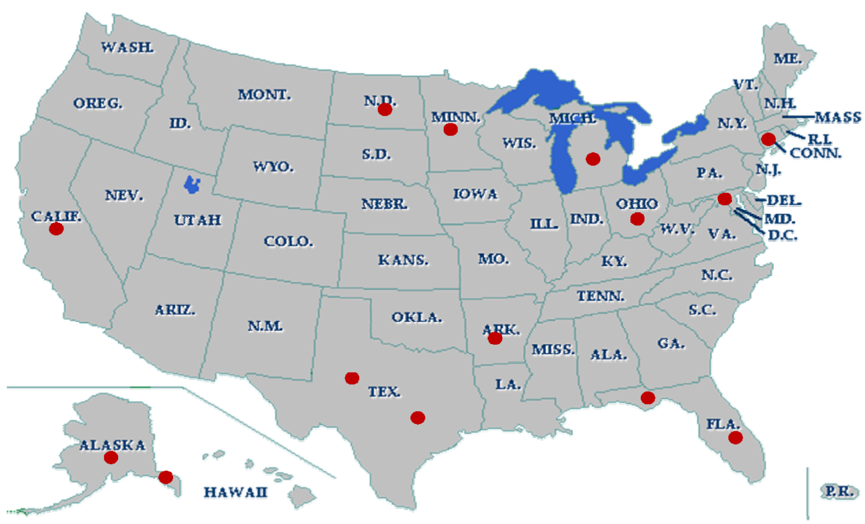
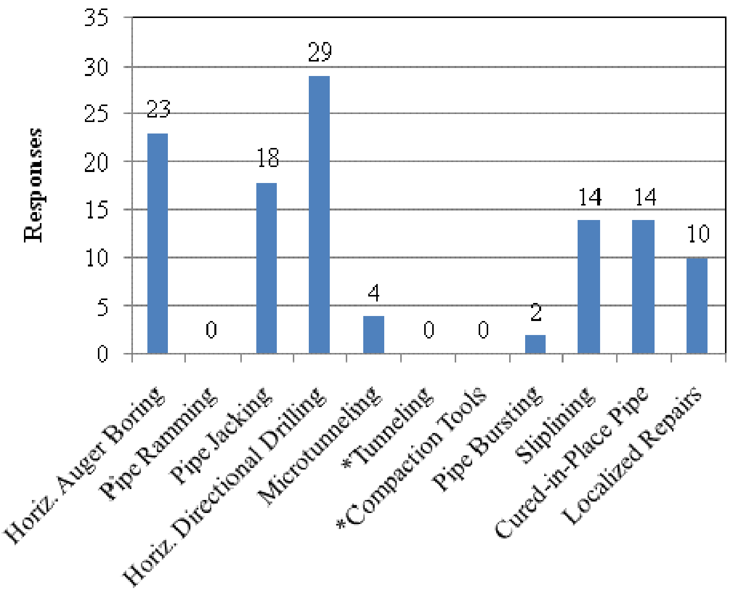


Figure 3.4. Geographical distribution of respondents to national survey

All three surveys questioned respondents about the types of trenchless technologies they had experienced in practice. Respondents were asked to select from a list each of the methods they had encountered. The question was asked in a slightly different way in the three surveys; 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 responses were useful to help researchers develop an understanding about which trenchless technologies are commonly used in Iowa, the Midwest, and the country and provide more information on the current level of familiarity with different trenchless methods. Results from the Iowa, Midwest, and national surveys are shown in Figures 3.5, 3.6, and 3.7, respectively. Note that in all of these graphs some respondents selected more than one method.



*These categories were not included in the Iowa survey.

Figure 3.5. Trenchless technologies experienced in practice by Iowa survey respondents

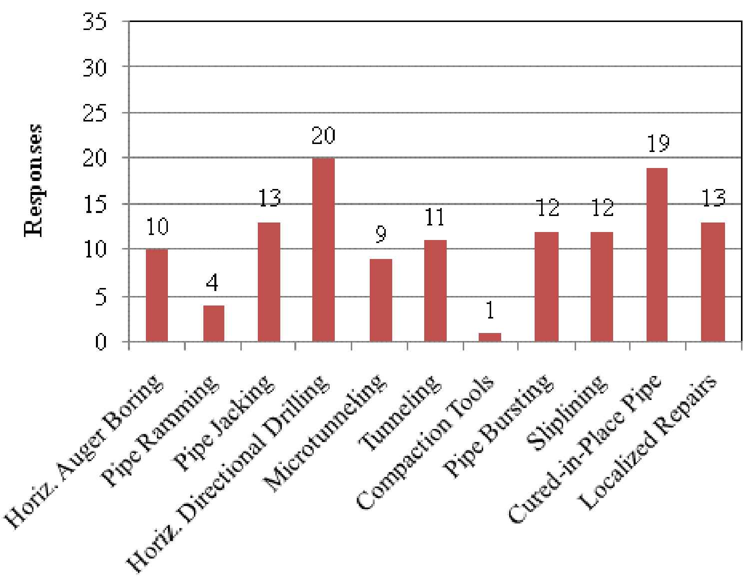


Figure 3.6. Trenchless technologies experienced in practice by Midwest survey respondents

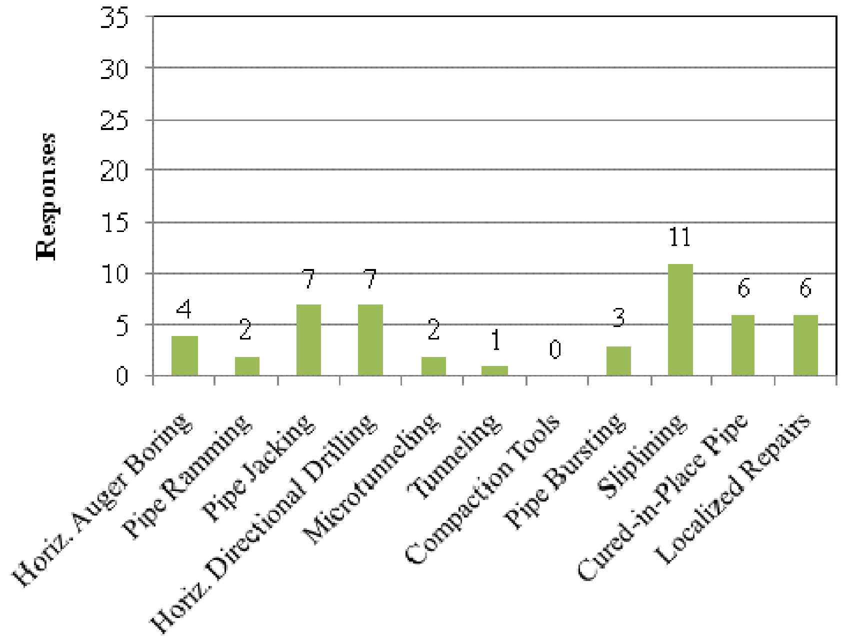


Figure 3.7. Trenchless technologies experienced in practice by national survey respondents

The survey responses demonstrated that HDD was the method most often encountered by professionals in Iowa and the surrounding Midwestern states. Iowa respondents also reported significantly higher experience with horizontal auger boring than respondents working in the Midwest or nationally. A high use of CIPP and LOR as TRMs reported in the Midwest survey may partially reflect more responses by contractors and manufacturing/sales people who may specialize in trenchless rehabilitation.

Respondents to the national survey indicated a high level of familiarity with sliplining, which suggests that sliplining may be practiced more commonly outside of the Midwest. Additionally, the results suggest a lower level of HDD use outside of the Midwest.

The following question dealt with pavement distress caused by trenchless technologies. Just under half of the Iowa survey respondents reported seeing pavement distress or other problems as a result of using trenchless methods (see 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 one-third of Midwestern respondents said they had seen deformations occur (see Figure 3.9). This question was not asked in the national survey.

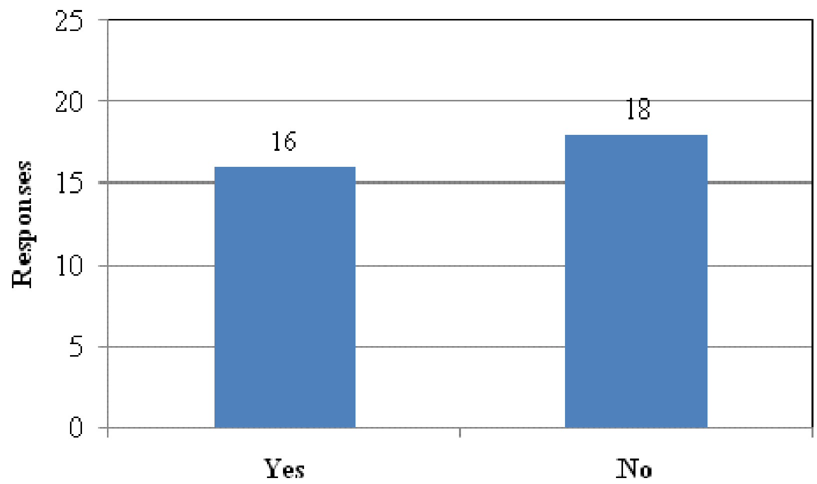


Figure 3.8. Iowa survey respondents seeing pavement distress or other problems as a result of using trenchless methods

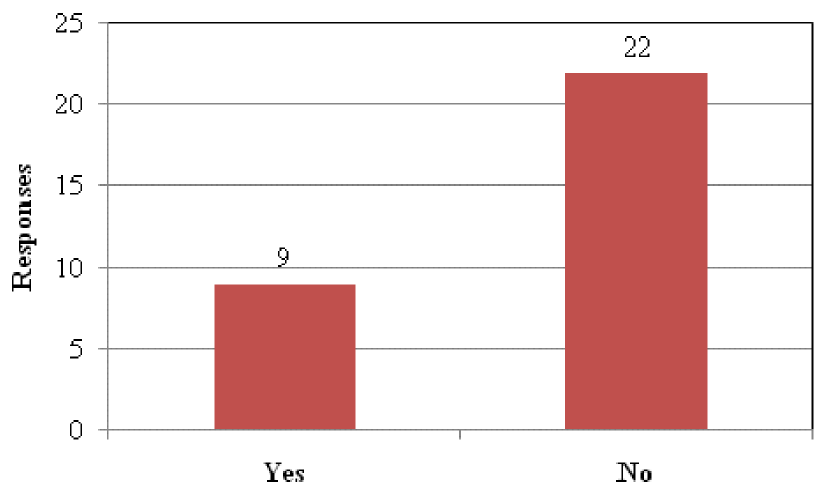


Figure 3.9. Midwest survey respondents encountering pavement deformations caused by trenchless methods

Iowa survey respondents 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 as follows:

- 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 their experience with the amount of vertical soil displacement caused by trenchless installation methods. Respondents noted that the soil type, depth of construction, size of borehole, and method all have an effect. A lower depth was generally thought to cause less surface displacement. Experienced displacements were 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, approximately 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 LORs are not likely to cause heaving or settlement. Sands were thought to cause little surface heave, while clay and gravel could potentially cause several inches of heave.

Iowa survey respondents were also asked what QC/QA methods were currently being used for trenchless projects. Responses are summarized as follows

- 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
- 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. Approximately 60% of Midwest survey respondents answered that current levels of QC/QA were appropriate, while only 40% indicated that current levels of QC/QA were not appropriate (see Figure 3.10). None of the national survey respondents felt that current levels of QC/QA were appropriate (see Figure 3.11), with six respondents indicating “no” and eight respondents with no opinion. These negative responses were surprising considering the positive responses in the Midwest survey.

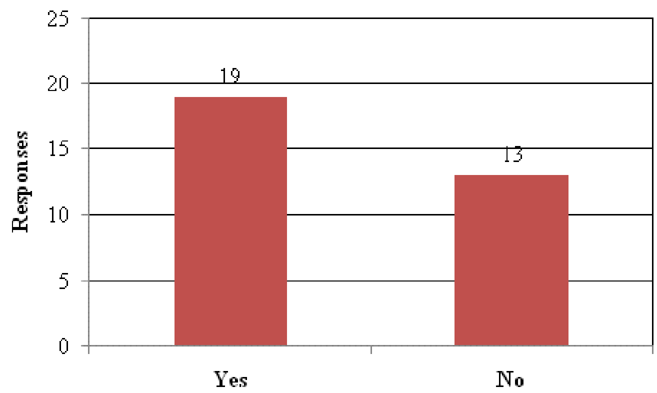


Figure 3.10. Midwest survey responses for appropriateness of current QC/QA for trenchless projects

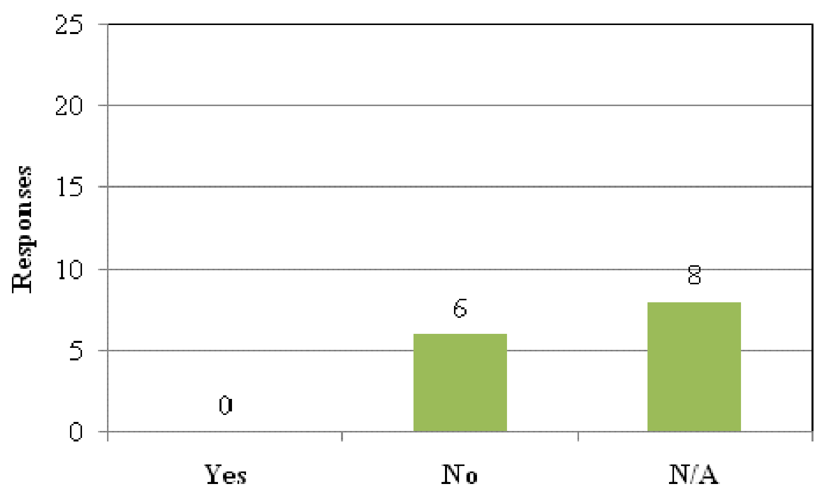


Figure 3.11. National survey responses for appropriateness of current QC/QA for trenchless projects

Iowa, Midwest, 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 respondents generally felt that current methods are not always adequate. Common answers were as follows:

- 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

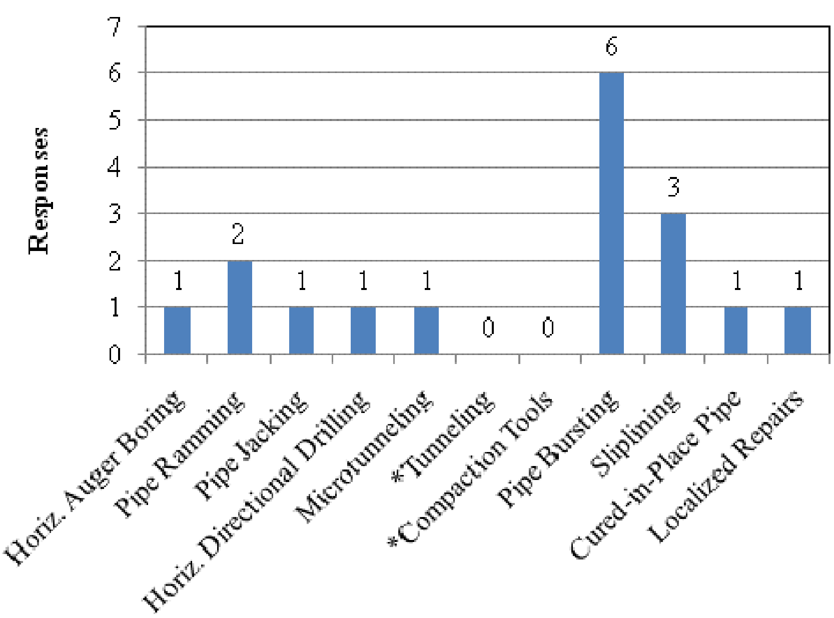
Iowa survey respondents were asked about the currently used methods of soil investigation prior to trenchless construction. The most common methods were as follows:

- 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 were as follows:

- 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 because the casing is advanced to minimize the amount of over-excavation.
- 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 (see 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 excavating service connections and modifying 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 regarding which trenchless method was considered least favorable

The Midwest and national surveys 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 those familiar with trenchless technologies believe that the reliability of the methods could be improved. National survey respondents expressed a more negative view of the reliability of trenchless technology. Only 5 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.

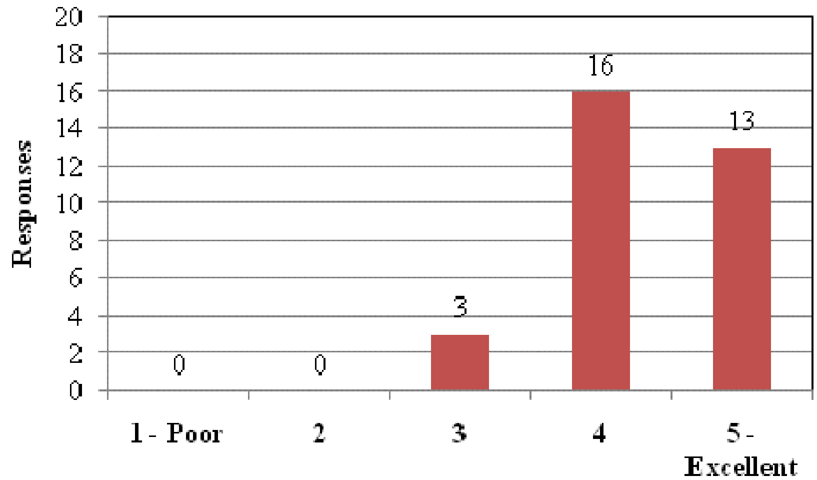


Figure 3.13. Midwest survey ratings of reliability of trenchless technologies

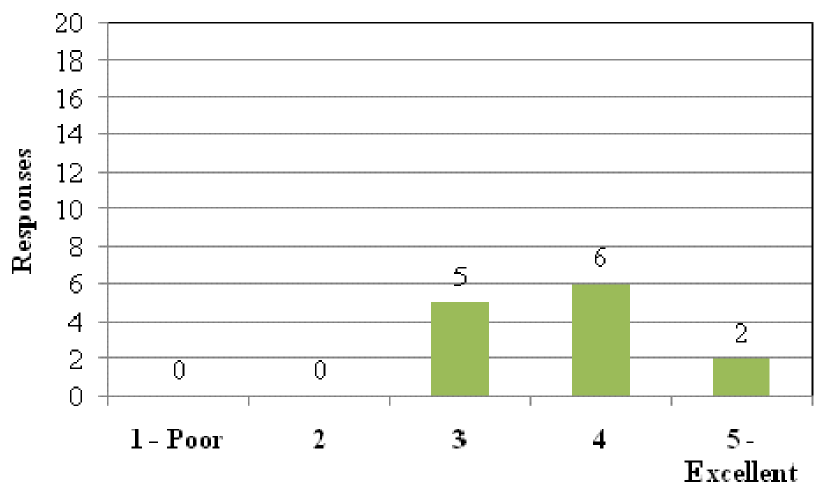


Figure 3.14. National survey ratings of reliability of trenchless technologies

Iowa survey respondents were asked to list research topics and possible improvements that could make trenchless technologies more robust; possible improvements were as follows:

- More requirements on exact final location of the piping after installation
- Improved machine control and monitoring systems
- More certification programs for contractors
- Tighten 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. The following are comments related to trenchless design considerations and soil testing.

- Given the necessary specifications, experienced contractors need to be allowed the flexibility to use whatever methods they deem necessary to complete a job. 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 the water table. They interpret soil with blow counts of 1 or 2 to indicate a soil of "toothpaste"-like consistency, blow counts of 10 to 25 to indicate reasonable soil, and blow counts of greater than 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 two feet away from any other utility.
- In moling, for each inch of borehole diameter, 10 inches to 1 foot of soil cover is required.
- In pipe bursting, you need two feet of clearance between the pipe to be burst and the nearest pipe.

Comments related to difficulties encountered during trenchless construction are summarized as follows:

- Projects with large pipes are very difficult because large pipes displace so much soil.
- It can be very difficult to predict heave and settlement. For example, 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 to 6 inches in diameter at shallow depths. These conduits 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 portland cement concrete (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 to be concerned about heaving.
- Most common problems encountered during HDD are (1) existing utilities, (2) rock, and (3) water.

During an interview, a city designer told the research team about an HDD bore that was drilled 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 because utility companies develop informal plans in-house. The subcontractor installed 6 4-inch-diameter PVC pipes in one 21-inch-diameter borehole. A distance of 2 to 3 meters was allowed between the boring and a nearby retaining wall. Apparently, bentonite was over-pumped through what may have been shale. This action 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, the utility company and contractor did not seek soil information 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.

Figure 3.15. Pavement cracking caused by 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 a 12-inch-diameter conduit at a depth of 5 feet (see Figure 3.16).

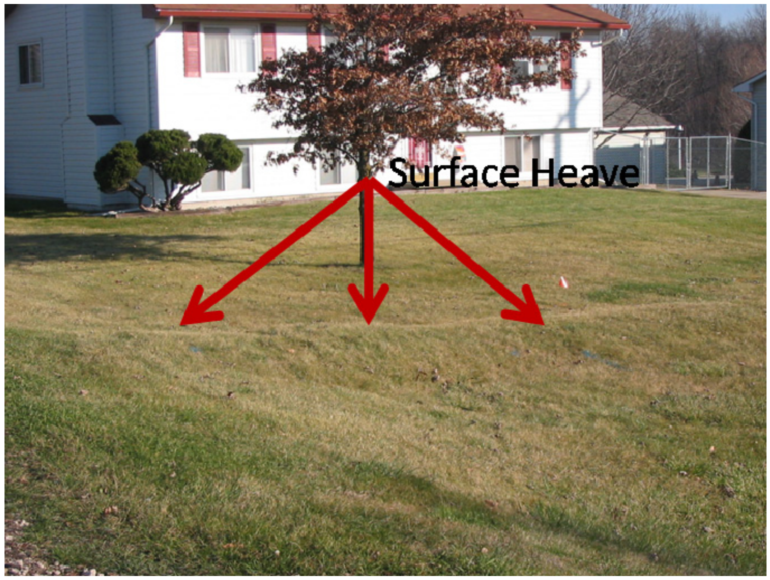


Figure 3.16. Surface heave on HDD project

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 5 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 supporting vehicles.

CHAPTER 4. SITE INVESTIGATION

4.1 Introduction

Nineteen trenchless construction projects were observed as part of this study. The research team visited trenchless jobsites in Iowa with the goals of gaining a better understanding of how this work is performed, how to identify the risks involved in trenchless technologies, and how those risks can be minimized. The trenchless construction techniques observed in these projects included auger boring, HDD, tunneling, pipe jacking, and impact moling (see Figure 4.1).

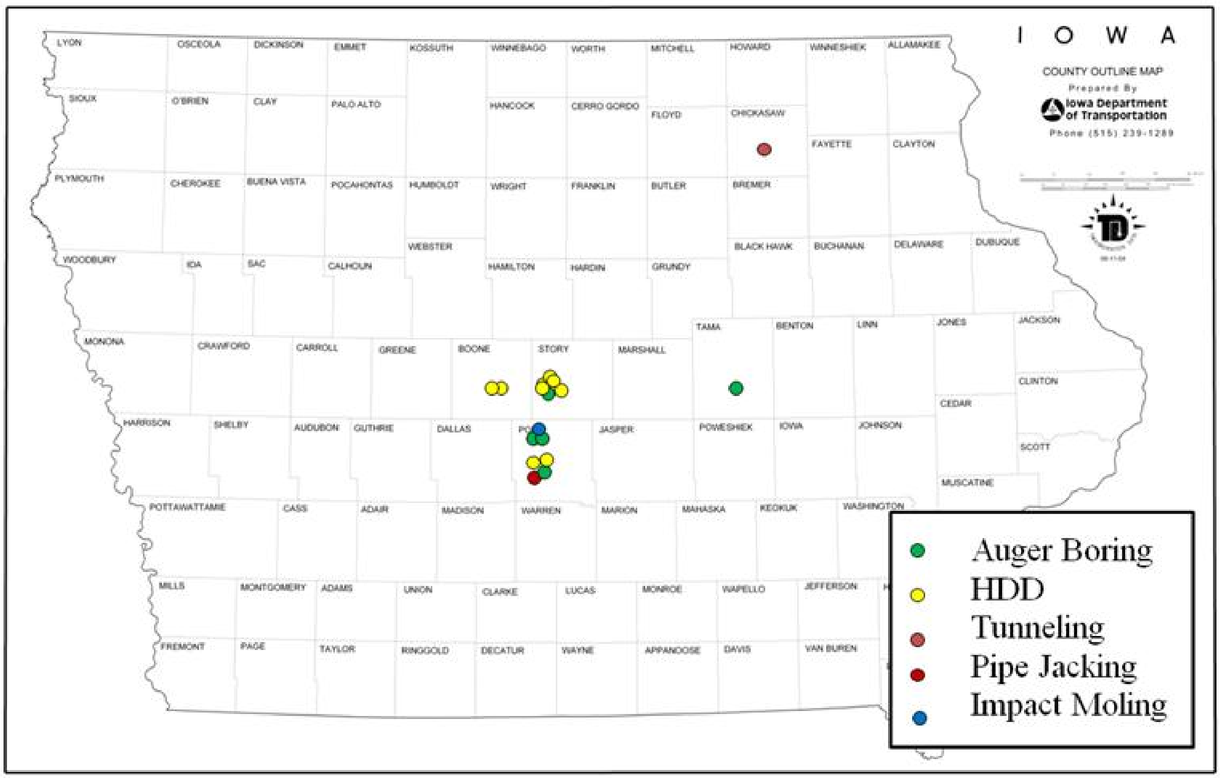


Figure 4.1. Locations of trenchless construction projects visited

The research team's approach to these projects is divided into two categories. The first category is the "Site Visit" portion, in which the research team visited a jobsite to observe and document construction practices, evaluate soil properties, and document 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 six projects. Field monitoring was confined to central Iowa due to logistical challenges. The goal of the "Field Monitoring" was to gain a more in-depth study of trenchless construction projects. Undisturbed soil samples were recovered for laboratory testing, soil stresses were measured during 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.

A total of 19 projects were investigated, including 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 box culvert of 10 feet by 5 feet. Installation lengths ranged from 24 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 also involved surface heave.

Each of the 19 projects that was observed is described in detail either in this chapter or in the Appendix D. The projects described in this chapter were selected by the authors as the best or only example/examples of a particular trenchless installation method. Projects described in Appendix D are available to provide further examples of auger boring and HDD installations and to provide complete documentation of research activities. All projects that involved lateral earth pressure measurements are described in this chapter in order to support the analysis of lateral earth pressure results.

4.2 Site Visits

This section documents the sites that the research team visited. Research activities were limited to visual observations, interviews, collection of plans and specifications, and limited soil testing.

Table 4.1. Summary of projects

Site	Trenchless Method	Pipe Diameter (in)	Length (ft)	Depth (ft)	Location of Description	Lateral Earth Pressure Measurements Taken?	Comments
Des Moines, Keo Way	pipe jacking	10’x5’	24	20	Main report	No	Box culvert
Chickasaw County	tunneling	66	44	3	Main report	No	
Ankeny, State Street	auger bore	36	170	20-30	Main report	No	Flooding from wet sand seam
Des Moines, 6th and 64th	auger bore	32	85	16	Appendix D	No	
Ankeny, Delaware and 47th	auger bore	30	110	22	Main report	No	
Tama Co., RR crossing	auger bore	60	80; 80; 80	3	Appendix D	No	
Ames, Osborn Drive 1	auger bore	24	80; 80	10	Appendix D	No	
Ames, Johnny Majors Field	HDD	2 pipes of 4” together	400; 400	17	Main report	No	Heave, frac-out
Des Moines, 62nd and Grand	HDD	16	140	12	Appendix D	No	
Ames, Osborn Drive 2	HDD	8	330	6-9	Appendix D	No	
Ames, Osborn Drive 3	HDD	8	85; 495; 325	6	Main report	Yes	Some frac-out
Ames, Seed Science Bld.	HDD	6	240; 240	6	Main report	Yes	
Ames, Forker	HDD	4	180; 180	6	Appendix D	No	
Ames, Hub	HDD	6	120	6	Main report	Yes	
Boone 1	HDD	8	100	8	Appendix D	No	
Boone 2	HDD	16	73	8	Appendix D	No	
Ames, Pammel Drive	HDD	8	480	6	Main report	Yes	
Ankeny, safe city demo	impact moling	0.75	48	4	Main report	Yes	
Ames, State Ave.	HDD	8	30	8	Main report	Yes	

4.2.1 State Street, Ankeny, Auger Bore

4.2.1.1 Project Information

This project was located on State Street in Ankeny, Iowa, between Oralabor Road and Magazine Road (see Figure 4.2) during April 2007. The auger boring technique was used to install a 36-inch-diameter steel casing for a 24-inch PVC sanitary sewer pipe. The casing was bored for a length of 180 feet at a depth of 20 to 30 feet to the top of the pipe. 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. Location of auger boring project (bore path in red) on State Street in Ankeny, IA

4.2.1.2 Trenchless Method Selection

Trenchless construction was selected to avoid closing the separated four-lane State Street (see Figure 4.3). Auger boring was used by the contractor on this project. Although the contractor had experience with HDD, the contractor chose to use the auger boring procedure because the steel casing was 36 inches in diameter, which was larger than the normal sizes that the contractor had 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.

Figure 4.3. North section of State Street above boring; insertion pit visible at right

4.2.1.3 Soil Conditions

Neither the contractor nor the owner conducted soil testing before starting the auger bore, mainly due to 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.

4.2.1.4 Trenchless Installation

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

Figure 4.4. Launching pit

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 so that slight adjustments could be made to the grade of the steel casing.

The first 20-foot section of casing pipe was lowered into the launching pit with the 20-foot length of auger inside the pipe. The pipe and auger were connected to the machine, and boring 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-foot-long pipe section, the crew checked the line and grade before welding each pipe section to the pipeline.

Figure 4.5. Pipe and auger boring machine operation, with fluid supply line, water line, and steering rod running along top of pipe

After about 100 feet of boring, a sand seam (with no boulders) was encountered (see Figure 4.6). The sand was accompanied by a large quantity of water, which flooded the borehole and left 6 inches of standing water in the launching pit, 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 feet of grout to the cutting face through one of the small pipes located immediately above the casing pipe. Pumping 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 pavement damage was observed, which was a result of the contractor's actions.

Figure 4.6. Sandy soil deposited out of auger boring machine

Boring through the sand continued for about 30 feet, at which point the soil transitioned back to weathered shale until the completion of the installation (see Figures 4.7 and 4.8). Boring through sand took about one and a half days. The transition to weathered shale greatly increased the rate of construction progress. It was observed that while boring, 10 feet of pipe was installed in 10 minutes.

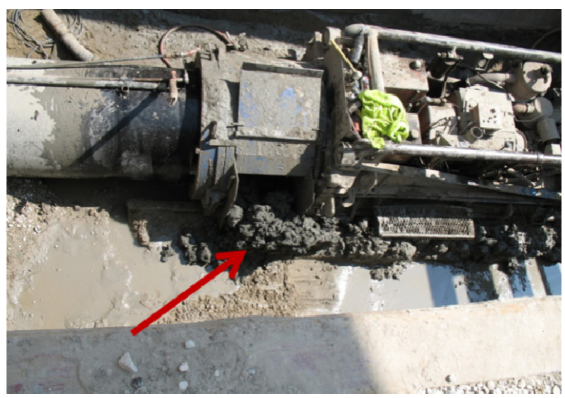


Figure 4.7. Clayey weathered shale deposited out of auger boring machine

Figure 4.8. Weathered shale deposited out of auger boring machine

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 to allow proper positioning (see Figure 4.9). The PVC pipe was then placed in the casing using a backhoe. This process completed the pipeline installation. No additional problems were encountered on this project, and no surface heave or settlement was observed.

Figure 4.9. Spacers on 18-inch PVC carrier pipe to allow pipe to “float” in casing

4.2.1.5 Research Team Actions

The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. Researchers were unable to recover soil samples for this project.

4.2.1.6 Key Findings

This project provided a good example of using 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–foot-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; this action prevented immediate subsidence or other pavement damage.

Line and grade were critical to the project because the carrier pipe was going to be gravity-flow pipe. The pipe deflected off course during the transition into and out of the sandy soil, and this deflection was corrected 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.

4.2.2 Keo Way, Des Moines, Pipe Jacking

4.2.2.1 Project Information

This project was located at the intersection of Keo Way and Crocker Street in Des Moines, Iowa, in 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 of about 13 feet to top of pipe. Precision was required because 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.

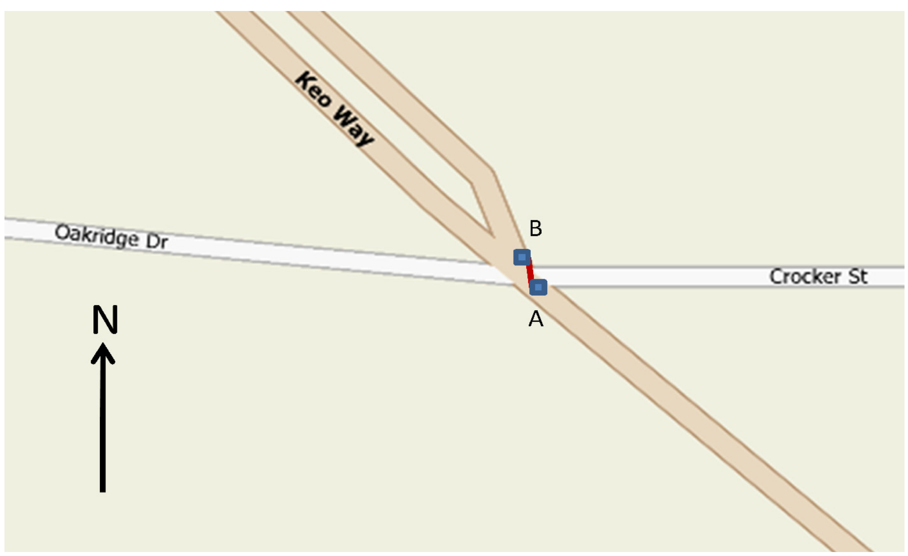


Figure 4.10. Location of pipe jacking project (bore path in red) on Keo Way in Des Moines, IA

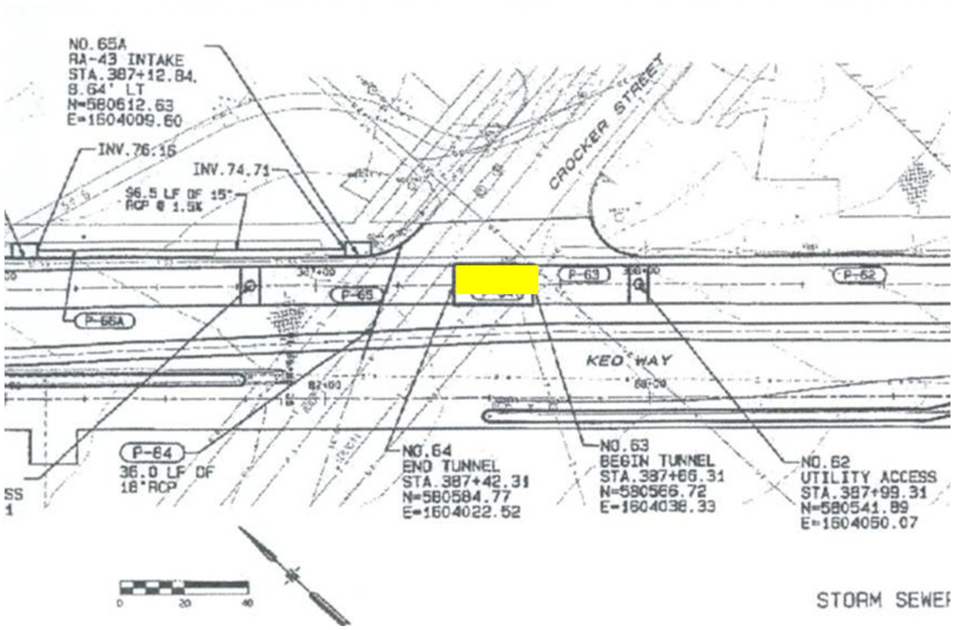


Figure 4.11. Plan view of project site; bore path in yellow

4.2.2.2 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.

4.2.2.3 Soil Conditions

The contractor did not conduct soil testing before starting the pipe jacking installation, mainly due to 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 depth of installation was considered appropriate for pipe jacking construction, and problems related to soil conditions were not anticipated.

4.2.2.4 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 Figures 4.12 and 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 approximately 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.

Figure 4.12. Launching pit with trench box and steel plates

Figure 4.13. Trench box before installation began

The first 6-foot-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. However, progress was halted immediately because of 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 was important to launch the tunnel correctly because it is difficult to correct an initial misalignment.

Figure 4.14. Segments of 10-foot by 5-foot RCBCs

The first box pipe was then jacked into the soil, using a mini road header to excavate the soil (see Figures 4.15 and 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.

Figure 4.15. Jacking begins on the first box pipe section

Figure 4.16. Mini road header used to excavate soil as pipe was jacked in

Figure 4.17. Skid loader used to move spoil from pipe to trench box to be removed by backhoe

Unexpected problems occurred before the first of the four pipe sections could be installed. A leak 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 2 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.

4.2.2.5 Research Team Actions

The research team observed the installation, interviewed crew members, and examined the overlying pavement for signs of damage. The research team did not recover soil samples due to the difficulty of accessing the launching pit bottom and due to the contamination caused by the sewer leak.

4.2.2.6 Key Findings

This project provided a good example of using pipe jacking to install reinforced concrete box pipe. A 10-foot by 5-foot rectangular sanitary sewer pipe was installed for a distance of 24 feet at a depth of 13 feet to the top of the pipe. 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 are considered appropriate for pipe jacking.

The installation was successfully completed using pipe jacking technology. The only important 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.

4.2.3 Highway 63, Chickasaw County, Tunneling

4.2.3.1 Project Information

This project was located on Highway 63 (locally known as McCLoud Avenue), five miles north of New Hampton in Chickasaw County, Iowa. The project took place 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 Department of Transportation (Iowa DOT) hot mix asphalt (HMA) resurfacing with milling project.

The tunneling machine was set up at point A and bored west to point B, as shown in Figure 4.18. The research team observed the installation and interviewed crew members.

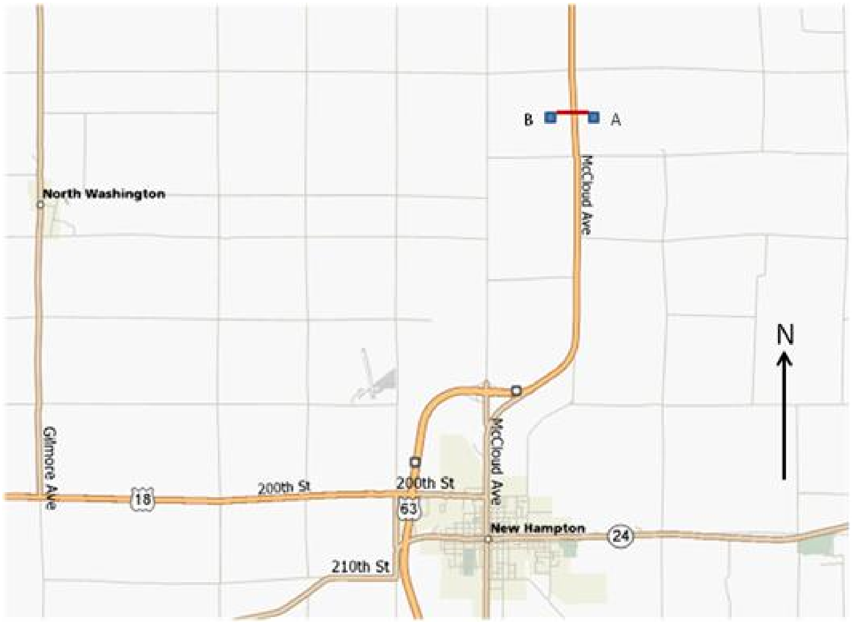


Figure 4.18. Location of the project site (bore path in red) on Highway 63 (McCloud Avenue) near New Hampton, IA

4.2.3.2 Trenchless Method Selection

Trenchless construction was selected to allow Highway 63 to stay open during the installation, as mandated by the Iowa DOT. An advantage of tunneling over other trenchless methods was the ability to install large-diameter pipes and the ability for workers to access the cutting face by climbing inside the tunneling machine.

4.2.3.3 Soil Conditions

Four experimental vertical boreholes with diameters of 4 inches were drilled for the project in order to characterize the soil and to determine if tunneling at such a shallow depth would be likely to damage the overlying pavement (see Figure 4.19). Boreholes were drilled near point A, in each lane, and at point B. Figure 4.19 shows two 4-inch-diameter sample corings visible in the asphalt and a red lathe in the background marking the end of bore at the edge of the gravel shoulder .

Soil classifications resulted in the following profile: the 1.8-foot-thick road and subbase were underlain by 4.2 feet of stiff black sandy clay fill. Below this layer was 4 feet of stiff black silty clay, and 7 feet of firm gray brown glacial clay were below that. A very firm dark gray glacial clay extended from a depth of 17 feet below ditch level for at least an additional 14 feet. The water table was established at 14 feet below the pavement. Additionally, glacial boulders were expected because they are commonly found during construction in this area. A much higher than expected concentration of these boulders was discovered, which caused considerable delays.

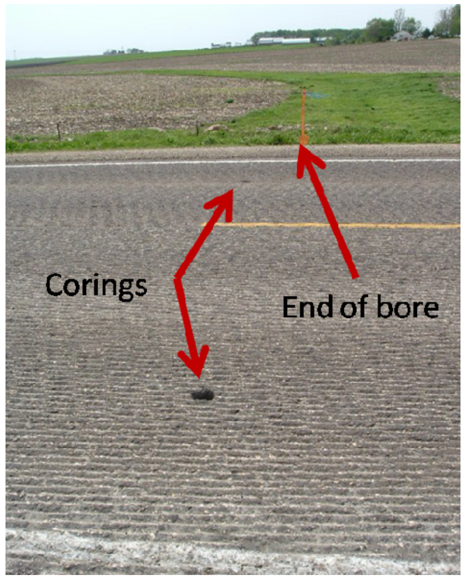


Figure 4.19. View from point A looking west across Highway 63 toward point B

4.2.3.4 Trenchless Installation

After nearby existing utilities were located, the launching pit at point A was dug, and a bed of 0.75-inch maximum size gravel, 6 inches thick, was laid in the launching pit. The track for the TBM was then placed on the gravel bed at the correct line and grade. The distance from the top of the pipe to the pavement was approximately 2.6 feet at the closest point, which occurred at the beginning of the installation on the east side (see Figure 4.20).

Figure 4.20. Partially flooded launching pit at point A, with track installed prior to tunneling

The TBM had limited steering capability, so it heavily relied 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 from being made during several days. Sumps were used in the launching pit to keep the pit dry.

Figure 4.21. TBM before starting the bore

The TBM was jacked to the start of the bore and the cutter blades were spun, excavating the soil. As progress began, sloughing off the pea 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 would not propagate as far as the pavement surface, so the bore proceeded. As predicted, this did not develop into a serious problem, and the lost pea gravel was replaced.

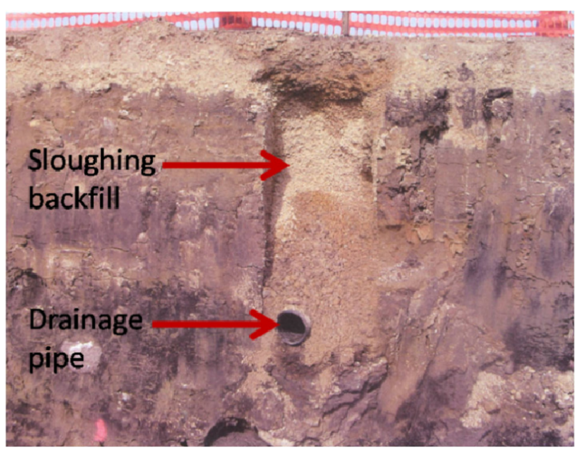


Figure 4.22. Existing granular backfill over a drainage pipe begins to slough away

Shortly after the boring commenced, it was discovered that a fiber optic cable running parallel to 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 shown on the drawing, possibly interfering with the tunneling operation. An excavator was then used to assist a worker in locating the cable manually.

The TBM cut the soil away, 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. A large number of glacial boulders between 12 and 18 inches in diameter that were encountered significantly slowed progress (see Figure 4.23). A much larger quantity of these boulders was found than had been expected. The reason they were found at such a high concentration and close to the surface is likely because past highway construction practices allowed boulders to be included in the fill.

Figure 4.23. Boulders removed from the borehole (12 to 18 inches in diameter)

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 remove the boulders manually. Picks, crowbars, air hammers, and jack hammers were used to dislodge the boulders from 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 with a chain that was 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. Soon after the boulders were discovered in the fill, the crew switched cutter blades from dirt to rock blades. This switch had little effect on the actual speed of the operation but it limited equipment damage. However, the crew planned to switch back to the dirt blades if there were fewer boulders and if clay soil became predominant, because the rock blades would clog easily in clay 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.

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

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 successfully finished after three weeks. This project took far longer than the three days that were expected because of the large number of boulders and the large amount of rain, which prevented work for several days.

4.2.3.5 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 taken to the laboratory in sealed plastic bags. Additional soil data were obtained from the owner.

Tests conducted 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 and 8.1 feet. The owner's soil data indicated that stiff black sandy clay was found between 1.8 and 6 feet and that stiff black silty clay was found from 6 to 10 feet in depth. The water table was located at 14 feet. The samples recovered by the research team confirmed that the soil at a 5-foot depth was sandy clay. The gradation curve is shown in Figure 4.25.

Table 4.2. Chickasaw County project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0–1.8	Asphalt and subbase	-	-	-	-
1.8–6.0*	Sandy lean clay (CL)	30.0	40.8	26.7	14.1
6.0–10.0	Stiff black silty clay#	-	-	-	-
10.0–17.0**	Firm gray-brown glacial clay#	-	-	-	-
17.0–31.0	Very firm dark gray glacial clay#	-	-	-	-

*Top of the 66 inch pipe at depth of 2.6 feet
**Depth of water table at 14 feet
#From the owner's soils report
-Indicates no laboratory tests were performed

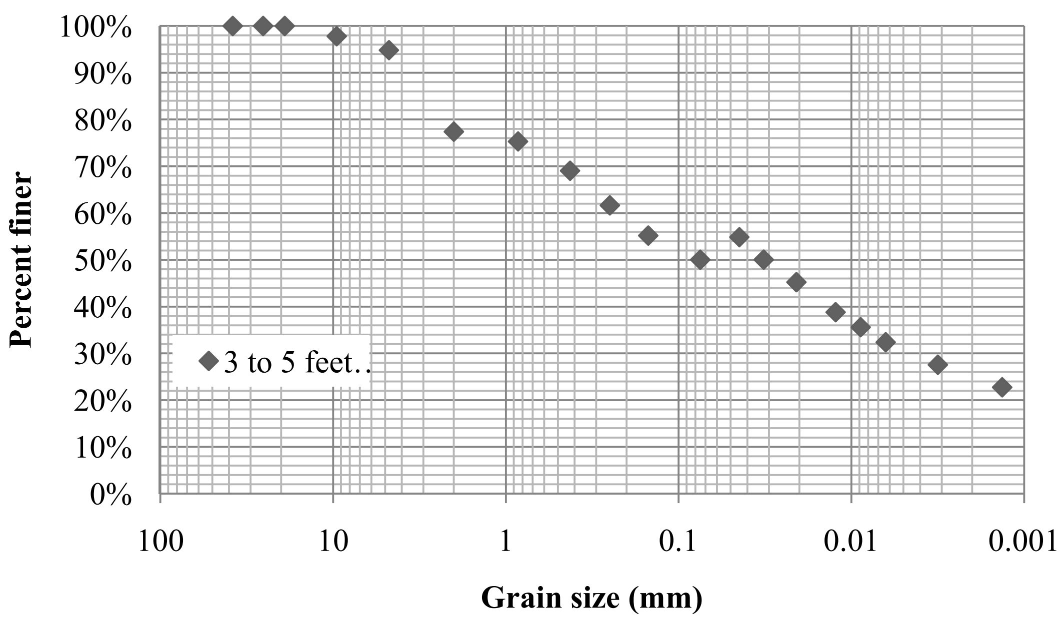


Figure 4.25. Soil gradation curves for depths of 3 to 5 feet at Highway 63 site

4.2.3.6 Key Findings

This project provided a good example of using 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 TBM was used to excavate the soil, while hydraulic jacks were used to advance the TBM and the pipe behind it. Problems were encountered because the crew was unable to work for several days due to large amounts of rain and a large quantity of boulder-sized rocks that 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 providing 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 because of construction practices at the time that the highway was constructed.

The installation was successfully completed 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.

4.2.4 Delaware Avenue and 47th Street Intersection, Ankeny, Auger Bore

4.2.4.1 Project Information

This project was located near the intersection of Delaware Avenue (also known as NE 22nd Street) 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 PVC sanitary sewer pipe. The casing was bored for a length of 110 feet at a depth of 24.5 feet to 25 feet to the top of the pipe. Accuracy was important because the gravity-flow carrier pipe had to meet grade requirements.

The purpose of the project was to connect the then-under-construction Otter Creek Golf Course with the sanitary sewer system pipes in the residential area on NE 47nd Street. The auger bore was set up at point A and bored west to point B, as shown in Figure 4.26. Figure 4.27 shows the profile view of the project.

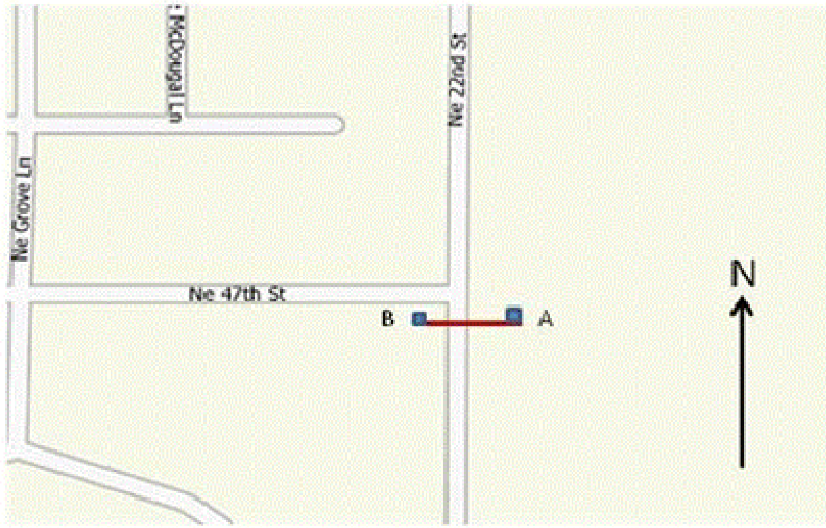


Figure 4.26. Location of auger boring project (bore path in red) at Delaware Avenue in Ankeny, IA

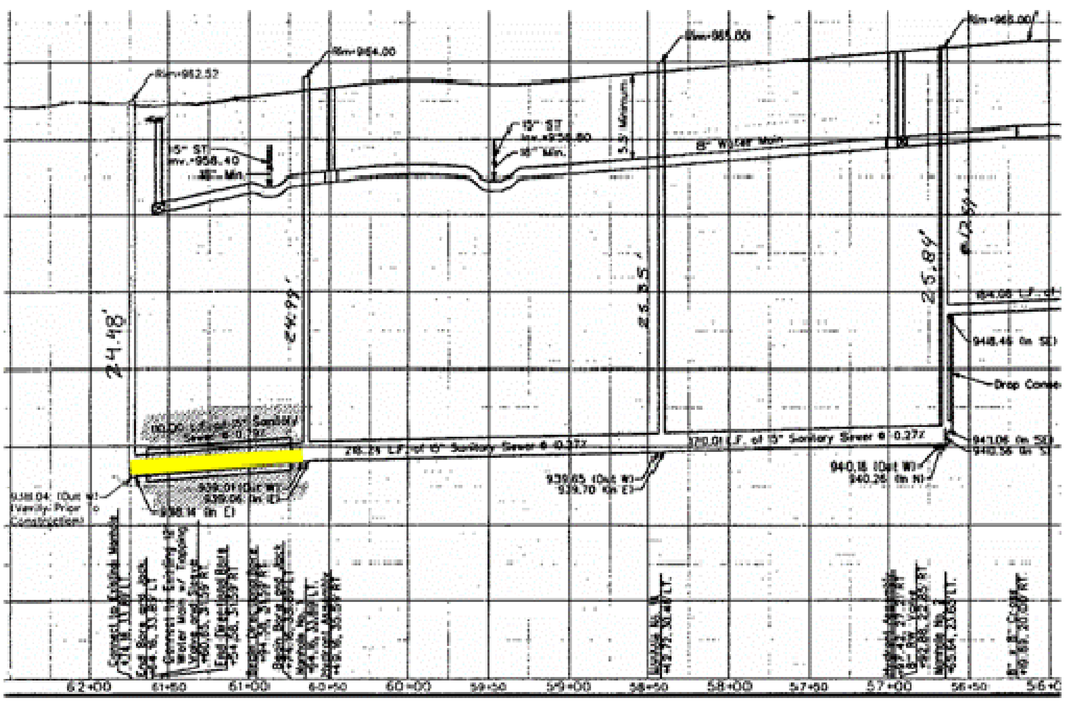


Figure 4.27. Profile view of the project; bore path in yellow

4.2.4.2 Trenchless Method Selection

Trenchless construction was selected to avoid closing Delaware Avenue and because the installation was relatively deep (24 feet). Auger boring was the trenchless method chosen because it was appropriate for the pipe size, installation depth, soil conditions, and cost. HDD was not chosen because that method is inappropriate for rigid steel pipe.

4.2.4.3 Soil Conditions

Neither the contractor nor the owner conducted soil testing before starting the HDD installation, mainly due to 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.

4.2.4.4 Trenchless Installation

After nearby existing utilities were located, the launching pit at point A was dug (see Figure 4.28). Next, steel shoring was installed (see Figure 4.29), and track for the boring machine was laid on a crushed rock base at the proper grade in the launching pit. The auger boring machine was then lowered into the launching pit and placed on the tracks.

Figure 4.28. Launching pit with Delaware Avenue in background

Figure 4.29. Launching pit with shoring and gravel base

The first 20-foot-long section of pipe was lowered into the launching pit and connected to the machine (see Figure 4.30). 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; a correction was required. A wing was installed on one side of the pipe. This wing deflected the pipe back on course and was removed after the installation was complete.

Figure 4.30. Launching pit before a new 20-foot pipe section is lowered into place

There was no clear reason why the bore was drifting left. The pipe may have encountered a rock in the bore path. Another possibility is that one of the welds that connected each of the 20-foot pipe sections may have broken. Pipes sometimes deflect when welds break.

The course of the pipe was corrected, and the installation was successfully completed (see Figure 4.31). The carrier pipe emerged at the manhole vault at point B within accuracy tolerance.

Figure 4.31. Protruding casing after boring is finished

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 4.32). 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.

Figure 4.32. PVC carrier pipe with visible spacers

Before and after construction pictures were taken of Delaware Avenue above the bore path to monitor for any pavement damage. A comparison of these photographs showed that the bore had made no visible effect.

4.2.4.5 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 installation depth. These samples were moved to the laboratory in sealed plastic bags. Additional soil data were obtained from conversations with contractor personnel.

Tests carried out on the disturbed soil samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.3. The 30-inch steel casing 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 feet. The gradation curve is shown in Figure 4.33. From 18 feet down to 28 feet, gray-black hard clay with gravel was observed. The water table was located deeper than 28 feet.

Table 4.3. Delaware Avenue and 47th Street auger bore project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0–1	Peat (Pt)	-	-	-	-
1–18	Silty sand (SM)	28.9	NA	NA	NA
18–28*	Gray-black hard clay with gravel#	-	-	-	-

*Top of the 30 inch pipe at depth of 24 feet,
Location of water table is below this level
#Reported by the contractor
-No laboratory tests conducted

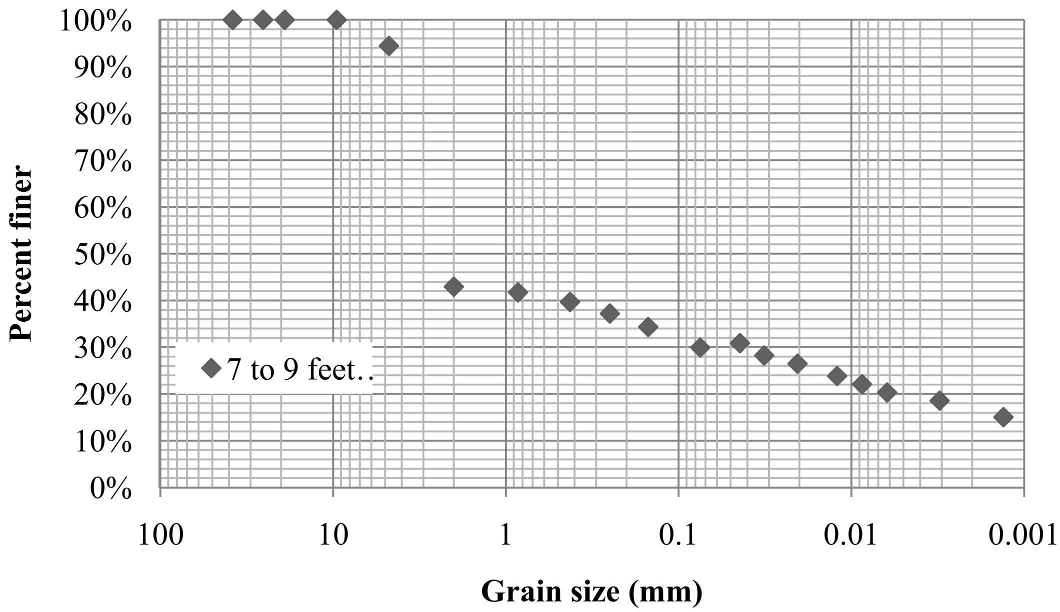


Figure 4.33. Soil gradation curves for depths of 7 to 9 feet at Delaware Avenue and 47th Street site

4.2.4.6 Key Findings

This project provided a good example of using 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 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 a gravity flow pipe. Also, the casing needed to connect with a manhole vault at the termination of the bore. At one point during installation, the pipe deflected to the left, but the crew corrected the problem and was able to complete the bore in a way that met 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.

4.2.5 Johnny Majors Field, Ames, HDD

4.2.5.1 Project Information

Renovations to Jack Trice Stadium at Iowa State University (ISU) included installing over 1,060 feet of electrical conduit under the adjacent outdoor practice field and around part of Jack Trice Stadium itself. 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-foot run crossing the Johnny Majors Practice Field from point A to point D and then from point D to point E (see Figure 4.34). During this operation, the conduits were installed in 18-inch-diameter boreholes at a depth of up to 17 feet below ground. The second stage of the project was a series of shallow bores to install a single 2-inch-diameter HDPE conduit in several shorter runs, totaling 500 feet around the outside of the stadium. These bores ran between points F and G, G and H, and H and I. These smaller bores were 4 inches in diameter at a depth of only 3 feet.

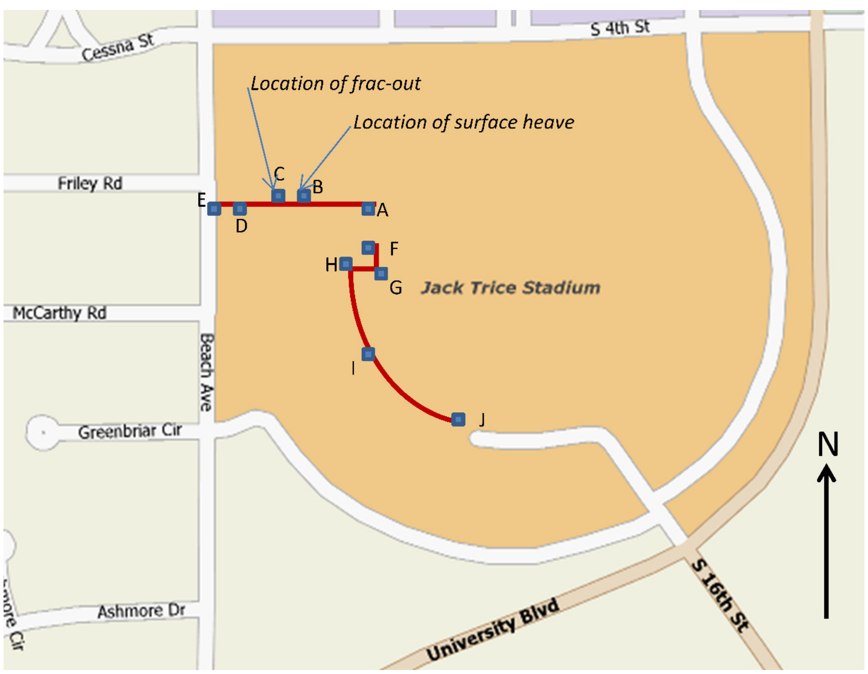


Figure 4.34. Location of HDD (bore path in red) at Johnny Majors Practice Field in Ames, IA

Significant problems were encountered while boring from point A to point D, which led to high drilling fluid pressures that caused fractures in the subsoil. As the fractures opened up, they released the built-up fluid pressure by allowing the excess drilling fluid to flow to the surface, causing an “inadvertent return” or “frac-out.” Additionally, this pressure caused the 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.

4.2.5.2 Trenchless Method Selection

The owner chose to use trenchless construction to install electrical conduits beneath the Johnny Majors Practice Field while the facility stayed in use; it was important that the facility remain open because football practice was in session during construction. Additionally, trenchless installation allowed for a relatively deep installation (17 feet) and fast completion. The planned borings are part of the larger electrical project illustrated in Figure 4.35.

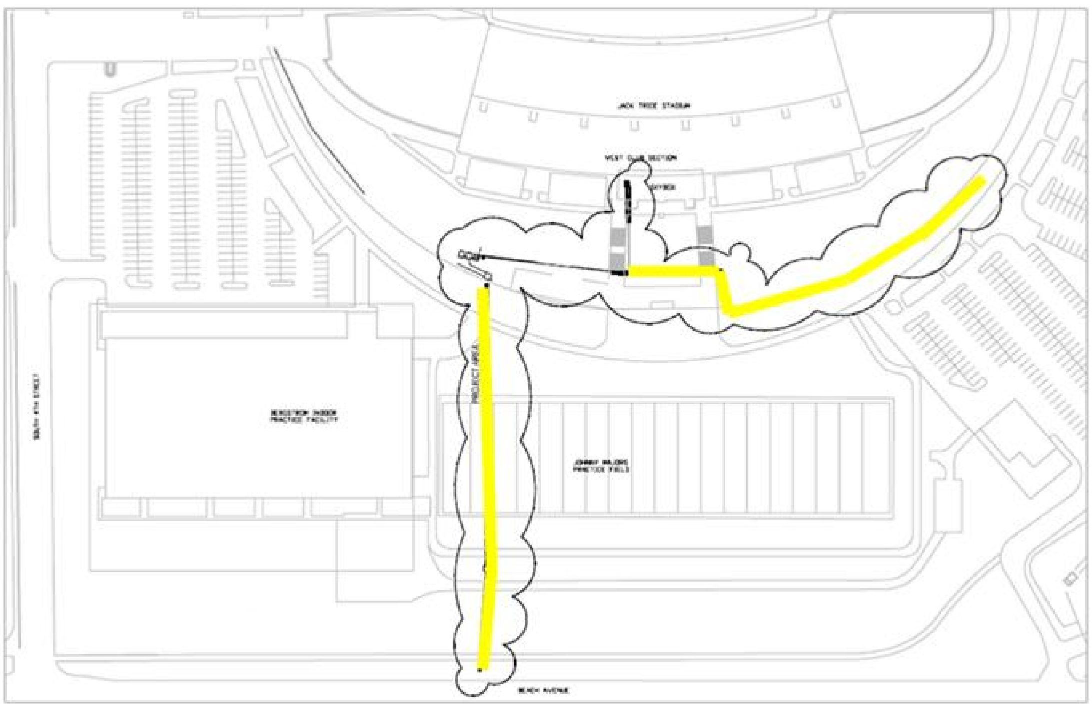


Figure 4.35. Plan view of the project site (bore path in yellow) taken from the plan set

4.2.5.3 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 the 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 primarily 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.

4.2.5.4 Trenchless Installation

The contractor decided to start the bore at 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 its HDD drill rig at point A in preparation to bore west toward point D. The plan was to first bore 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 then bore the remaining 160 feet from D to E. Preparations also included digging an exit pit at point D. This pit would allow recovery of the 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.

The contractor used a drilling fluid that was high in sodium bentonite to reduce the risk of borehole collapse if sandy soil was encountered during drilling because the contractor considered sandy soil to be of greater risk than clay. The selection of drilling fluid materials was focused toward the possibility that sand would be encountered. 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 crossed an existing 4.5-inch-diameter electrical conduit at a 5-foot depth. To avoid affecting the existing conduit, it was necessary to pothole to the old conduit to verify its actual depth and position. When the hole reached the expected depth of the existing pipe, the backhoe continued to excavate at 2-inch increments, and then a hand probe was used to find the conduit. Clay described as "blue/black" was found at the location where the two conduits intersected. It was verified that the top of the existing pipe was 57 inches below grade at the spot. This depth was acceptable because it meant that a clearance of greater than 1 foot would exist between the existing conduit and the new one above.

The pilot bore began by attaching a 4-inch drill bit to the directional drilling machine (see Figure 4.36) and then using the machine to push the drill bit into the ground. The pilot bore proceeded with periodic adjustments being made to keep the bore aligned. The pilot bore was successfully conducted, and the drill bit emerged in the exit pit at point D during the first day of boring.

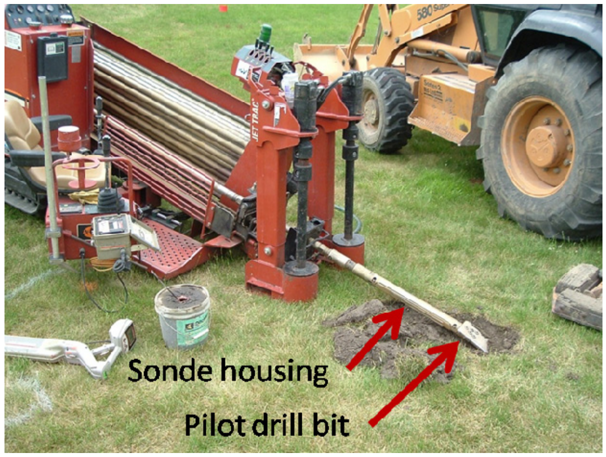


Figure 4.36. Directional drilling machine pushing pilot bore from point A to point D

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 pre-reaming stage (see Figure 4.37). The directional drilling machine was switched from push mode to pull mode, and the drill string with the attached reamer 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. However, progress was extremely slow. Only about 200 feet of the borehole had been enlarged during the first 12 hours of pre-reaming.

Figure 4.37. Twelve-inch-diameter reamer

A clay soil that had high gravel content was encountered during the pullback, which is a factor of the 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 area. This clay is often very stiff and can cause reaming to be slow and difficult.

Roughly six gallons of drilling fluid were pumped per minute through the hollow drill rods and out of small perforations in the reamer. Due to the slow pullback speed, 200 gallons of drilling fluid were being pumped for each 6.5 feet of drill rod. During this process, the excess drilling fluid should have emerged at either the entry or exit pit. However, not as much liquid as expected was observed to be exiting at those locations. This observation suggested the possibility that a frac-out was occurring. Towards the end of the second day of 12-inch pre-reaming, a frac-out was observed on the practice field surface 20 feet away from the bore path while boring through the gravelly clay soil. The pre-reaming process was continued and, several hours later, surface heave occurred directly over a portion of the bore path (see Figure 4.38). This heave occurred about 10 feet after switching from gravelly soil to clay.

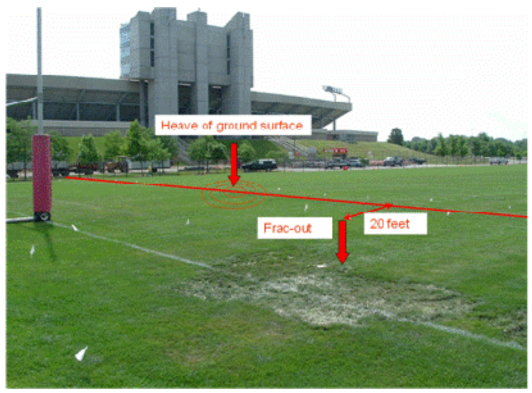


Figure 4.38. Surface heave in background and frac-out in foreground

A possible initial cause of the frac-out is that the gravel may have caved in behind the reamer. In addition, stiff clayey soil cuttings may have clogged the previously enlarged hole and prevented the evacuation of drilling fluid to the exit pit. The reamer itself may have also become 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 clogging may have led to rising fluid pressures and the eventual soil fracturing along planes of greatest weakness. Another possibility is that the drilling fluid flowed out of the borehole in a direction roughly perpendicular to the bore path and finally reached the surface 20 feet away as a visible frac-out. The appearance of these inadvertent returns can be seen in Figure 4.39, 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; sandy soil was observed in that area. Additional damage was done to subsurface sprinkler water main pipes at a depth of 10 feet.

Figure 4.39. Edge of frac-out region with drilling fluid at the ground surface

The surface heave that followed the frac-out several hours later is thought to have resulted from a similar mechanism. At the location of the surface heave, the subsurface investigation showed that stiff clay existed 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 slow the progress so that fluid pressures in the borehole may have reached sufficiently high levels to rupture the sides of the borehole. If the drilling fluid could not escape, it may have displaced the soil upward. The heave mechanism may have released enough pressure so that a frac-out did not occur.

The 12-inch-diameter pre-reaming process was completed at the end of the third day of pre-reaming. Before its completion, a second frac-out occurred late in the third day. This frac-out was located in the area of the earlier surface heave and may have been caused by the continued high drilling fluid pressures inside the borehole, possibly because the borehole and the reamer became clogged with clay so that drilling fluid could not flow to 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. The location of this occurrence is shown in Figure 4.40.

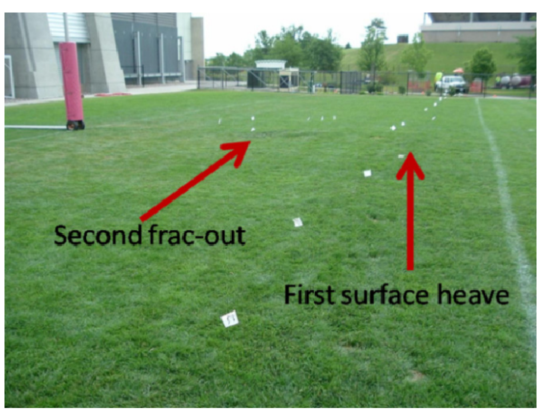


Figure 4.40. Second frac-out near location of earlier surface heave

After the contractor finished pulling the 12-inch-diameter reamer back from the exit pit to the entry pit, the reamer was removed. A more powerful drill rig had been rented to increase the speed of the bore and to accomplish the pullback of an 18-inch-diameter reamer with the product pipe attached during the same pull. The 2-inch-diameter drill string was easily pushed back through the borehole to the exit pit. There, an 18-inch-diameter reamer was attached to the drill string so that it could be pulled back through the borehole to increase the diameter (see Figure 4.41). Two 4-inch HDPE conduits were also connected to the back of the 18-inch reamer. The two conduits simultaneously pulled side by side created an effective conduit outside diameter of 10 inches in an 18-inch borehole, which falls within the HDD rule of thumb for a borehole–to-pipe diameter ratio between 1.5 and 2 to 1.

Figure 4.41. Reamer shown jetting drilling fluid in point D exit pit just before pullback

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. The pullback of the18-inch-diameter reamer 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 so that a second attempt could be made to pull back the reamer and pipe.

The 18-inch reamer was then reattached to the drill string with the product pipe. Although the borehole had already been pre-reamed once with the 18 inch reamer, it was necessary to reattach the reamer for a second pullback 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 because the already enlarged borehole required only a reasonable amount of pullback force (see Figure 4.42).

Figure 4.42. Installation of two conduits connected behind 18-inch reamer at point D

The new drill rig had a gauge for monitoring pullback forces, and it was observed that the successful pullback of the pipe and reamer through the already enlarged borehole required a nominal pullback force (3,000 pounds), while the previous pullback of the 18-inch reamer through the 12-inch hole required much less pullback force (500 pounds). This force difference indicates that the pullback friction of the pipe against the edges of the borehole caused a large increase in the required pullback force. The 18-inch reamer pullback and the following 18-inch reamer with product pipe pullback took a combined three days to complete.

The project required that the two 4-inch electrical conduits continue 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 point E and pushed the same 4-inch-diameter pilot bit used for the earlier practice field crossing down to the exit pit at point D. A depth of roughly 6 feet was maintained during the bore, and the soil cuttings returned in the drilling fluid confirmed the soil had high clay content.

In light of the difficulties encountered during the practice field drilling, a different drilling fluid mix was used for the second bore. This time, a polymer-based drilling fluid with a clay inhibitor additive and less bentonite was used. The pilot bore was quickly finished, and the effectiveness of the drilling fluid mix encouraged the contractor to back ream a 12-inch-diameter reamer attached to the two 4-inch HDPE conduits. Although this procedure created a 12-inch hole for the 10-inch-diameter product, which violated the 1.5 to 1 rule of thumb, the contractor considered it adequate given the ease with which the pilot bore had been accomplished and the relatively short distance. This operation was completed successfully and without incident in one day (see Figure 4.43).

Figure 4.43. Capped 4-inch HDPE pipes after installation at point E next to Beach Avenue

The contractor's drill rig that was used for the bore from point A to point D was the rig used to complete the additional 500 feet of boring around the outside of Jack Trice Stadium. These bores were made for the installation of a single 2-inch-diameter HDPE electrical conduit. The drill rig was repositioned three times to make four separate runs. These bores ran from points F to G, G to H, H to I, and I to J. These bores were much shallower than the practice field bores, with depths of only 3 feet. Installation was completed by first pushing the 4-inch-diameter pilot bore through 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 conduit were then pulled back from the exit pit to the entry pit. The installations were completed successfully.

4.2.5.5 Research Team Actions

The research team completed an elevation survey to find the height of the surface heave. It was found that the ground surface had been vertically heaved 9 inches at the highest point.

The research team drilled five test boreholes near the areas of frac-out and surface heave to better understand the soil types in these areas and to formulate theories for the reasons behind these occurrences (Figure 4.44). Samples of the disturbed soil were taken at various depths, and 2- and 3-inch-diameter thin-walled Shelby tubes were recovered. The soil samples from the Shelby tubes and the bagged samples of disturbed soil were tested in the laboratory.

Figure 4.44. Drill rig used by research team to recover soil samples

4.2.5.6 Soil Characterization

Tests carried out on the disturbed soil samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.4 for soil from point B and in Table 4.5 for soil from point C. The gradation curves for points B and C are shown in Figures 4.45 and 4.46, respectively.

Table 4.4. Johnny Majors Practice Field project soil parameters at point B

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Compressive Strength (psf)
0–1.5	Poorly graded sand (SP)	-	-	-	-	-
1.5–10.0	Sandy clay (CL)	14.8	27.8	14.9	12.9	-
10.0–16.0#	Low plasticity clay (CL)	14.0	26.2	13.9	12.3	7019
16.0	Gravel layer	-	-	-	-	-
16.0–18.0*	Clayey sand (SC)	25.7	27.2	13.7	13.5	-
18.0–19.0	Stiff clayey sand (SC)	17.9	28.0	14.4	13.6	-

* Top of the 4 inch pipes at depth of 17 feet
#Depth of water table at 14 feet
-Laboratory tests not conducted

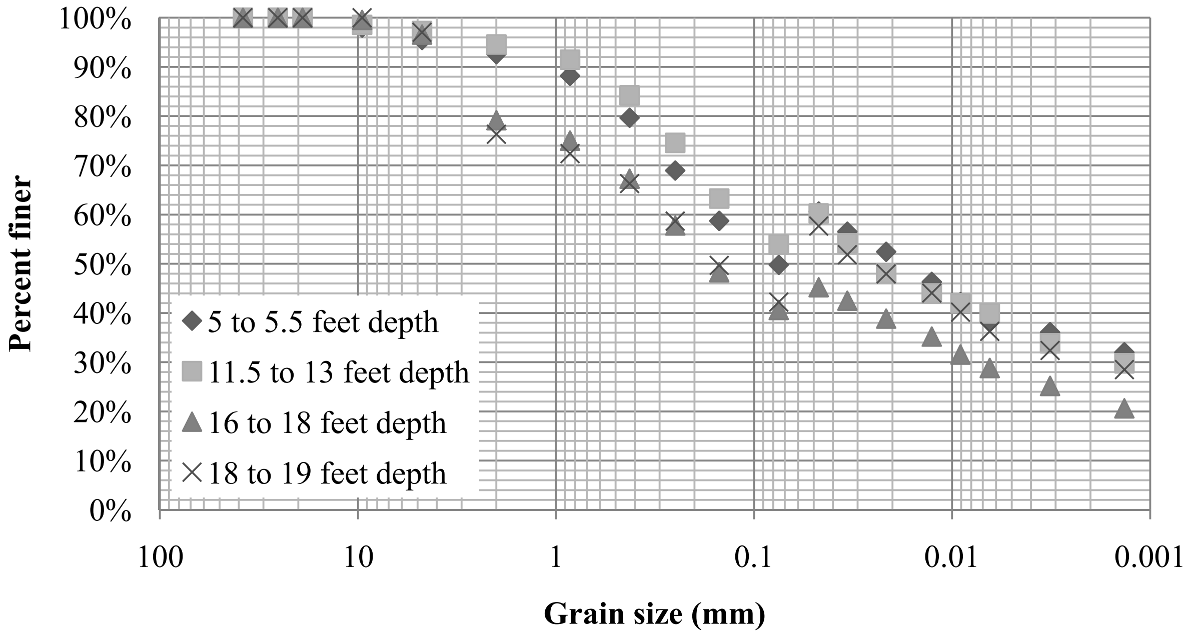


Figure 4.45. Soil gradation curves for point B

Table 4.5. Johnny Majors Practice Field project soil parameters at point C

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

* Top of the 4 inch pipe at depth of 17 feet
#Depth of water table at 14 feet

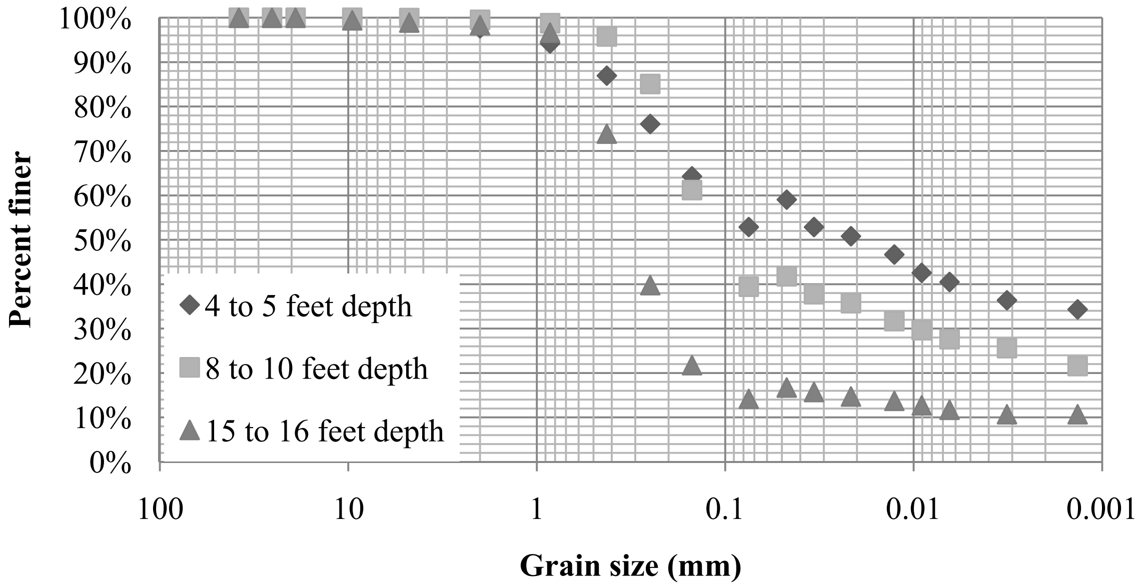


Figure 4.46. Soil gradation curves for point C

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 at point B was 7,019 psf, and the unconfined compressive strength of the soil at point C was 2,300 psf.

The consolidation test for a sample from point B at a depth of 11.5 to 12.1 feet indicated that the soil is normally consolidated, has a compression index of 0.17, and has a pre-consolidation pressure of 0.455 tons per square feet. Figure 4.47 is a consolidation test graph for the vertical effective stress versus void ratio for soil from depths of 11.5 to 12.1 feet at point B. The average coefficient of consolidation is 0.38.

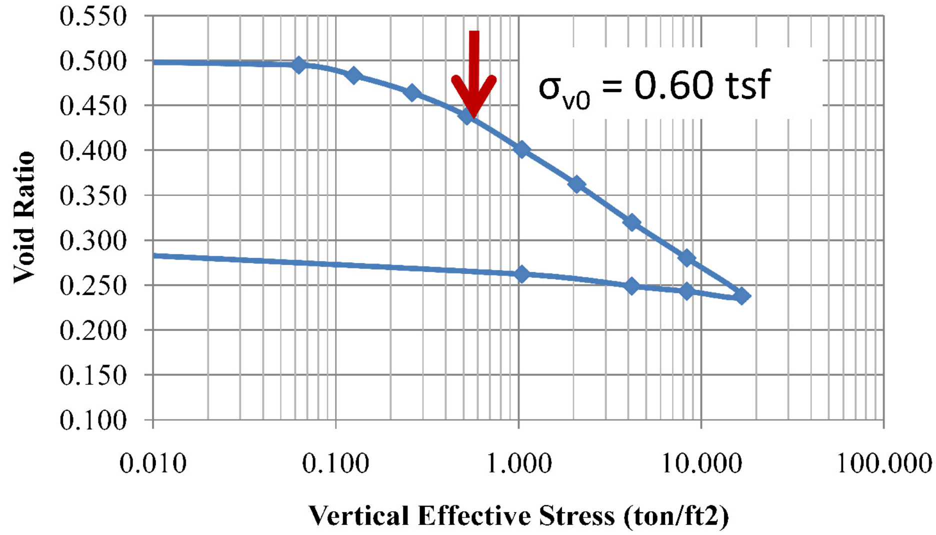


Figure 4.47. Vertical effective stress vs. void ratio for soil from depths of 11.5 to 12.1 feet at point B

The consolidation test for a sample from point C at a depth of 6.1 to 6.7 feet provided an overconsolidation ratio of 2.48, a compression index equal to 0.11, and a pre-consolidation pressure of 0.86 tons per square feet. Figure 4.48 shows a consolidation test graph for the vertical effective stress versus void ratio for soil from depths of 6.1 to 6.7 feet at point C. The average coefficient of consolidation is 0.46.

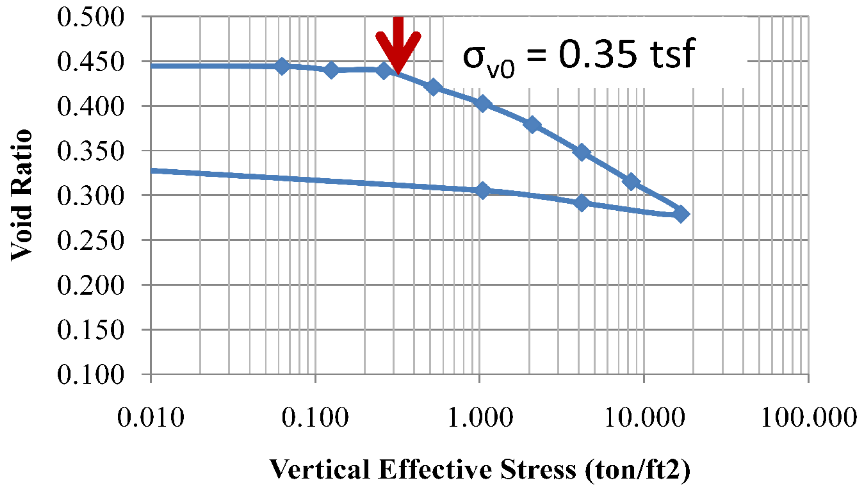


Figure 4.48. Vertical effective stress vs. void ratio for soil from depths of 6.1 to 6.7 feet at point C

A multistage consolidated undrained test was conducted on the soil at point B using confining pressures of approximately 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 s in Figure 4.49. 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 soil modulus was 70, 240, and 240 tsf at confining pressures of 3.6, 8.9, and 14.0 psi, respectively.

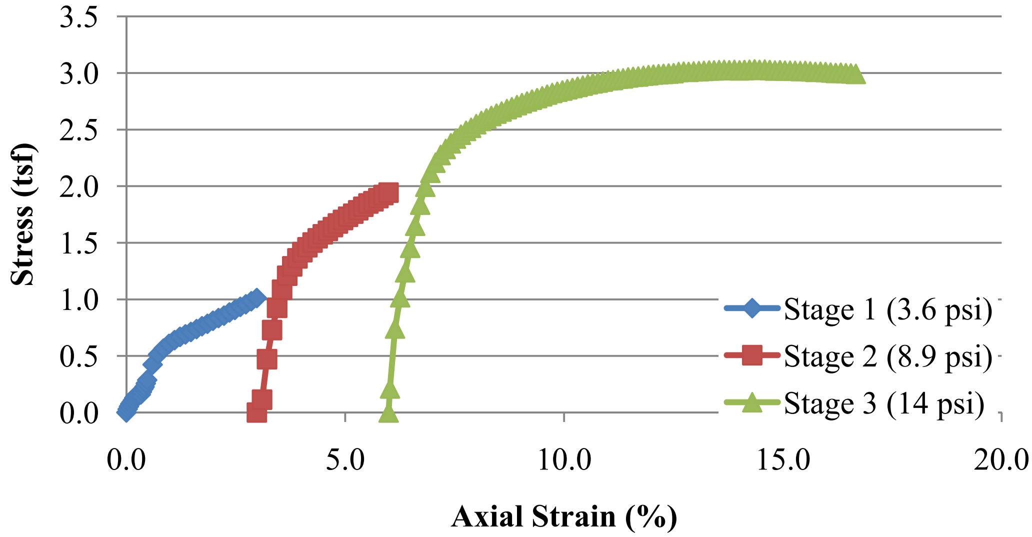


Figure 4.49. Multistage consolidated undrained test stress vs. strain for soil at point B

Results were found by comparing the q' vs. p' relationship for the point B soil (see Figure 4.50). The parameter α was found to equal 33.1°, and the intercept "a" was found to equal zero.

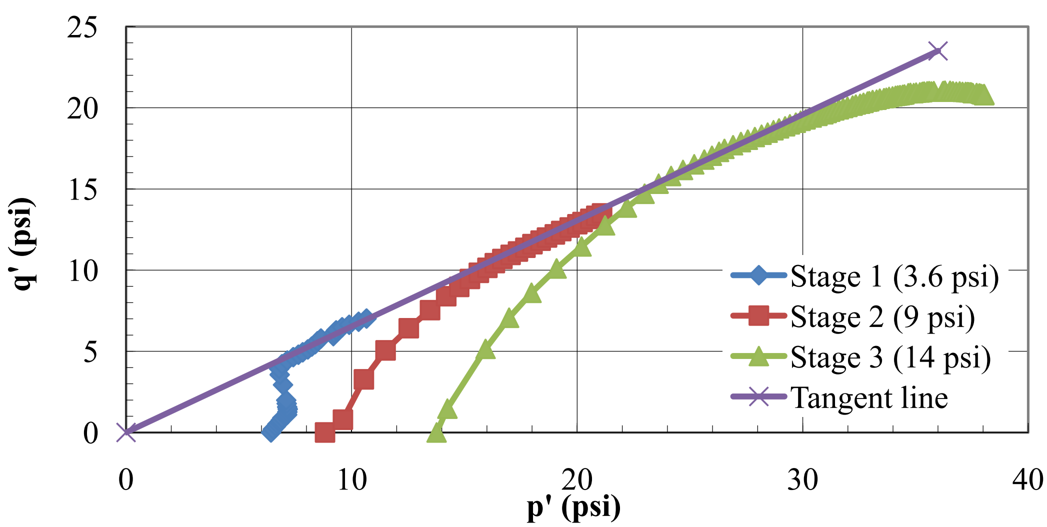


Figure 4.50. Multistage consolidated undrained test q′ vs. p′ for soil at point B

A similar multistage consolidated undrained test was conducted 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 shown in Figure 4.51.

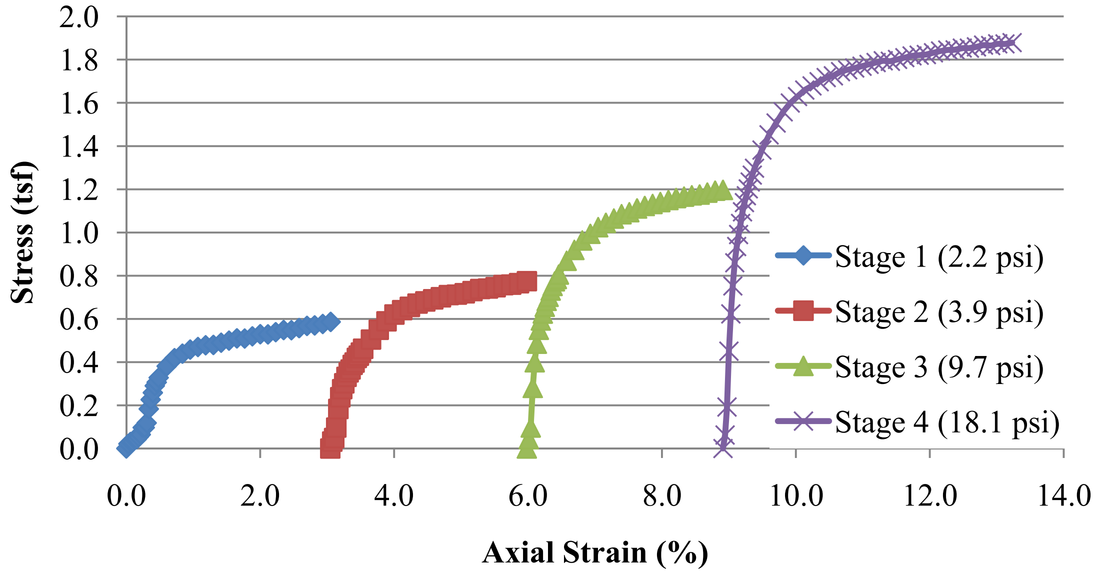


Figure 4.51. Multistage consolidated undrained test stress vs. strain for soil at point C

Results found by comparing the q′ vs. p′ relationship for the point C soil are shown in Figure 4.52. The parameter α was found to equal 33.3°, and the intercept "a" was found to equal 0.5 psi.

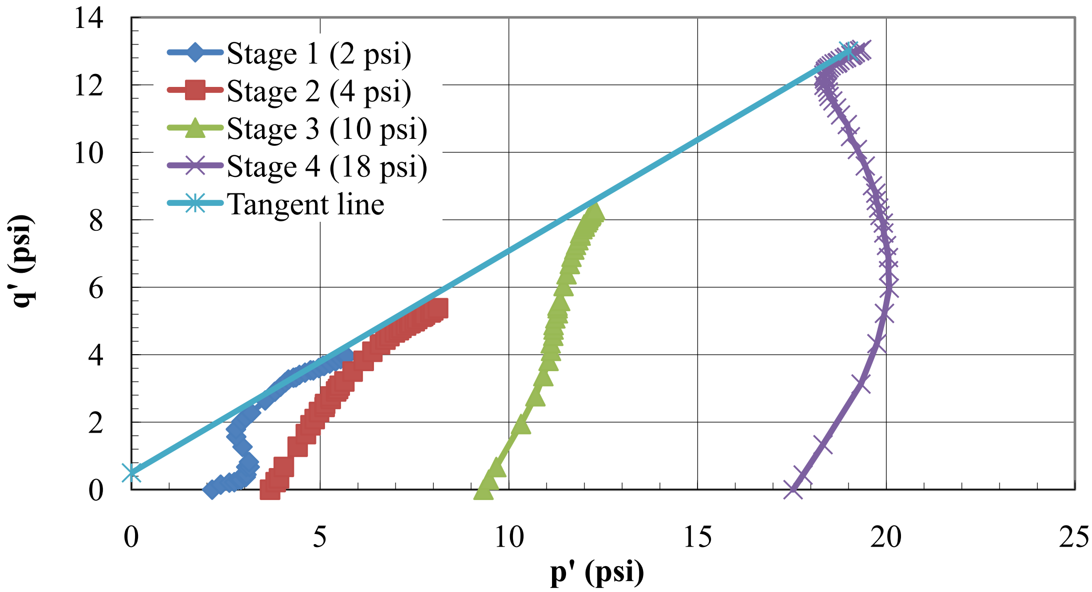


Figure 4.52. Multistage consolidated undrained test q′ vs. p′ for soil at point C

4.2.5.7 Key Findings

The Jack Trice Stadium renovations provide examples for using HDD to install HDPE electrical conduit in HDD bores. 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, frac-out, and surface heave.

A possible explanation for the frac-out and surface heave is that gravel collapsed into the borehole and sealed the borehole off in the direction of the exit pit. This collapse, combined with the clay cuttings that may have clogged the reamer, hindered fluid flow to the entry and exit pits. Because the contractor continued to pump drilling fluid, pressure built up, eventually resulting in the frac-out and surface heave. Similar circumstances soon led to issue, this time a suface heave without an accompanying frac-out. Compared to the areas where frac-outs occurred, the area where only the surface heave occurred had a more impermeable soil. Despite this, the veins of weakness created by the heave eventually gave way, and ultimately another frac-out occurred as the contractor continued to pump drilling fluid during times of slow reaming process.

These problems may not have occurred if a polymer drilling fluid that emulsifies clay been used instead of the sodium bentonite slurry that was selected because sodium bentonite is more effective for granular material. 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 4.53. However, the contractor used bentonite because of the concern of possibly encountering sand. A polymer slurry would not have been as effective as bentonite in stabilizing the borehole if sand were encountered, and a collapse could have resulted. Bentonite was considered to be a lower-risk option.

Figure 4.53. Clay-caked drill bit shown after emerging into the exit pit at point D

Despite several challenges, the installation was completed using HDD. This project serves as an example of HDD applications, but it also demonstrates some problems that can occur.

4.3 Field Monitoring

The field monitoring portion of the fieldwork involved observing projects and measuring soil stresses during construction. These investigations were more thorough compared to the investigations described in the “Site Visits” portion (Section 4.2) of this chapter. Undisturbed soil samples were recovered and tested in the laboratory. Samples were classified, and unconfined compression, consolidation, and multistage 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 4.54). Two of these instruments, each with a range of 25 psi, were purchased. This range was exceeded on several occasions; however, the manufacturer informed the research team that the effect would be a shift in baseline readings, which could be corrected by taking new zero readings after the pressure cells were removed. One note is that the pressure cells were designed to be installed by a conventional drill rig.

Figure 4.54. Geokon model 4830 push-in pressure cell (from Geokon)

A Campbell Scientific CR5000 datalogger was used to record the pressure cell readings. However, a Geokon model GK-404 handheld vibrating wire reader was used on the first two "Field Monitoring" projects while several software and hardware problems associated with the datalogger were resolved. When the handheld reader was used, readings were only recorded when a researcher was able to manually record them. The datalogger was used after the problems were resolved. The datalogger was programmed to take readings every 10 seconds, which was considered often enough to obtain a nearly continuous record of pressure variations.

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

Because most planned projects were in close geographic proximity to the exprimentation site and the soils at these sites were stiff clay glacial till, the pressure cells were tested at the Spangler Geotechnical Laboratory experimentation site in Ames, Iowa (a site with similar soil properties), in April 2008 to collect sample data (see Figure 4.55). An open area was selected where soil data were available from previous research (see Table 4.6). A conventional drill rig was brought on-site to install the instruments. The drill rig first drilled an instrumentation hole through the clayey soil to a depth of 5 feet, at which point the drill bit and auger were removed from the drill rig and push-in pressure cell A was attached to a drill rod. The drill rig then vertically pushed the pressure cell into the ground to a depth of 5 to 7 feet. Due to the instrument configuration, the sensors in the pressure cell were centered at a depth of 6.5 feet, which is a common depth for water main installations. A similar process was used to install pressure cell B. The pressure cells took readings for 27 days, which are shown in Figure 4.56.

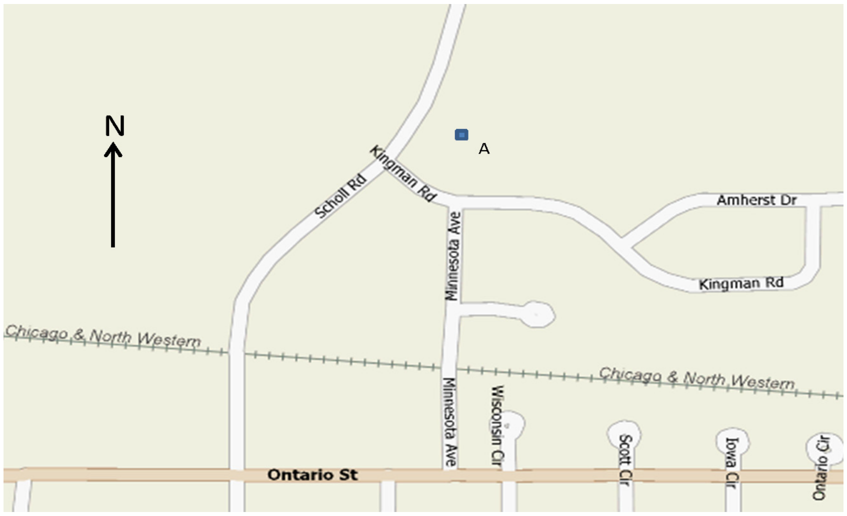


Figure 4.55. Location of pressure cell testing at Spangler Geotechnical Laboratory in Ames, IA

Table 4.6. Spangler Geotechnical Laboratory testing soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Compressive Strength (psf)
0–1.0	Dark gray silty sandy clay with organics#	12.5	-	-	-	-
1.0–5.0	Brown silty sandy clay#	12.3	-	-	-	-
5.0–25*	Orange-brown silty sandy clay #	13.6	-	-	-	-

* Location of water table is below this level
#Reported by prior research
-Test not conducted

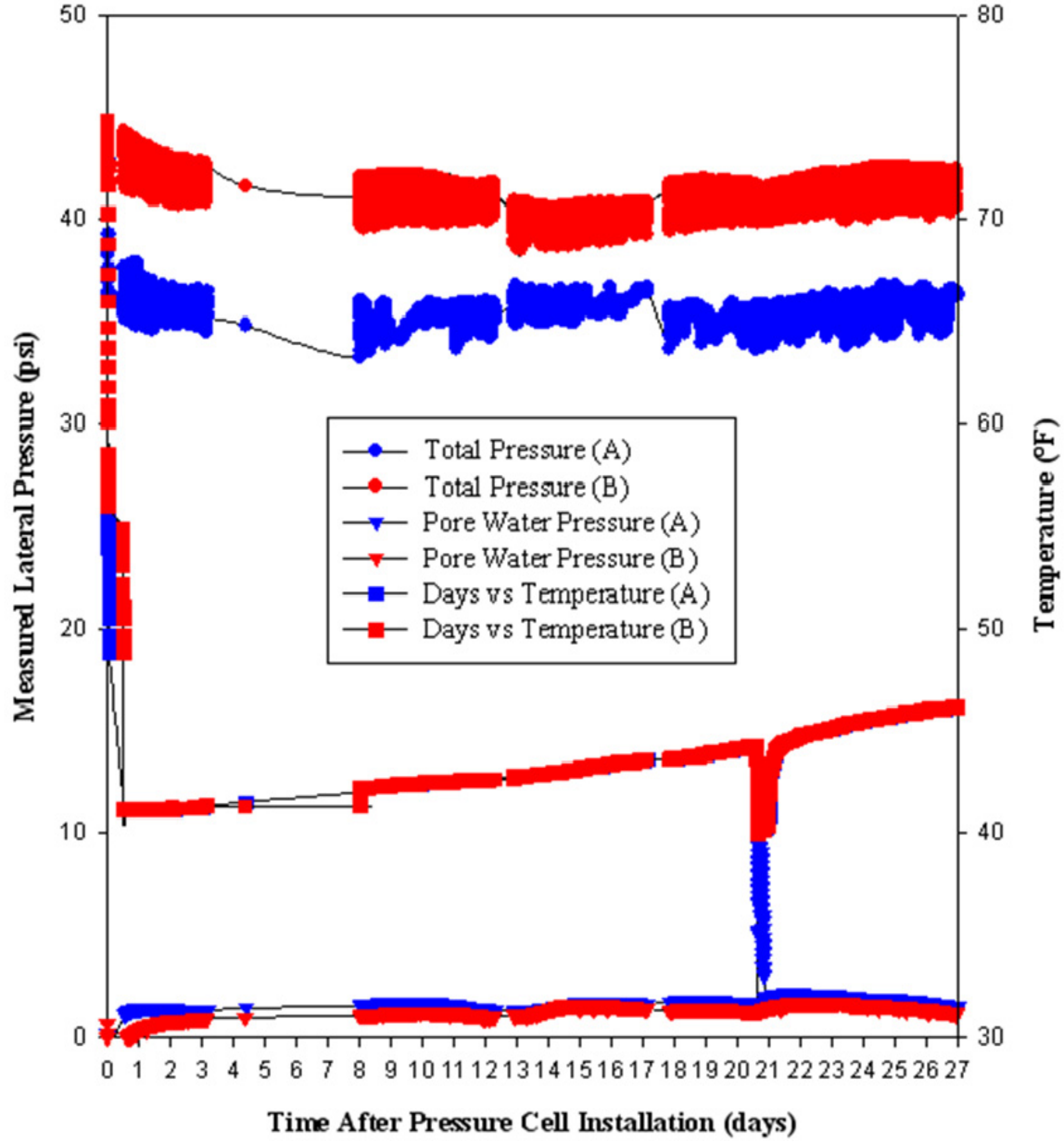


Figure 4.56. Total lateral earth pressure, pore water pressure, and temperature measured at depth of 6.5 feet

During the time of installation, the total pressure readings were observed to change around 2 to 3 psi for both instruments. The gaps in the data occurred due to problems with the datalogger. The pressure cells recorded a higher total pressure directly after installation and stabilized after approximately three days. After that, the pressure cells exhibited some minor drift over the long time period. These experimental data were used as a reference when examining the data that were recorded on actual jobsites.

Figure 4.56 shows that the pressure cells recorded a higher total pressure directly after installation, and the total pressures stabilized after two to three days. During the time of data recording, the total pressure readings changed around 2 to 3 psi for both instruments. For this specific installation, it is believed that these minor changes were caused by a loose connection between the pressure cells and the datalogger. Such changes were not observed in other installations where wires were connected more carefully. It was also observed that the pore water pressure stabilized after two days. The gaps in the data occurred because of problems with the datalogger. Because both the total pressure and pore water pressure readings stabilized approximately two days after installation, the push-in pressure cells were installed at the jobsite two days before the start of construction at most jobsites.

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 following information describes each of these projects.

4.3.1 Osborn Drive, Ames, Domestic Water, HDD

4.3.1.1 Project Information

This project was located along Osborn Drive on the ISU campus from late October through early November 2007. HDD was used to install an 8-inch-diameter HDPE domestic water pipe. The installed pipe was 920 feet long and was located at a depth of 6 feet to the top of the pipe, except when the bore had to be steered underneath an underground obstacle.

The project was completed using a series of three bores (see Figure 4.57): one for each straight line portion of the pipeline. The boring runs north of MacKay Hall and east of the Palmer Human Development and Family Studies (HDFS) Building on the ISU campus, as shown in Figure 4.58. This path includes a north-south bore approximately 85 feet long that crosses Osborn Drive (between points A and B in Figure 4.57), an east-west bore 495 feet long that runs along the north side of Osborn Drive (between points B and C in Figure 4.57), and a north-south bore 325 feet long that crosses under Osborn Drive and the parking lot between the Palmer HDFS Building and Bessey Hall (between points C and D in Figure 4.57). The research team installed a vibrating wire push-in pressure cell at point E to observe changes in lateral earth pressure during installation.

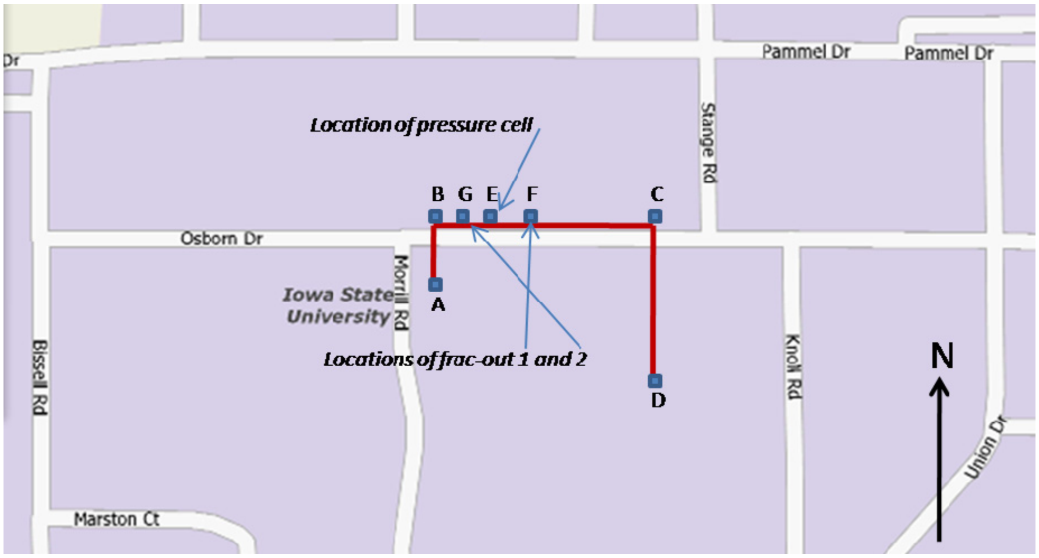


Figure 4.57. Location of HDD project (bore path in red) at Osborn Drive in Ames, IA

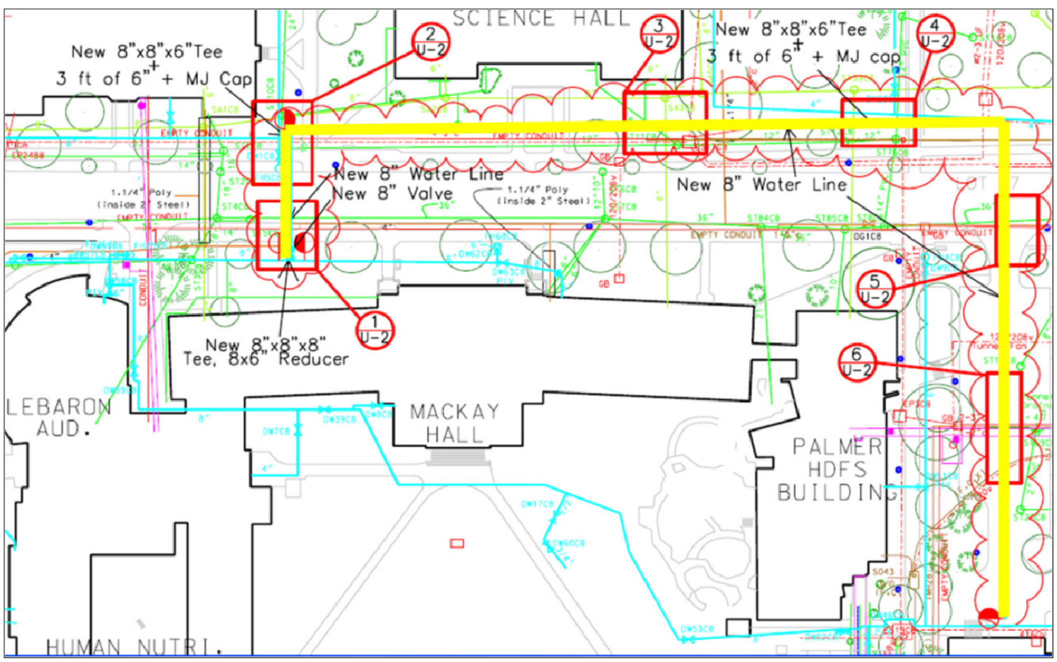


Figure 4.58. Plan view of the project site; bore path in yellow

4.3.1.2 Trenchless Method Selection

The owner selected trenchless construction to avoid open cutting across and along Osborn Drive, which is one of the main roadways within the ISU campus. Trenchless construction, specifically HDD, also allowed landscaping and sidewalks to remain intact during installation; the pipe was installed below sidewalks and several large trees. Additionally, although ISU classes were in session during construction, the installation proceeded with minimal disturbance to vehicles and pedestrians in the area.

4.3.1.3 Soil Conditions

Neither the contractor nor the owner conducted soil testing 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, below which was a deep stiff clay layer. Dewatering was not necessary because the boring was performed above the water table. This soil was considered appropriate for HDD construction.

4.3.1.4 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 points 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 depth of 6 feet. All existing utilities near the bore path, including a 6-foot-high steam tunnel, were manually located by potholing (see Figure 4.59). Potholing was done at the contractor's discretion because the contractor was required to repair any existing utilities damaged during the installation, unless the damaged utility was unmarked.

Figure 4.59. Potholing to verify position of nearby utilities

A drilling fluid consisting of BORE-GEL™ mix and water was used during the boring. This mix contained sodium bentonite, a clay-inhibiting polymer, and soda ash. BORE-GEL™ is marketed as improving 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 pushing the bit into the ground. As the pilot bore proceeded, periodic adjustments were made to the depth and direction to keep the bore online and to avoid nearby utilities. The borehole was drilled to a depth of about 6 feet to the expected top of pipe, except where vertical steering was necessary to avoid existing pipe. Figure 4.60 shows the position of the drill rig at point B while pilot boring from point B to point C. The pilot bore was successfully conducted, and the drill bit emerged in the exit pit after about two hours.

Figure 4.60. Directional drilling machine

The 4-inch drill bit from the pilot bore was removed from the drill string and replaced by a 14-inch reamer for the pre-reaming stage (see Figure 4.61). The directional drilling machine was switched from push mode to pull mode, and the drill string with the reamer attached was pulled back through the pilot bore toward the directional drilling machine. The reamer, which enlarged the hole from 4 inches to 14 inches in diameter, was pulled back using the existing pilot bore for guidance.

Figure 4.61. Fourteen-inch reamer

The pre-reaming stage continued until frac-out of the drilling fluid was observed directly above the bore path at point F on Figure 4.57, a distance of 30 feet east of the pressure cell located at point E. Drilling fluid under pressure ruptured the outer walls of the borehole and flowed to the ground surface, where it was observed as a mud puddle. Drilling stopped while the excess drilling fluid was vacuumed. The pre-reaming 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 was started by pushing the drill string from the drilling machine to the exit pit, which was quickly accomplished because the 2-inch-diameter drill rods were easily pushed through the 14-inch-diameter borehole that had been pre-reamed the previous day. Once the drill string emerged at the exit pit, the same 14-inch reamer used before and the 8-inch HDPE pipe were attached. The reamer cleaned out the borehole and ensured that the borehole diameter was 14 inches. Additionally, the reamer cleared any closure of the borehole that may have occurred overnight.

The pipe installation commenced as the drilling machine pulled the reamer and pipe back through the hole. As the installation progressed, periodic pauses were necessary so that 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 4.62 shows one of the heat-welded joints, and Figure 4.63 shows the pipe segments being pulled into the borehole.

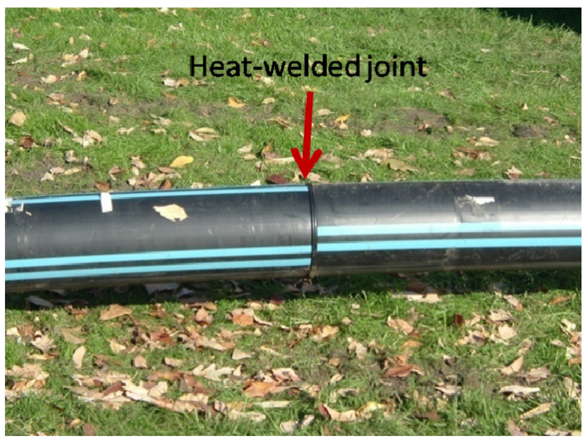


Figure 4.62. Heat-welded HDPE pipe joint

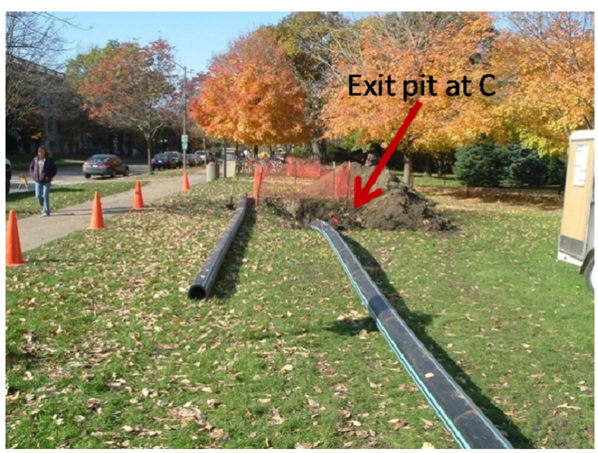


Figure 4.63. HDPE pipe being pulled from exit pit at point C to launching pit at point A

The pipe installation continued as expected until another frac-out occurred at point G, approximately 30 feet east of the pressure cell at point E. This point was about 100 feet 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 that propagated to the ground surface. The installation was again halted, and a vacuum was used to remove the drilling fluid from the ground surface (see Figure 4.64).

Figure 4.64. Drilling fluid at ground surface resulting from frac-out

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 pullback was decreased to keep drilling fluid pressure low enough to prevent further frac-outs. 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 was located in an asphalt parking lot between the Palmer HDFS Building and Bessey Hall. Potholing was also necessary in several spots at this location, so small cuts were made through the asphalt.

Once it was verified that no existing utilities lay near the path of the planned bore, the contractor installed the 8-inch HDPE pipe using the same procedure as the first bore. This installation was successfully conducted, and surface heave, frac-out, or settlement was not observed.

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

4.3.1.5 Lateral Earth Pressure Monitoring

To characterize the soil at this site, the research team drilled two vertical test holes near the bore path using a conventional augered drill rig. Disturbed soil samples were taken at various depths, and a 3-inch-diameter thin-walled Shelby tube was pushed through the bottom of the first hole 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 bagged samples were tested in the laboratory.

Additionally, a third vertical instrumentation hole 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 4.65). This pressure cell was then vertically pushed into the ground by the drill rig to a depth of 5 to 7 feet (see Figure 4.66). The flat side of the pressure cell was parallel to the planned horizontal borehole. The sensors in the pressure cell were centered at a depth of 6.5 feet, matching the intended HDD installation depth. 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 4.67). The contractor's pilot bore commenced one hour after the research team finished installing the pressure cell.

Figure 4.65. Push-in pressure cell before installation

Figure 4.66. Push-in pressure cell after installation

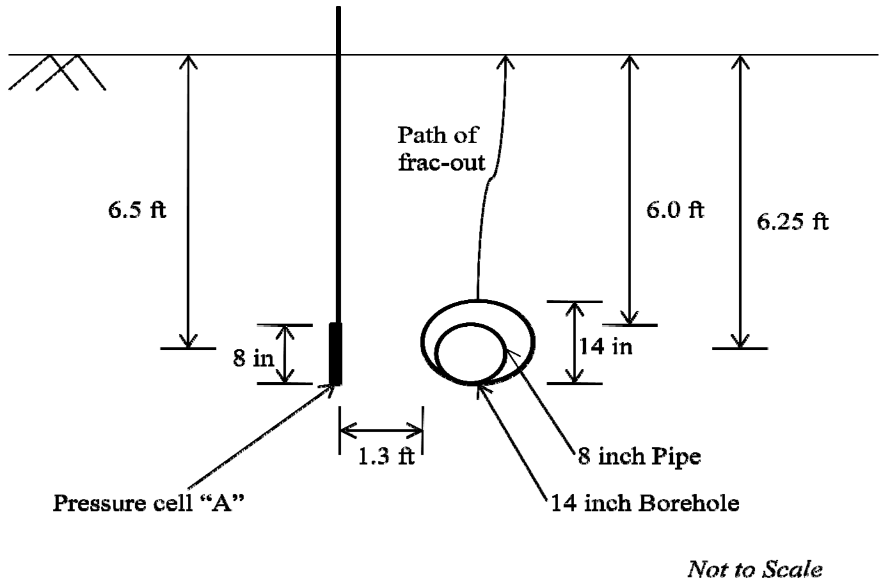
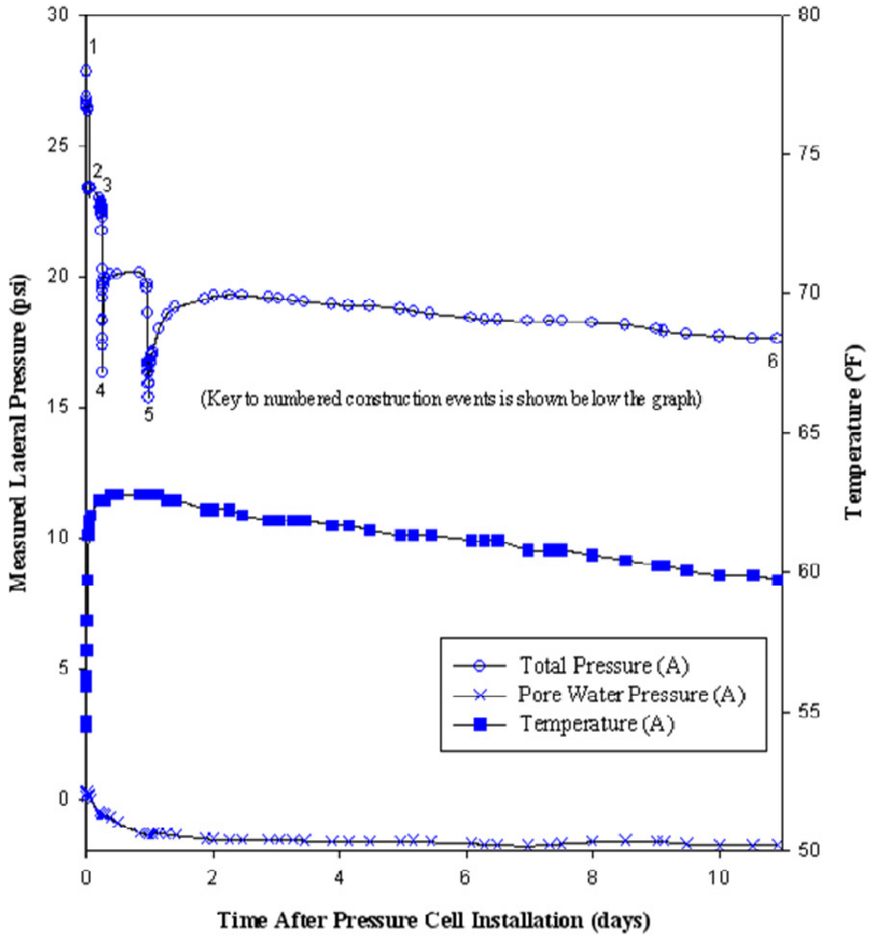


Figure 4.67. Profile of pressure cell and borehole (looking east)

Using the push-in pressure cell, the total lateral earth pressure, piezometric pressure, and temperature were recorded during the initial east-west boring process from B to C. Due to a programming problem with the datalogger, a handheld vibrating wire reader was used to take the readings, which were then manually recorded. Because of the manual recording process, fewer readings were recorded than would have been possible if a datalogger had been used.

Figure 4.68 shows that the lateral earth pressure changes recorded when the boring implement passed resulted in a decrease in pressure instead of an expected increase. The pressure cells registered changes of -3.0 psi for the pilot bore, -6.3 psi for the pre-reaming, and -4.3 psi for the pipe installation. The research could not explain this counterintuitive observation.

Additionally, the two incidences of frac-out of the drilling fluid were not accompanied by a measured pressure increase in the surrounding soil. This observation indicates that the fluid may have followed preexisting cracks in the soil to reach the surface rather than experiencing the pressure buildup required to fracture the borehole walls.



Key to Numbered Construction Events	
1)	Pressure cell A was installed to read at a depth of 6.5 feet.
2)	Four-inch pilot bore passes the pressure cell with its center at a depth of 6 feet and a lateral distance to the pressure cell of 1.3 feet.
3)	Fourteen-inch reamer pullback was begun and then paused before reaching the pressure cell due to frac-out.
4)	Fourteen-inch reamer passed 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)	Fourteen-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 was removed.

Figure 4.68. Total lateral earth pressure, pore water pressure, and temperature measured 1.3 feet from bore centerline

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

4.3.1.6 Soil Characterization

Recovered soil samples included one 3-inch-diameter and one 2-inch-diameter Shelby tube sample 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 soil 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 4.69. The soil profile between the ground surface and the pipe locations consists of two soil layers. The first layer, between the ground surface and a depth of 1.5 feet, consists of clay with organic 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 4.7. Osborn Drive second HDD project soil parameters

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

*Top of the 8 inch pipe at 6 foot depth,
Location of water table is below this level
-Test not conducted

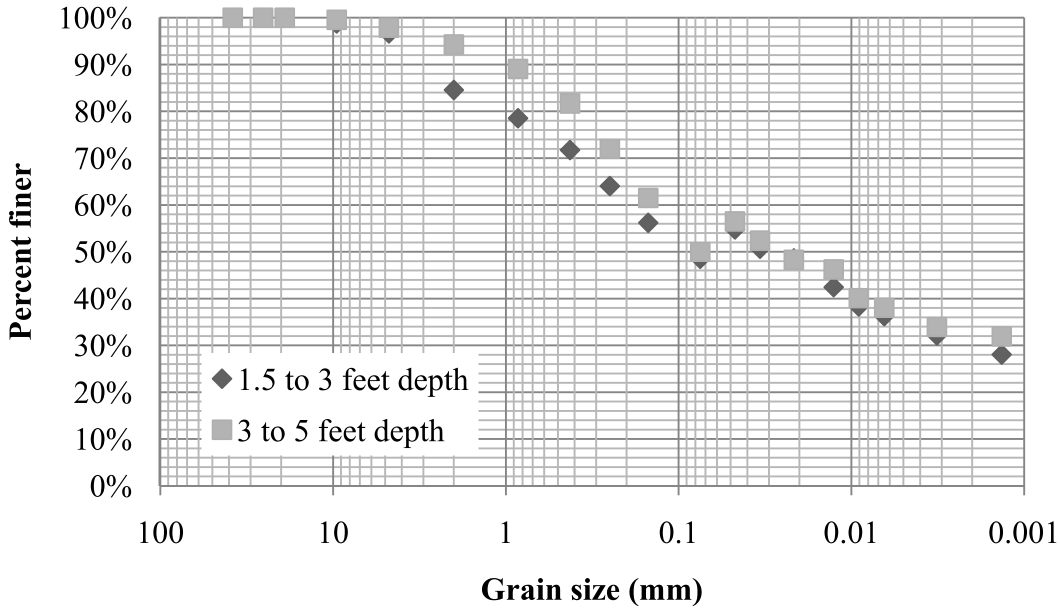


Figure 4.69. Soil gradation curves for depths of 1.5 to 3 feet and 3 to 5 feet

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 over-consolidation ratio of 3.2, a compression index equal to 0.17, and a pre-consolidation pressure of 1.04 tons per square feet (see Figure 4.70). The average coefficient of consolidation is 0.70.

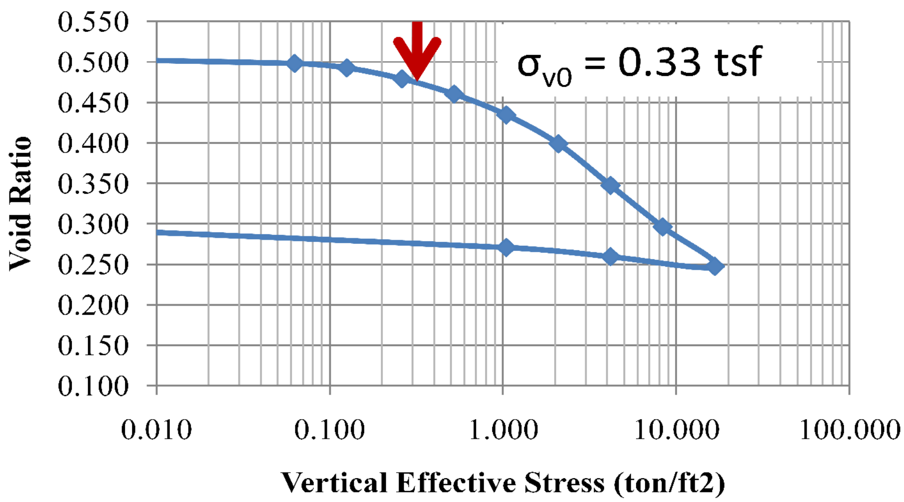


Figure 4.70. Consolidation test graph showing vertical effective stress vs. void ratio

The multistage 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 4.71. 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.

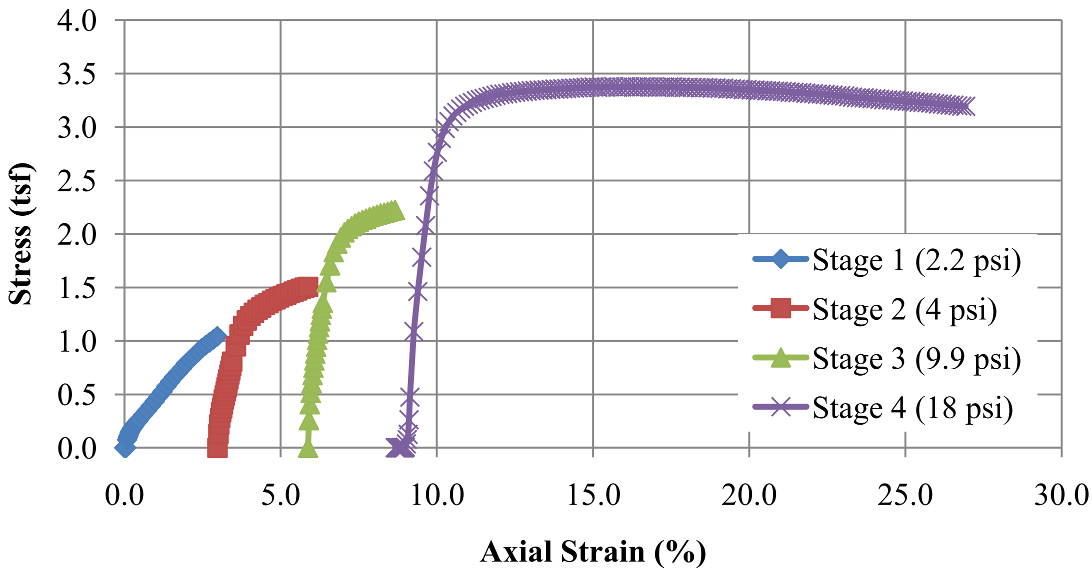


Figure 4.71. Stress vs. strain curves from multistage consolidated undrained test

Results given by comparing the q' vs. p' relationship is given in Figure 4.72. The graph indicates that α equals 29.3° and "a" equals 1.0 psi.

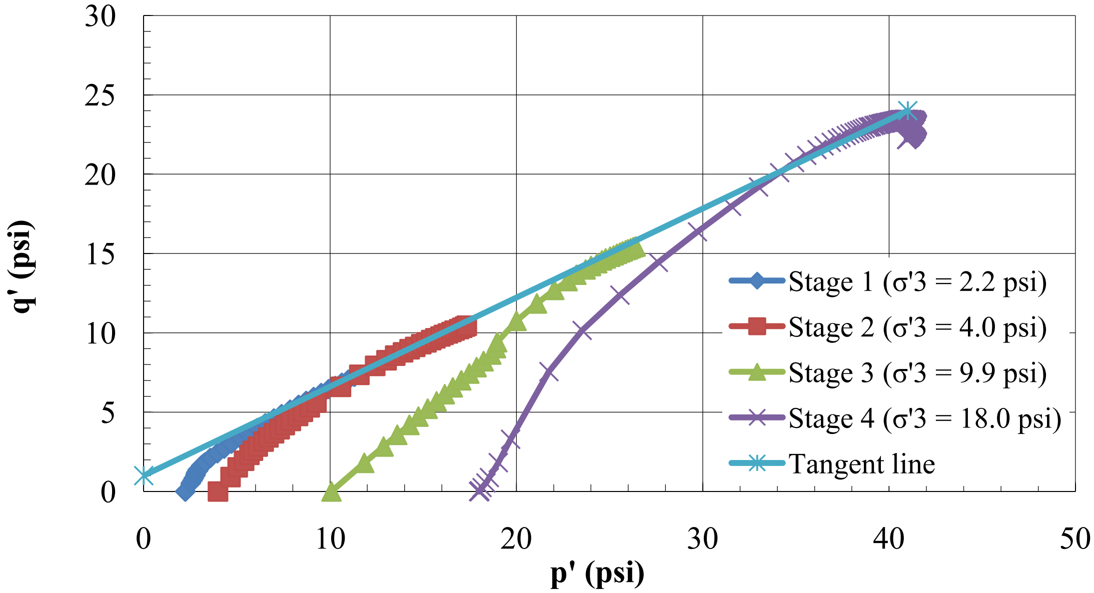


Figure 4.72. q’ vs. p’ curves from multistage consolidated undrained test

4.3.1.7 Key Findings

This project provided a good example of using HDD to install HDPE water pipe. A 920-foot-long pipe was installed at a depth of 6 feet to the top of the pipe through sandy lean clay soil. The 8-inch-diameter pipe was installed by drilling a pilot bore, pre-reaming with a 14-inch reamer, and attaching the reamer to the product pipe that was pulled through the hole. The bore was successfully completed; however, there were two instances of frac-out that 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 from this instrument show clear changes in the pressure readings as the boring proceeded past the pressure cell. These results indicate that the boring had an effect on the surrounding soil.

The soil was classified as sandy lean clay, which is an appropriate soil type for HDD. Frac-out could have been caused by one or more 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 to breaching the borehole walls, (3) a lack of stability of the borehole walls contributed to the problem, and/or (4) the speed with which the reamer was pulled through the borehole may have affected the soil stability. However, while any of these factors were possible, it is impossible to positively identify the cause of the frac-out.

The installation was successfully completed using HDD technology. The only significant problems encountered were due to frac-out. However, because the frac-out 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 HDD technology, while also demonstrating problems that may arise.

4.3.2 Seed Science Building, Ames, HDD

4.3.2.1 Project Information

This project was located on the ISU campus during November 2007. HDD was used to install two parallel 6-inch-diameter HDPE chilled water pipes. Each pipe was 240 feet long and was installed at a depth of 6 feet to the top of the pipe, except when the bore had to be steered below an underground obstacle.

The pipes were installed by completing two parallel bores (see Figure 4.73): one for each pipeline. The bore paths run north of the Seed Science Building across Wallace Road to the east, as shown in Figure 4.74. The research team installed two vibrating wire push-in pressure cells for this project at point B in Figure 4.73 to observe changes in lateral earth pressure during installation.

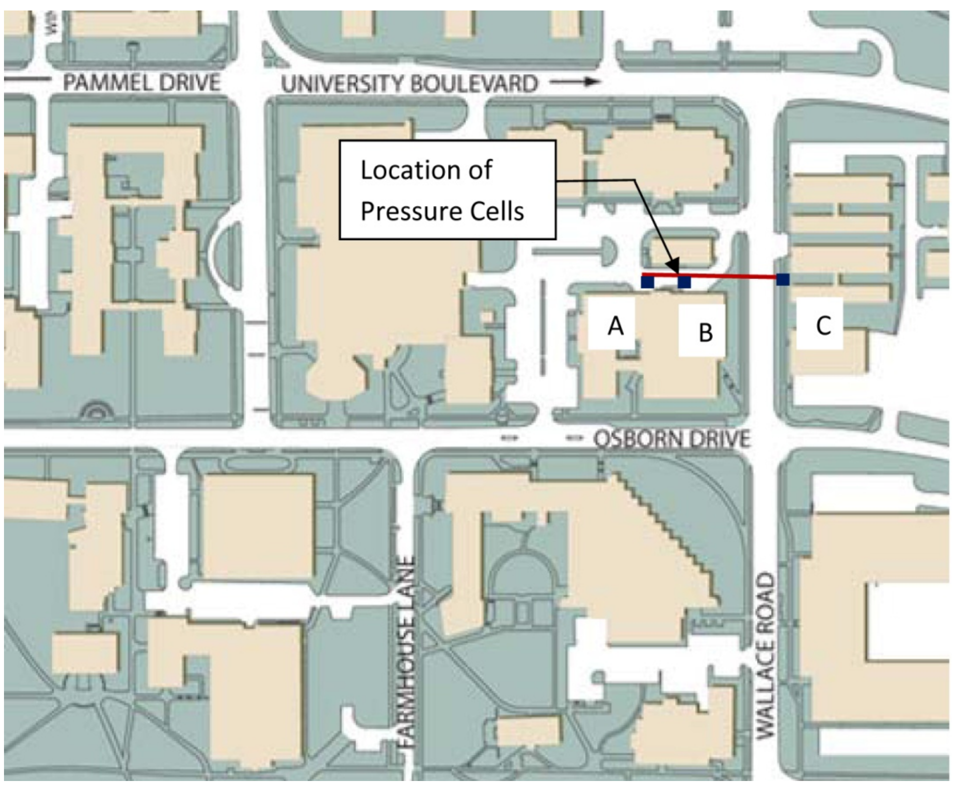


Figure 4.73. Location of HDD project (bore path in red) across Wallace Road in Ames, IA

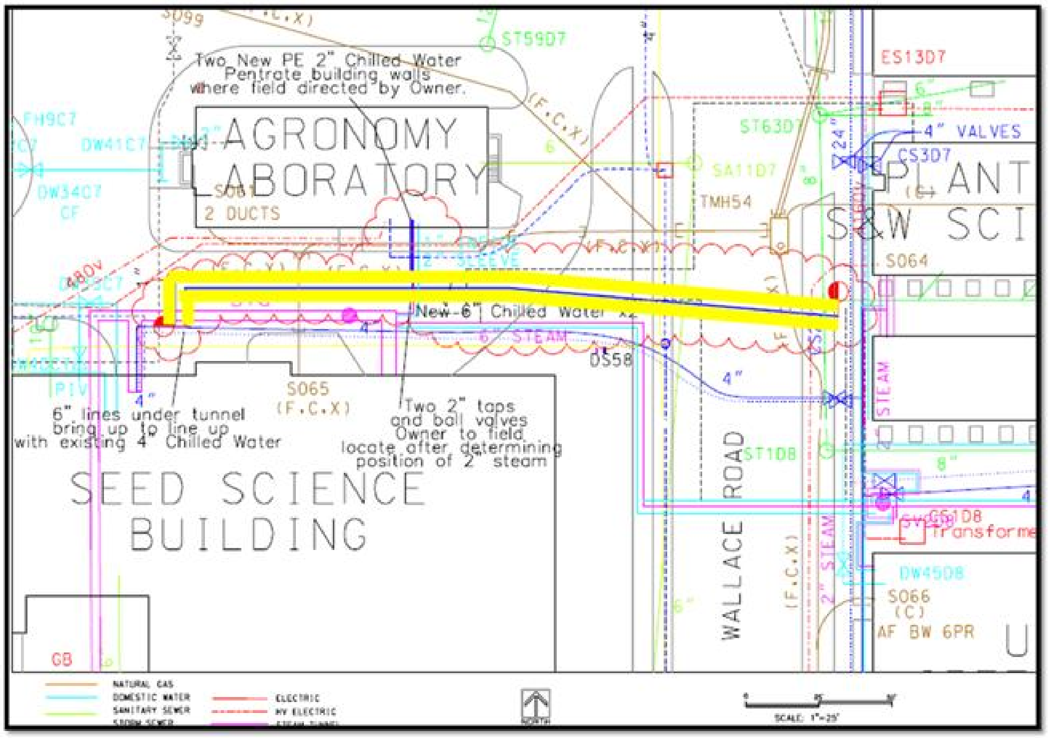


Figure 4.74. Plan view of project site; bore path in yellow

4.3.2.2 Trenchless Method Selection

The owner selected trenchless construction so that Wallace Road and the sidewalks along it could remain open. Open-cut trenching would have closed the road and sidewalks and forced motorists and pedestrians to detour around the area. Also, numerous existing utility pipes crossed the proposed bore path; HDD was considered an appropriate method to avoid these obstacles.

4.3.2.3 Soil Conditions

The project specifications gave the contractor the option to conduct soil testing. However, the contractor considered soil testing unnecessary because of general familiarity with the sandy lean clays common to the area. An exit pit dug at point C prior to boring also provided the contractor with information regarding soil conditions (see Figure 4.75). Figure 4.75 also shows pipe sections and the pipe welding machine at the site.

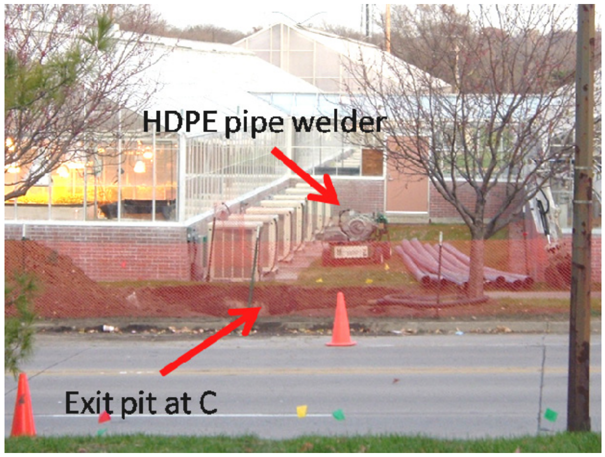


Figure 4.75. Looking east across Wallace Road toward exit pit at point C

Dewatering was not necessary because the boring was performed above the water table. These soil conditions were considered appropriate for HDD.

4.3.2.4 Trenchless Installation

The contractor removed the asphalt surface in the driveway near the site where the boring would begin (point A in Figure 4.74), and all nearby utilities were located to confirm that the intended bore path was a safe distance away. Potholing was done at the contractor's discretion because the contractor, unless the damaged utility was unmarked, was required to repair any existing utilities that were damaged during the installation. An exit pit was excavated at the planned termination site (point C) to allow the pipe to emerge at the proper 6-foot depth (see Figure 4.75). The HDD rig was then set up at point A facing east toward point C.

A drilling fluid consisting of BORE-GEL™ mix and water was used for the boring. This mix contained sodium bentonite, a clay-inhibiting polymer, and soda ash. This product is marketed 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 4.76) and using the machine to push the drill bit into the ground. The pilot bore proceeded with periodic adjustments to the depth and direction to keep the bore online (see Figure 4.77). The borehole was drilled to a depth of about 6 feet to the expected top of the pipe. The pilot bore was successfully conducted, and the drill bit emerged in the exit pit at point C.

Figure 4.76. Pilot bore drill bit attached to drill rig

Figure 4.77. Directional drilling machine

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 pre-reaming stage (see Figure 4.78). 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, enlarging the hole from 4 inches to 10 inches in diameter. The pullback was paused after every 80 feet (five 16-foot-long drill rods), and the reamer was pushed back toward point C to clean out the hole and to help prevent frac-out. The pullback was then resumed. The pre-reaming 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.

Figure 4.78. Ten-inch reamer

Next, the 10-inch reamer was reattached to the drill string and the 6-inch-diameter HDPE pipe was attached behind the reamer. Pipe in 20-foot sections was butt-fused together to reach the total length of 280 feet. The pipe and reamer were then pulled into the borehole at point C. The reamer in front of the pipe cleaned out the borehole and ensured that the diameter was a full 10 inches. The pipe was successfully installed without any problems. The pipe was then capped until it was 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 pipe, 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 feet, and the same 4-inch-diameter pilot bore drill bit and 10-inch-diameter reamer were used. This bore was also successfully completed with no problems (see Figure 4.79).

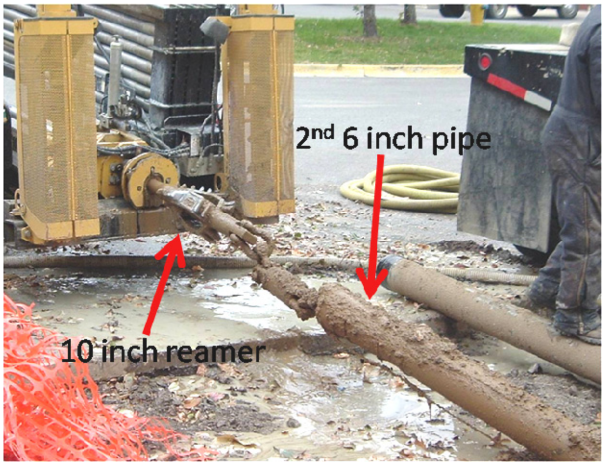


Figure 4.79. Second pipe emerges at point A and reamer is visible in front of the pipe

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

4.3.2.5 Lateral Earth Pressure Monitoring

In order to characterize soil at this site, the research team drilled two vertical test holes near the bore path using a conventional augered drill rig. Disturbed soil samples were collected at various depths, and a 3-inch-diameter thin-walled Shelby tube was pushed through the bottom of both holes 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 relatively high sand content of the soil. The soil lacked the cohesion necessary for the sample to remain in the tube during extraction. The bagged disturbed soil samples, on the other hand, were tested in the laboratory.

Two vibrating wire push-in pressure cells were installed at point B. To install them, a hole was first drilled at a lateral distance of 3 feet from the planned bore path (see Figure 4.80). This instrumentation hole was drilled to a depth of 5 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 drill rig then vertically pushed the pressure cell into the ground to a depth of 5 to 7 feet. This procedure placed 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,” at a depth of 6.5 feet but at a lateral distance of 4.6 feet away from the planned bore path. Pressure cell B was located 3 feet away from pressure cell A so that they were located as close as possible while still leaving a buffer (see Figure 4.81). This buffer was to ensure that the soil deformations induced by the installation process for one pressure cell would not affect the other pressure cell’s readings.

Figure 4.80. Drilling a borehole through asphalt

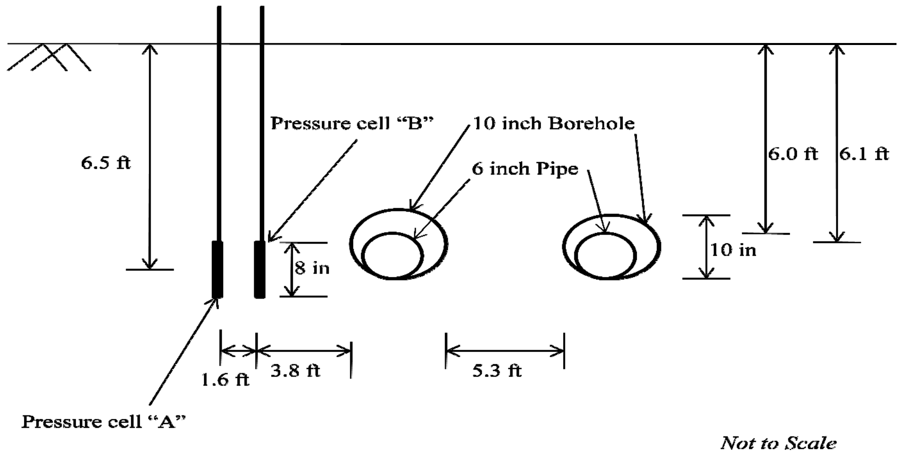


Figure 4.81. Cross-section showing pressure cells and borehole (looking west)

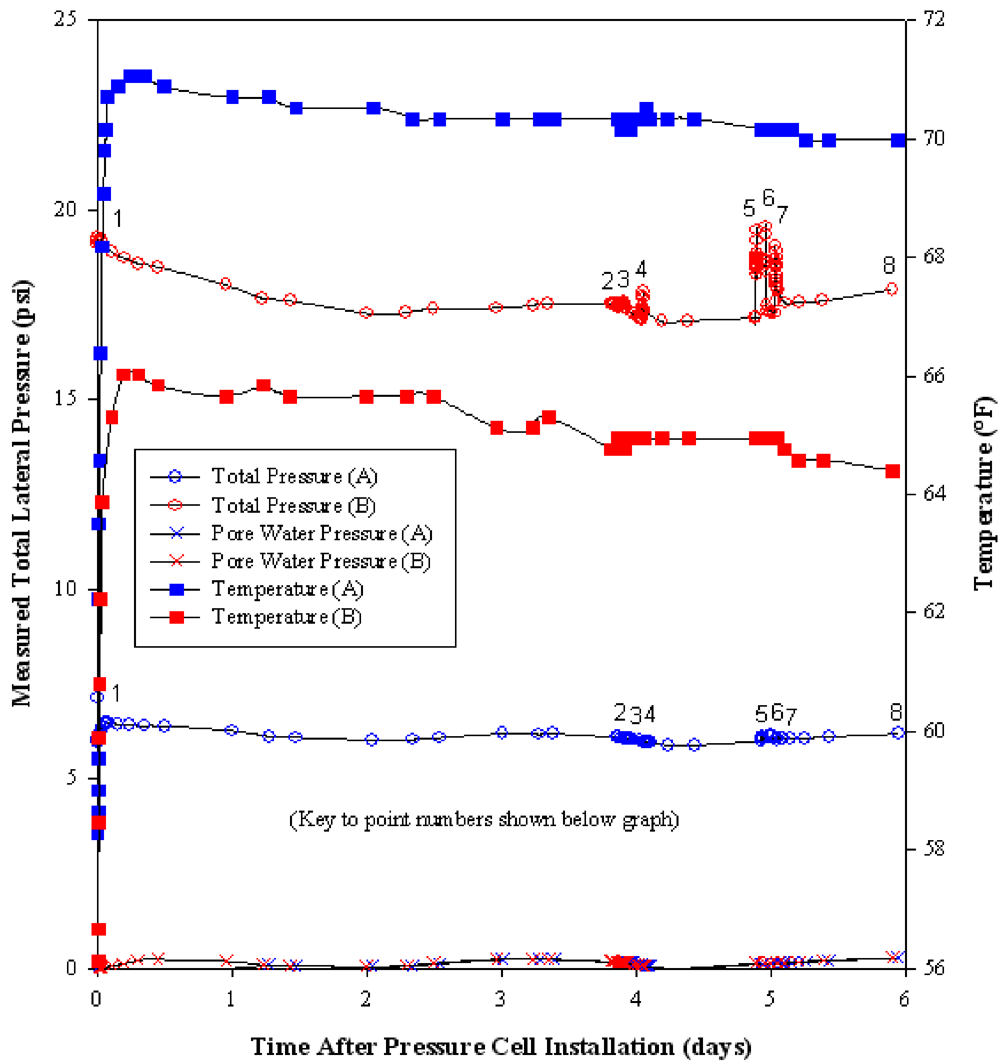
The two vibrating wire push-in pressure cells recorded the total lateral earth and piezometric pressures and the temperature during the 280 foot boring processes from point A to point C. Due to a programming problem with the datalogger, a handheld vibrating wire reader was used to take the readings, 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 to capture the effects of the directional drilling process on the lateral earth pressure of the surrounding soil.

The pressure cells were installed four days before the HDD work began, which was considered an adequate amount of time to allow the localized pressure increases due to soil deformations induced by the pressure cell installation 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 Figures 4.82 and 4.83). Note that in Figure 4.82, the visible height difference between the two drill rods only reflects the available rod sizes during installation. The research team 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 of 9.9 feet to the center of bore, 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 of 11.5 feet 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.

Figure 4.82. Push-in pressure cells in place at point B (looking south)

The installation of the second pipe produced higher readings because it was closer to the pressure cells. During the second installation, pressure cell B, reading at a depth of 6.5 feet and a distance of 3.8 feet to the center of bore 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 of 5.4 feet to the center of bore, recorded a slight pressure increase of 0.1 psi. The 10-inch reamer for the second 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.



Key to Numbered Construction Events	
1)	Pressure cells A and B were installed to read at a depth of 6.5 feet.
2)	Four-inch pilot bore for the first pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B.
3)	Twelve-inch reamer for the first pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B.
4)	Twelve-inch reamer with the first 6-inch product pipe passes pressure cells with its center at a depth of 6 feet and a lateral distance of 11.5 feet to A and 9.9 feet to B. The pipe was installed at a depth of 6 feet to the top of pipe.
5)	Four-inch pilot bore for the second pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B
6)	Twelve-inch reamer for the second pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B
7)	Twelve-inch reamer with the second 6-inch product pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 5.4 feet to A and 3.8 feet to B. The pipe was installed at a depth of 6 feet to the top of pipe.
8)	Pressure cells A and B were removed

Figure 4.83. Total lateral earth pressure, pore water pressure, and temperature measured 3.8, 5.4, 9.5, and 11.5 feet from centerlines of bores

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 these properties relate to the HDD process.

4.3.2.6 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 4.8. The 6-inch-diameter 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 and 7 feet. The gradation curve is shown in Figure 4.84. The water table was located deeper than 7 feet.

Table 4.8. Ames Wallace Road project soil parameters

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

*Top of the 6 inch pipe at 6 foot depth,
Location of water table is below this level
-Test not conducted

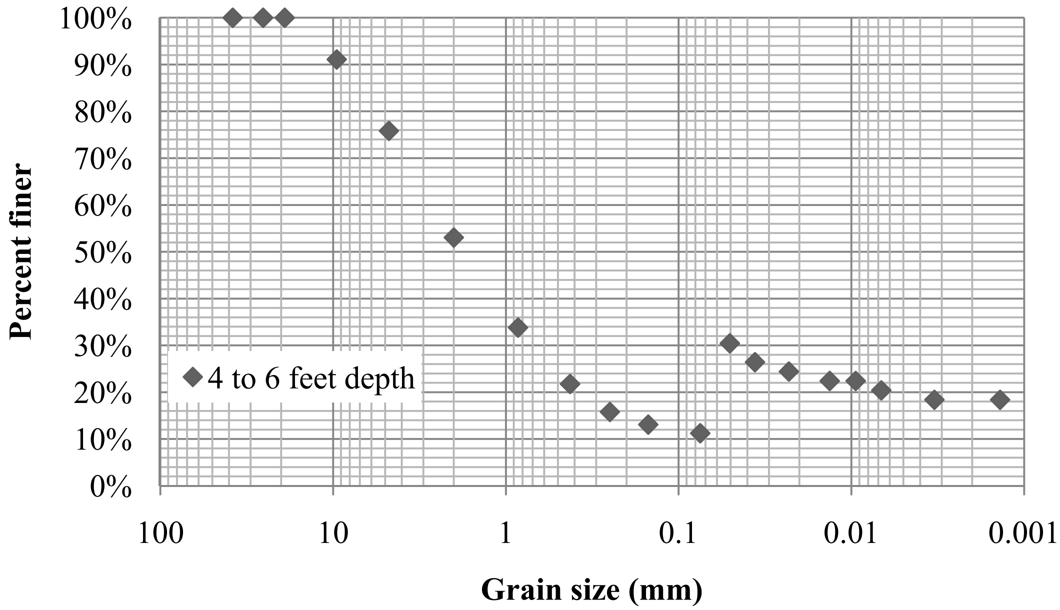


Figure 4.84. Soil gradation curves for depths of 4 to 6 feet

4.3.2.7 Key Findings

HDD was used for this project to install two parallel 6-inch-diameter HDPE chilled water pipes. Each pipe was 280 feet long and installed at a depth such that the top of the pipe was 6 feet deep. 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 drilled, followed by pre-reaming 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 pre-reaming stage. These results indicate that the boring process had an effect on the surrounding soil.

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 HDD.

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

4.3.3 The Hub, Ames, HDD

4.3.3.1 Project Information

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

The pipes were installed by completing two parallel bores (see Figure 4.85): one for each pipeline. The bores ran northeast of the Hub, which is a building that houses vending machines in the middle of the ISU campus, as shown in Figure 4.86. The research team installed two vibrating wire push-in pressure cells at point B in Figure 4.85 to observe changes in lateral earth pressure during installation.

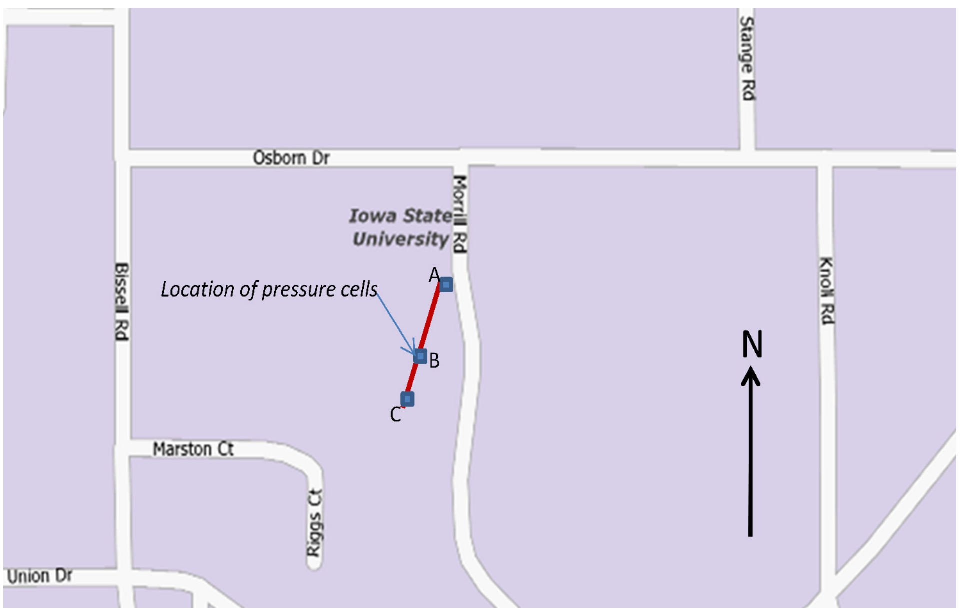


Figure 4.85. Location of HDD project (bore path in red) at the Hub in Ames, IA

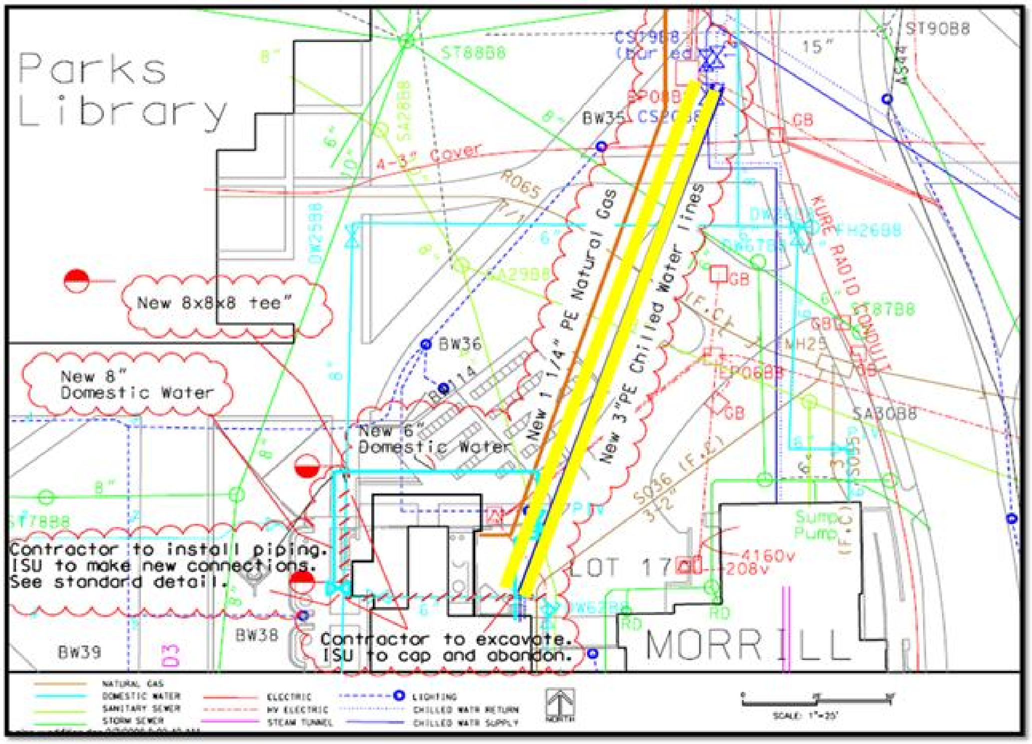


Figure 4.86. Plan view of project site; bore paths in yellow

4.3.3.2 Trenchless Method Selection

The owner selected trenchless construction to avoid the numerous existing utilities that crossed the planned bore path. Additionally, avoiding open-cut methods allowed several sidewalks to remain intact. Because the construction proceeded while university classes were in session, the continued use of these sidewalks was valuable to prevent disrupting the flow of pedestrian traffic through the middle of campus.

4.3.3.3 Soil Conditions

Neither the contractor nor the owner conducted soil testing prior to the HDD installation, mainly because of 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 because the boring was performed above the water table. This soil type was considered appropriate for HDD construction.

4.3.3.4 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 4.85) for the first bore going 200 feet northwest to southeast (from point A to point 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 the contractor was 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 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 installation. The polymer-based additive was WYO-VIS™, which is marketed 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 pushing it into the ground. The pilot bore proceeded with periodic adjustments to the depth and direction to keep the bore online and to avoid nearby utilities. The borehole was drilled to a depth of about 6 feet to the expected top of the pipe, except where vertical steering was necessary to avoid existing pipe. It was necessary to steer the bore down to a depth of 8 feet for at least two locations. The position of the drill rig at point A is shown in Figures 4.85 and 4.87. The pilot bore was completed successfully.

Figure 4.87. Directional drilling machine

The drill bit from the pilot bore was removed from the drill string and replaced by a 6-inch reamer for the pre-reaming stage. The directional drilling machine was switched from push mode to pull mode, and the drill string with the reamer attached was pulled back through the pilot bore toward the directional drilling machine. The reamer, enlarging the hole from 4 inches to 6 inches in diameter, was pulled back using the existing pilot bore for guidance. 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. Pipe was butt-fused together in 20-foot sections to reach the total length of 200 feet. The pipe and reamer were then pulled into the borehole at point C. The reamer in front of the pipe cleaned out the borehole and ensured that the diameter was a full 6 inches. A pit was then dug at point A to allow better access to the pipe end. The installation was successfully completed, and the pipe was then capped until it was 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 pipe, 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 successfully completed with no problems.

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

4.3.3.5 Lateral Earth Pressure Monitoring

The research team, using a conventional augered drill rig, drilled two vertical test boreholes near the bore path in order to characterize soil at this site. Disturbed soil 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 bagged disturbed samples were tested in the laboratory.

The two vibrating wire push-in pressure cells were installed at point B. To do this, an instrumentation hole was drilled at a lateral distance of 2 feet from the planned bore path and to a depth of 5 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 drill rig then vertically pushed the pressure cell into the ground to a depth of 5 to 7 feet. The cells were oriented so that the flat pressure-sensing side of the pressure cell was parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were centered at a depth of 6.5 feet, matching the intended depth of the HDD installation. A similar process was used to install the second pressure cell, denoted “B.” Pressure cell B was installed in a new instrumentation hole to take readings at a depth of 6.5 feet but at a lateral distance of 3 feet from the planned bore path (see Figure 4.88). Pressure cell B was located 3 feet away from pressure cell A so that they were located as close as possible while still leaving a buffer. This buffer ensured that the soil deformations induced by the installation processes would not affect the other pressure cell’s readings (see Figure 4.89).

Figure 4.88. Push-in pressure cell after installation

Figure 4.89. Push-in pressure cells in place

A problem arose prior to boring 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 discovery occurred after the pressure cells had been installed and meant that the borings would occur on their west sides rather than the east, resulting 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 4.90). Additionally, the bore nearest to the pressure cells was completed before the more distant bore, which would suggest that the readings for any lateral earth pressure increases caused by the second bore might be blocked by the completed first bore.

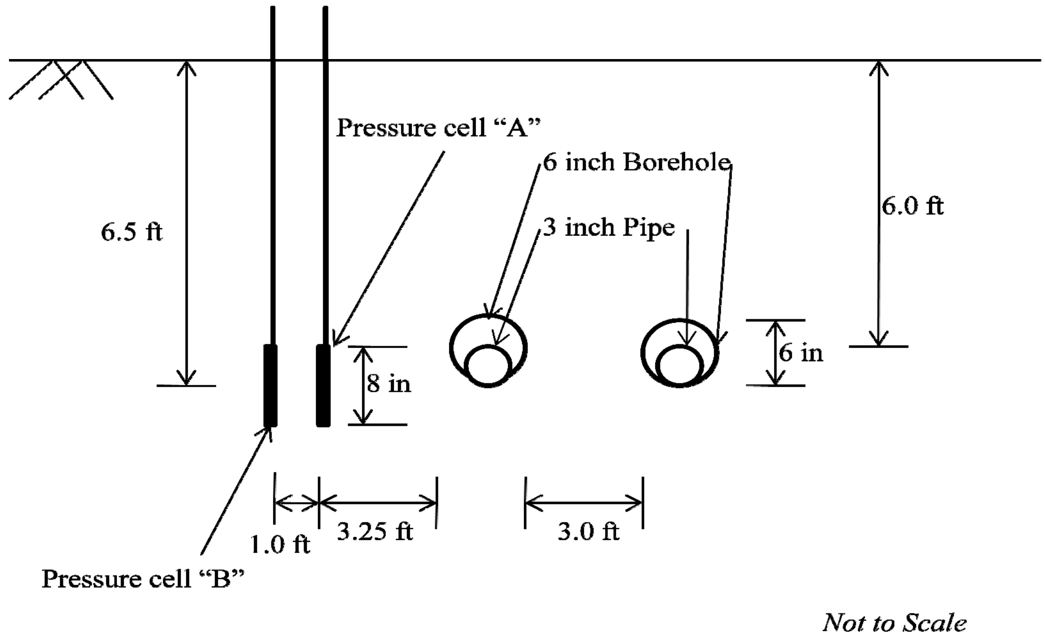


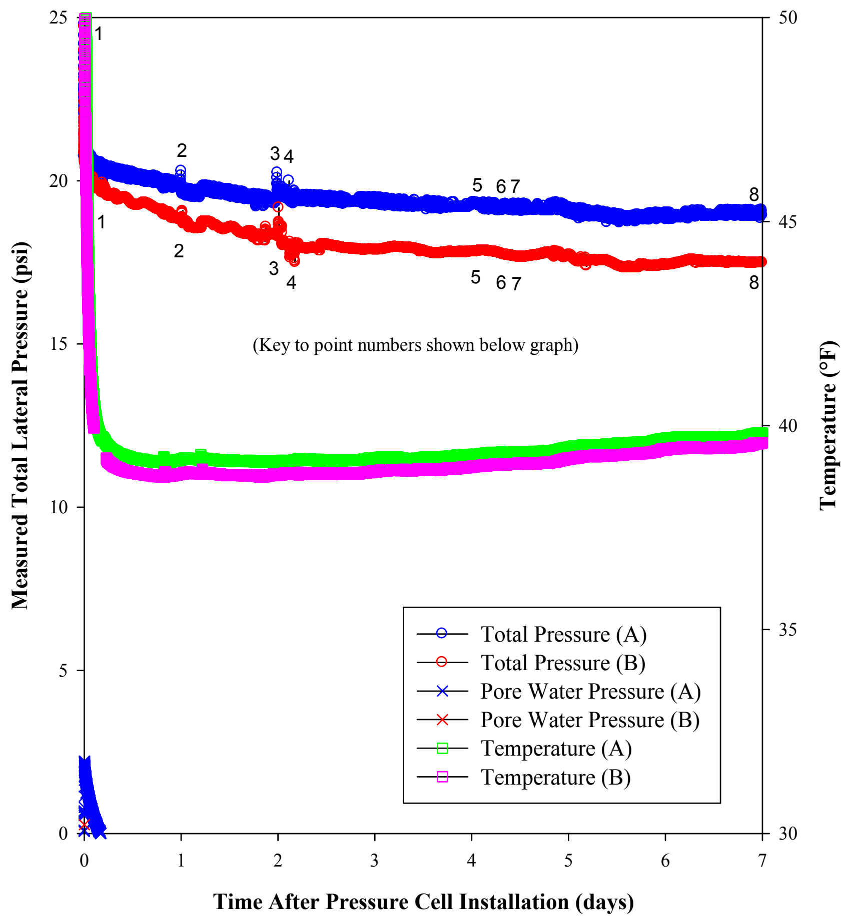
Figure 4.90. Profile showing pressure cells and borehole (looking southwest)

The two vibrating wire push-in pressure cells recorded the total lateral earth pressure and piezometric pressures and the temperature during the 200-foot boring process from points A to C. The pressure cells were connected to a datalogger that was stored in a pickup 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 one day before the HDD work began, which was considered enough time for lateral earth pressure readings to mostly stabilize. The pressure cells were removed two days after the installation was finished (see Figure 4.91).

Figure 4.91. Push-in pressure cells being removed

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 installation (see Figure 4.92). It was observed that both pressure cells A and B recorded the largest total pressure changes as the reamer passed during the installation of the first pipe. Pressure cell A, reading at a depth of 6.5 feet and a distance of 3.5 feet to the center of bore, 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 of 4.5 feet to the center of bore, 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, probably due to the presence of the first pipe between the second borehole and the pressure cells, which likely interfered with possible lateral earth pressure increases from the second bore hole. Additionally, the distance (7 and 8 feet) between the second borehole and the pressure cells might also explain the lack of pressure increase.



Key to Numbered Construction Events	
1)	Pressure cells A and B were installed to read at a depth of 6.5 feet.
2)	Four-inch pilot bore for the first pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B.
3)	Six-inch reamer for the first pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B.
4)	Six-inch reamer with the first 3-inch product pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 3.5 feet to A and 4.5 feet to B. The pipe was installed at a depth of 6 feet to the top of pipe.
5)	Four-inch pilot bore for the second pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 7 feet to A and 8 feet to B.
6)	Six-inch reamer for the second pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 7 feet to A and 8 feet to B.
7)	Six-inch reamer with the second 3-inch product pipe passed pressure cells with its center at a depth of 6 feet and a lateral distance of 7 feet to A and 8 feet to B. The pipe was installed at a depth of 6 feet to the top of pipe.
8)	Pressure cells A and B were removed.

Figure 4.92. Total lateral earth pressure, pore water pressure, and temperature measured 3.5, 4.5, 7, and 8 feet from centerlines of bores

The soil samples that had been recovered during pressure cell installation were analyzed to obtain a better understanding of the soil's properties and how these properties related to the HDD process.

4.3.3.6 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 soil samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table 4.9. The 3-inch pipes were installed at a depth of 6 feet. The recovered samples confirmed that low plasticity clay was found between depths of 5 and 7 feet. The gradation curve is shown in Figure 4.93. The water table was located deeper than 7 feet.

Table 4.9. Soil parameters for the Hub project in Ames, IA

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

*Top of the 3 inch pipes at 6 foot depth,
Location of water table is below this level
-Tests not conducted

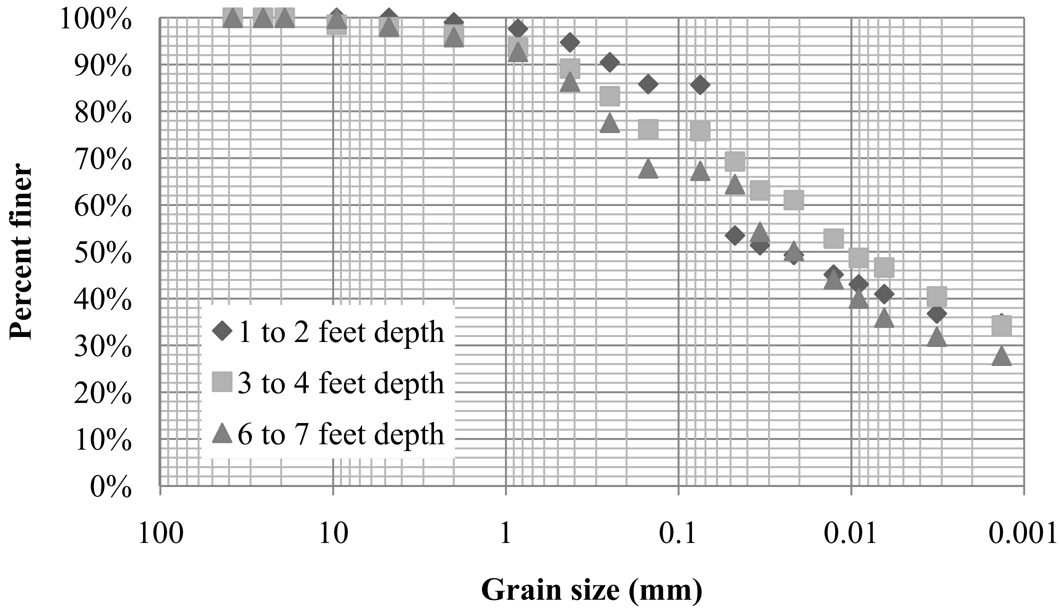


Figure 4.93. Soil gradation curves for the Hub

4.3.3.7 Key Findings

This project involved installing two parallel 3-inch-diameter HDPE water pipes using HDD. Each 200-foot pipe was installed at a depth of 6 feet to the top of the pipe.

The pipe was installed by first drilling a 4-inch pilot bore. Pre-reaming followed 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 pre-reaming stage. These results indicate that the installation affected the surrounding soil.

Soil samples were taken from the depth of the bore path, were tested in the lab, and were classified as sandy lean clay, which is common to the area. This soil is considered an appropriate soil type for HDD.

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

4.3.4 Pammel Drive, Ames, HDD

4.3.4.1 Project Information

This project was executed along Pammel Drive in Ames, Iowa, during June 2008. HDD was used to install an 8-inch-diameter HDPE water main pipe. The purpose of the project was to expand ISU's water distribution system to accommodate a new building scheduled to be constructed. HDD was used to install 480 feet of the water main at a depth of 6 feet to the top of the pipe.

The pipe was installed by drilling and enlarging two straight bores: a north-south bore between points C and D and an east-west bore between points A and C (see Figure 4.94). The research team installed two vibrating wire push-in pressure cells at point B (shown in Figure 4.94) to detect changes in lateral earth pressure during installation.

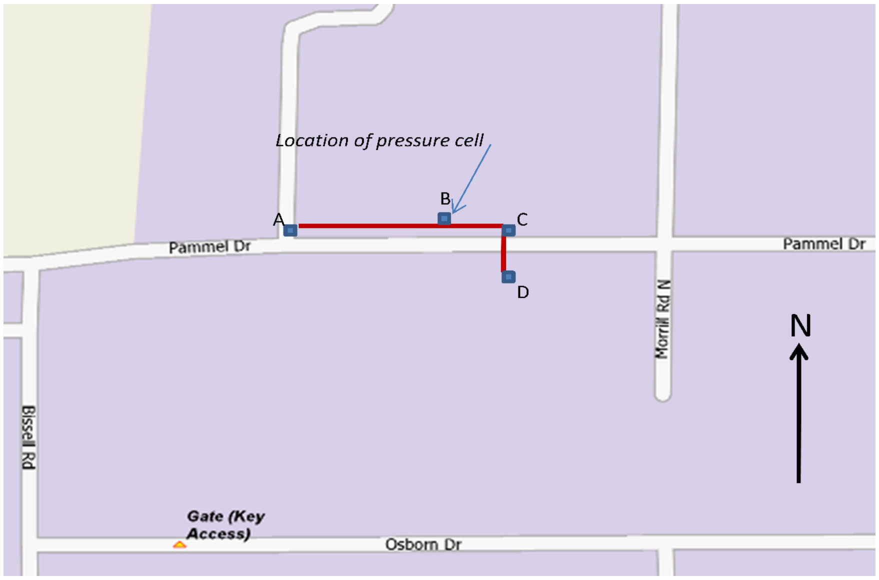


Figure 4.94. Location of HDD project (bore path in red) on Pammel Drive in Ames, IA

4.3.4.2 Trenchless Method Selection

The owner selected trenchless construction to allow Pammel Drive and the sidewalks along it to remain open. Open-cut trenching would have forced the road and sidewalks to be closed and would have forced motorists and pedestrians to detour around the area. Landscaping was also saved. Trenchless construction was required in the plan set; Figure 4.95 shows the two planned bore paths as part of a larger building construction project.

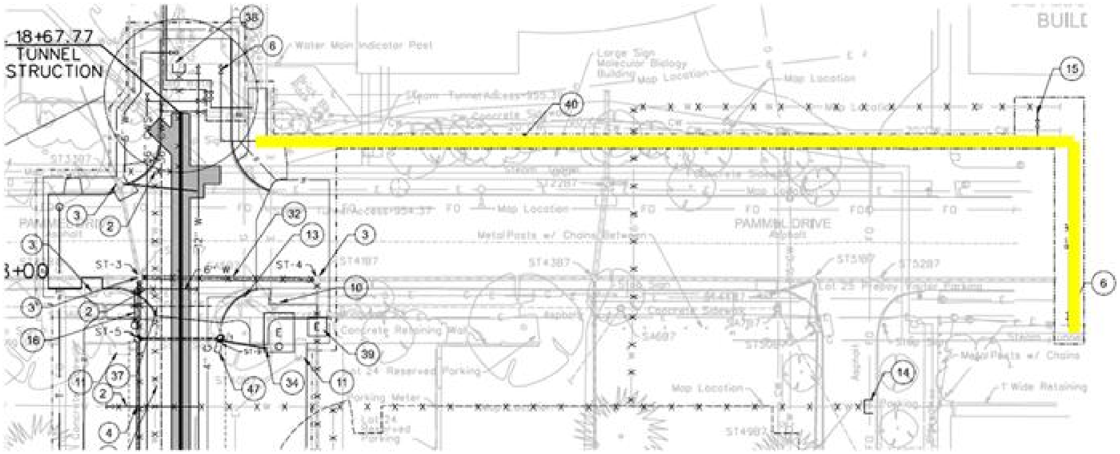


Figure 4.95. Plan view of project site taken from plan set; bore path in yellow

4.3.4.3 Soil Conditions

The project specifications gave the contractor the option to conduct soil testing. The contractor considered soil testing unnecessary because of 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 because the boring was performed above the water table. These soil conditions were considered appropriate for HDD.

4.3.4.4 Trenchless Installation

The contractor decided to first attempt the 80-foot bore between points C and D. Preparations were made by using a backhoe to dig an exit pit at point D. This pit was used to visually confirm the locations of three existing pipes running parallel to the south side of Pammel Drive. Additionally, the pit at point D allowed recovering boring equipment at the proper depth. The HDD rig was then set up at point C facing south toward point D. No other 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 sodium 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 4.96) and using the machine to push the drill bit into the ground. The drill bit was advanced by pushing and spinning the 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 4.97). The pilot bore proceeded with periodic adjustments to the depth and direction to keep the bore online. 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.

Figure 4.96. Directional drilling machine drilling from point C to D

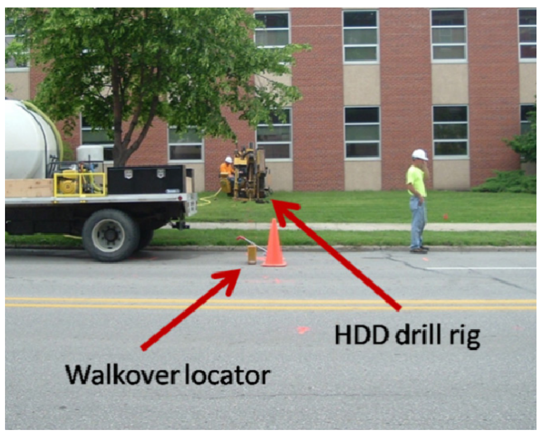


Figure 4.97. View north from point D to C of pilot bore advancing under Pammel Drive

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 pre-reaming 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 pre-reaming process was successfully executed, but some difficulties were encountered due to the large amount of unexpected debris in the soil, such as cobble-sized rocks, discarded rebar, and other items remaining from past construction projects. This debris 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 pipe sections were butt-fused together to reach the total length of 80 feet. The pipe and reamer were then pulled into the borehole at point D. The reamer in front of the pipe cleaned out the borehole and ensured that the diameter was a full 12 inches. The pipe was successfully installed without any problems. A pit was dug at point C to allow better access to the end of the pipe. The pipe was then capped until it could be connected to the rest of the pipeline (see Figure 4.98).

Figure 4.98. HDPE pipe (8-inch diameter) capped at point C after installation from points C to D; locate wire visible

The contractor followed a similar procedure for the 400-foot bore path 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 4.99). For this bore, a 5-inch pilot drill bit was used instead of the 4-inch drill bit used for the bore from points C to D. The larger drill bit was used because the contractor intended to create a larger borehole for this longer installation and to decrease pullback pressures when the product pipe was pulled through.

Figure 4.99. HDD rigs set up for 80-foot bore from points C to D (left machine) and 400-foot-bore from points C to A (right machine)

Due to a potential conflict with additional existing utilities that was not discovered until several days before construction began, the owner and the contractor were forced to shift the planned bore path south by 3 feet because an existing pipeline was 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 only referring to old plans.

Pilot boring began, although progress at first was somewhat slower than usual. One reason for the slow progress was that the contractor encountered ground debris similar to the debris that was encountered when drilling from points C to D. Also, stiff clay was encountered at a depth of 8 feet, which was shallower than the 20-foot depth the contractor had expected based on information obtained from the owner. An additional problem arose when, after 150 feet of boring, the sonde transmitter attached to the pilot drill bit became dislodged, which meant that it would be impossible to accurately locate the pilot bore in the ground for the remaining 250 feet. Therefore, the contractor was forced to stop the bore and pull the drill bit back to the machine at point C. The sonde was replaced, and the pilot bore began again. Soil conditions caused the rest of the bore to proceed slowly, but the run was eventually finished successfully. The drill bit reached the surface at point A, as shown in Figure 4.100.

Figure 4.100. Drill bit (5-inch diameter) emerging at point A

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 pre-reaming stage. The reamer was pulled back through the pilot borehole, enlarging the hole from 5 inches to 14 inches in diameter (see Figure 4.101). This pre-reaming process was also successfully executed, although there was 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.

Figure 4.101. Reamer (14-inch diameter) pulled into borehole at point A (clockwise from upper left)

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 4.102). The pipe was successfully installed following 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 completed.

Figure 4.102. HDPE pipe (8-inch diameter) pulled into borehole at point A

4.3.4.5 Lateral Earth Pressure Monitoring

Using a conventional augered drill rig, the research team drilled two vertical test holes near the bore path in order to characterize soil at the site. Disturbed soil samples were taken at various depths, and a 3-inch-diameter thin-walled Shelby tube was pushed through the bottom of the first test hole 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 bagged samples were tested in the laboratory.

Two vibrating wire push-in pressure cells were installed at point B. To install them, an instrumentation hole was first drilled at a lateral distance of 3 feet from the planned bore path. This instrumentation hole was drilled to a depth of 5 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 drill rig vertically pushed the pressure cell into the ground to a depth of 5 to 7 feet (see Figure 4.103). The cells were oriented 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 centered at a depth of 6.5 feet, matching the intended HDD installation depth. A similar process was used to install the second pressure cell, denoted “B.” Pressure cell B was installed in a new instrumentation hole to take readings 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 buffer was to ensure that the soil deformations induced by the installation process for one pressure cell would not affect the other pressure cell’s readings.

Figure 4.103. Installing push-in pressure cell

The two vibrating wire push-in pressure cells recorded the total lateral earth and piezometric pressures and the temperature during the 400-foot-long boring process from points B to D. The pressure cells were connected to a datalogger that was stored in a pickup truck. The datalogger recorded readings once every 10 seconds. This frequency of readings was considered sufficient to completely document the effects of the directional drilling process on the lateral earth pressure of the surrounding soil.

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

After the pilot bore had passed the pressure cells, it was possible to measure the actual lateral distance between the pressure cells and the actual center of the bore path. The walkover locator marked the position 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 4.104). These measured distances matched the target distances. The depth of the pilot bore also followed expectations; it was measured to be about 6.5 feet deep when it passed the pressure cells (see Figure 4.105).

Figure 4.104. Bore seen approaching the two pressure cells (left)

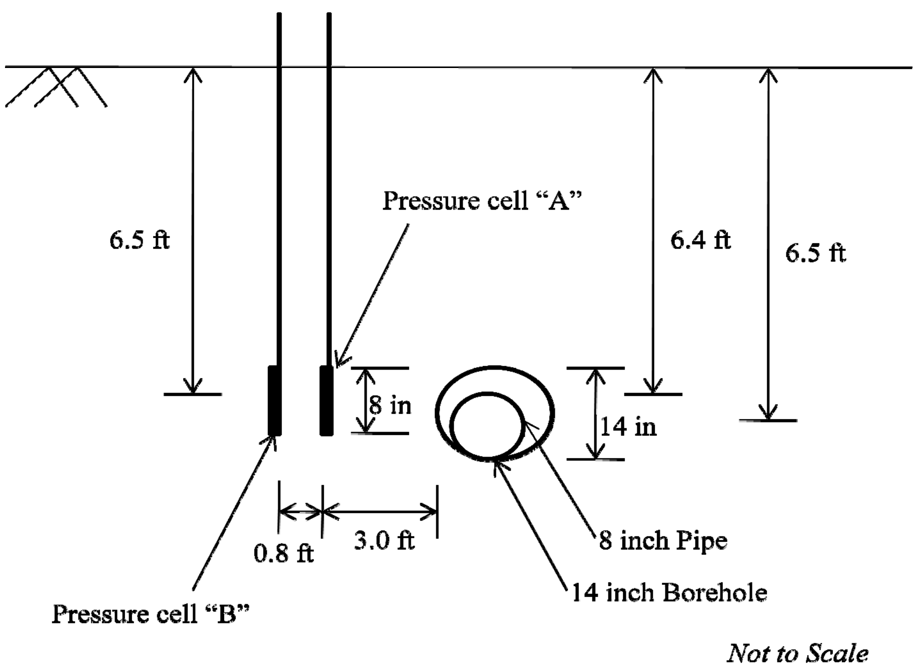
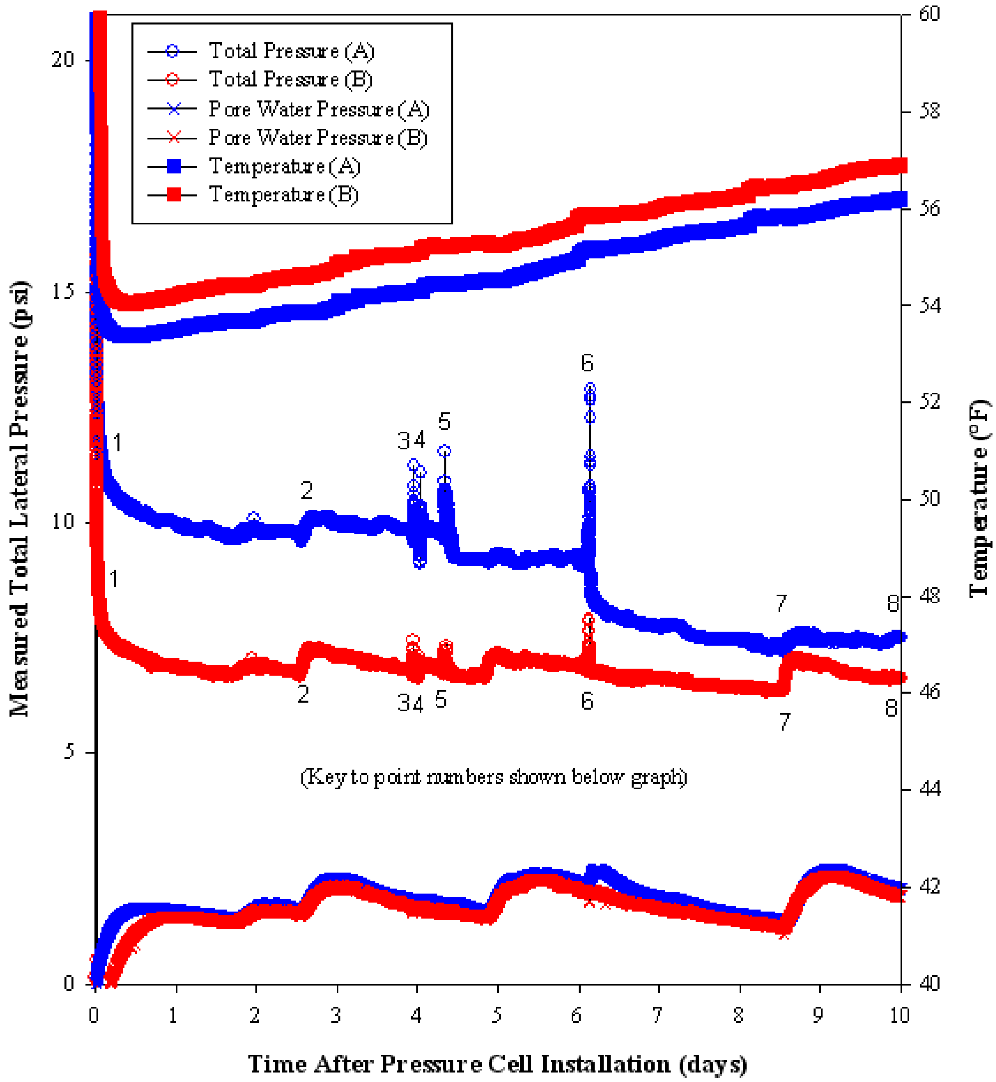


Figure 4.105. Profile showing pressure cells and borehole (looking west)

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 4.106). 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 of 3.0 feet to the center of the bore, 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 of 3.8 feet to the center of the bore, recorded a pressure increase of 0.7 psi. In the pre-reaming phase, the 14-inch reamer 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, which included strong rains and high winds, caused pore water pressure increases through the permeable sandy soil, which led to accompanying total pressure increases.



Key to Numbered Construction Events	
1)	Pressure cells A and B were installed to read at a depth of 6.5 feet.
2)	Weather-related overnight pressure increase.
3)	Five-inch pilot bore passed pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3 feet to A and 3.8 feet to B. This run was aborted due to the sonde becoming dislodged.
4)	Five-inch pilot bore passed pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3 feet to A and 3.8 feet to B. This run was completed.
5)	Fourteen-inch reamer passed pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3 feet to A and 3.8 feet to B.
6)	Fourteen-inch reamer with the 8-inch product pipe passed pressure cells with its center at a depth of 6.5 feet and a lateral distance of 3 feet to A and 3.8 feet to B. The pipe was installed at a depth of 6 feet to the top of pipe.
7)	Weather-related overnight pressure increase.
8)	Pressure cells A and B were removed.

Figure 4.106. Total lateral earth pressure, pore water pressure, and temperature measured 2.1 and 3.4 feet from centerline of bore from points C to A

4.3.4.6 Soil Characterization

The owner supplied a geotechnical report for the project, which made additional soil testing by the research team unnecessary (see Appendix E). Values for that report were used to estimate soil properties at the project site (see Table 4.10). The water table was located deeper than 10 feet.

Table 4.10. Soil parameters for Pammel Drive project

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Compressive Strength (psf)	Dry Density (pcf)
0–2.0	Peat (Pt)	17.4	-	-	-	-	-
2.0–6.0	Sandy lean clay (CL)	14.6	-	-	-	3500	122
6.0–10*	Sandy lean clay (CL)	11.5	-	-	-	8000	125

*Top of the 8 inch pipe at 6 foot depth,
Location of water table is below this level
-Test not conducted

4.3.4.7 Key Findings

The two separate HDD bores used to install HDPE water pipe provided examples of effective HDD. During the first bore, the contractor installed 80 feet of 8-inch-diameter pipe at a depth of 6 feet to the top of the pipe . The pipe was installed by first drilling a 4-inch pilot bore, followed by pre-reaming using a 12-inch reamer before finally pulling the 12-inch reamer attached to the product pipe through the hole. The bore was successfully completed, although with some delays because of unexpected debris being encountered underground. The second bore involved installing 400 feet of 8-inch-diameter pipe at a depth of 6 feet. However, the contractor approached this bore differently from the first because of the increased length. The contractor used a more powerful HDD rig than before and substituted a 5-inch-diameter pilot bore and a 14-inch reamer. 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 noticeable changes in the pressure readings as the boring proceeded past the pressure cells, especially during the pilot bore. These results indicate that the boring had an effect on the surrounding soil.

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

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

4.3.5 Safe City Demonstration, Ankeny, Impact Moling

4.3.5.1 Project Information

This project occurred at the Iowa Association of Municipal Utilities' (IAMU) Safety and Training Field in Ankeny, Iowa, during late August 2008. Impact moling, a horizontal boring method, was used to install a 48-foot-long, 0.75-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 shown in Figure 4.107. The research team installed two vibrating wire push-in pressure cells at point B to observe changes in lateral earth pressure during installation.

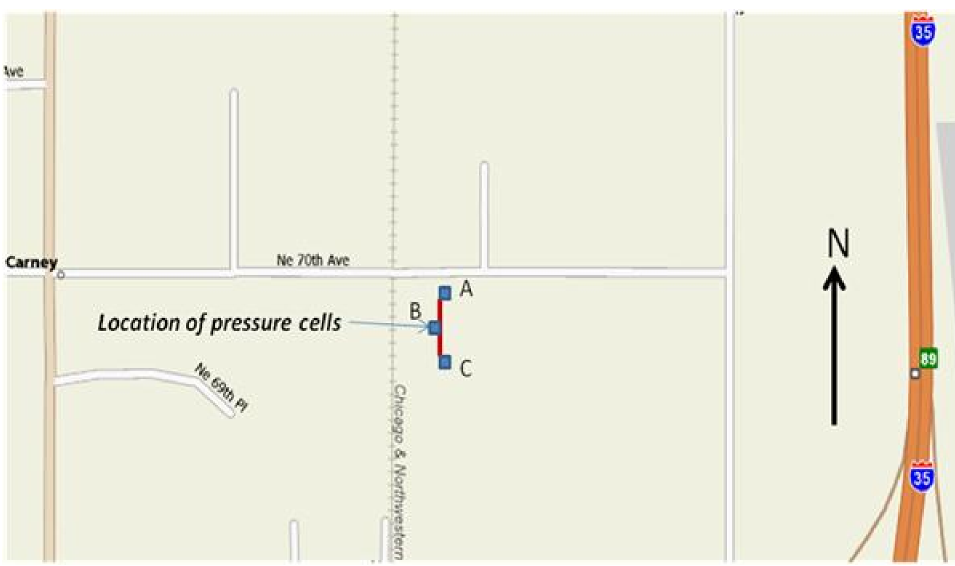


Figure 4.107. Location of IAMU's Safety and Training Field on NE 70th Avenue in Ankeny, IA; bore path shown in red

4.3.5.2 Trenchless Method Selection

IAMU selected impact moling 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, 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 because the air compressor can also be used for pressure testing. No plan set was created for this demonstration workshop.

4.3.5.3 Soil Conditions

The bore was drilled through native soils that were mainly composed of black fat clay. Impact moling is considered appropriate in these soil conditions, and the installation was considered low risk, so soil testing was not considered necessary. Additionally, digging entry and exit pits on either side of the bore path in preparation for the bore confirmed the soil type at the depth of boring. However, formal soil classification would have been encouraged for a normal project. Some dewatering was necessary because recent heavy rain and the flat topography of the area meant that the water table was within 3 feet of the ground surface.

4.3.5.4 Trenchless Installation

The contractor made preparations by using a backhoe to dig entry and exit pits at points A and C, respectively. Both pits were excavated to dimensions of around 2 feet by 5 feet. The pits were dug to a depth of approximately 4 feet because it was 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 4.108), which was used to dewater the entry pit and allow the mole to be launched.

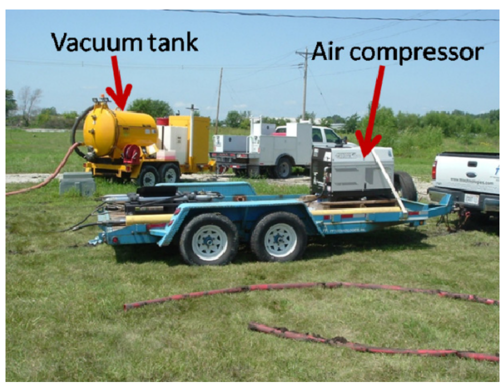


Figure 4.108. Contractor equipment including air compressor and vacuum tank used for dewatering

This demonstration used a 2.5-inch-diameter Grundomat impact mole, the head of which is shown in Figure 4.109. 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 remain on course if it encounters an underground obstruction, such as a rock. No drilling fluid is necessary for impact moling, and the equipment is not designed to deliver drilling fluid.

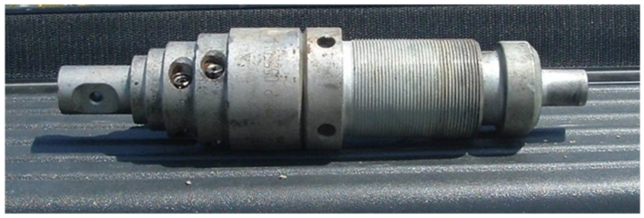


Figure 4.109. Grundomat reciprocating stepped-cone chisel head

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 4.110). 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.

Figure 4.110. Launching the mole and adjusting line and grade using telescopic aiming frame

The mole continued to advance without pausing until it successfully reached the exit pit at point C (see Figure 4.111). The pit had become flooded with 8 inches of water when the mole arrived at the pit. Dewatering was necessary so that the mole did not "swim"; swimming could occur if the soil was sufficiently saturated and soft so that the mole could no longer advance. The vacuum tank was emptied of its contents from dewatering the entry pit, and the exit pit was dewatered, allowing the mole to be retrieved (see Figure 4.112).

Figure 4.111. Locating mole after it emerged at point C into exit pit, which was flooded

Figure 4.112. Exit pit at point C dewatered to allow retrieval of the mole

Next, the 0.75-inch copper pipe was installed. This installation was done by disconnecting the pneumatic hose from the air compressor at point A and reconnecting it to the end of the copper pipe. The pneumatic hose was used like a rope and pulled out of the borehole at point C by hand while the copper pipe was pulled in at point A (see Figure 4.113). This process completed the installation.

Figure 4.113. Entry pit at point A as 0.75-inch copper pipe was pulled into 2.5-inch-diameter hole

The installation was finished, and no problems were encountered. Dewatering the entry and exit pits was the only challenge.

4.3.5.5 Lateral Earth Pressure Monitoring

Two vibrating wire push-in pressure cells were installed at point B two days before the boring started. To install these cells, an instrumentation hole 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 pushed into the ground by the drill rig to record pressure readings at a depth of 3.3 feet, closely matching the expected bore depth. The cell was oriented with the flat pressure-sensing side of the pressure cell parallel to the planned horizontal borehole. A similar process was used to install the second pressure cell, denoted "B." Pressure cell B was installed in a new borehole to also take readings 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 feet 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-foot distance ensured 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 and piezometric pressures and the temperature during the 48-foot boring process from points A to C. The pressure cells were connected to a datalogger that was stored in a pickup 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 4.114.

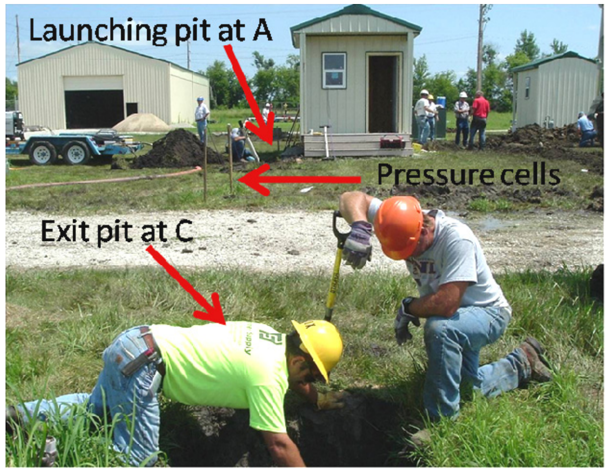


Figure 4.114. Exit pit at point C (foreground), entry pit at point A (background), and pressure cells at point B (middle)

After the mole had passed the pressure cell, it was possible to more accurately measure the lateral distance between the pressure cells and the actual center of the bore path. The walkover locator marked the position of the mole as it passed. The lateral distances were measured to be 1.6 feet from the face of pressure cell A and 1.6 feet from pressure cell B. These lateral distances were considered to be very close to the expected distances. However, the moling depth was slightly deeper than expected, as it was measured to be 3.8 feet instead of the 3.3 feet that was expected (see Figure 4.115). This slight depth discrepancy was expected to result in lower pressure readings.

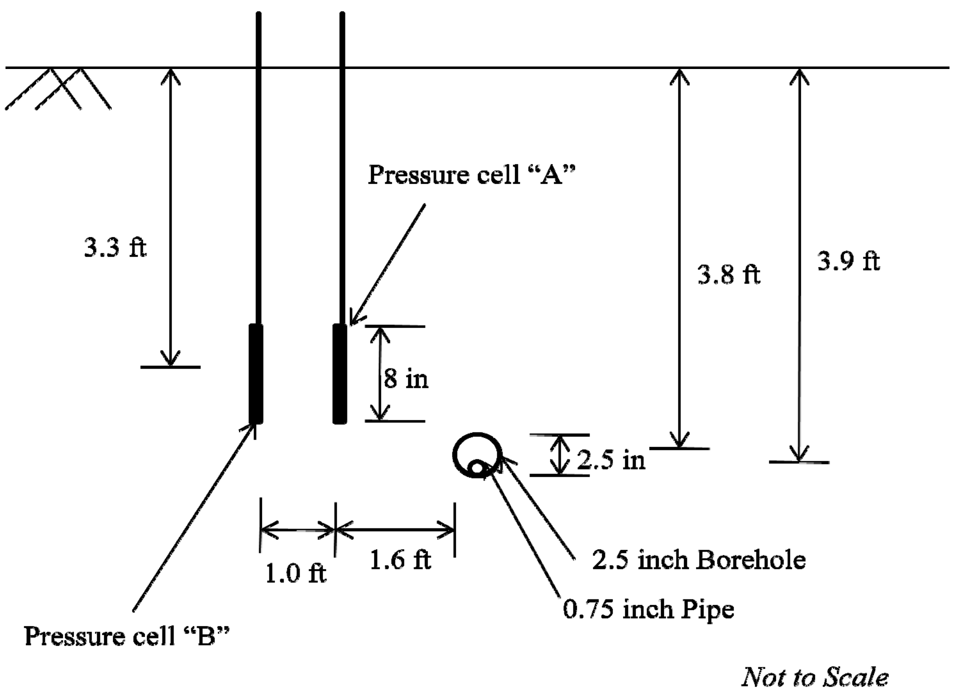
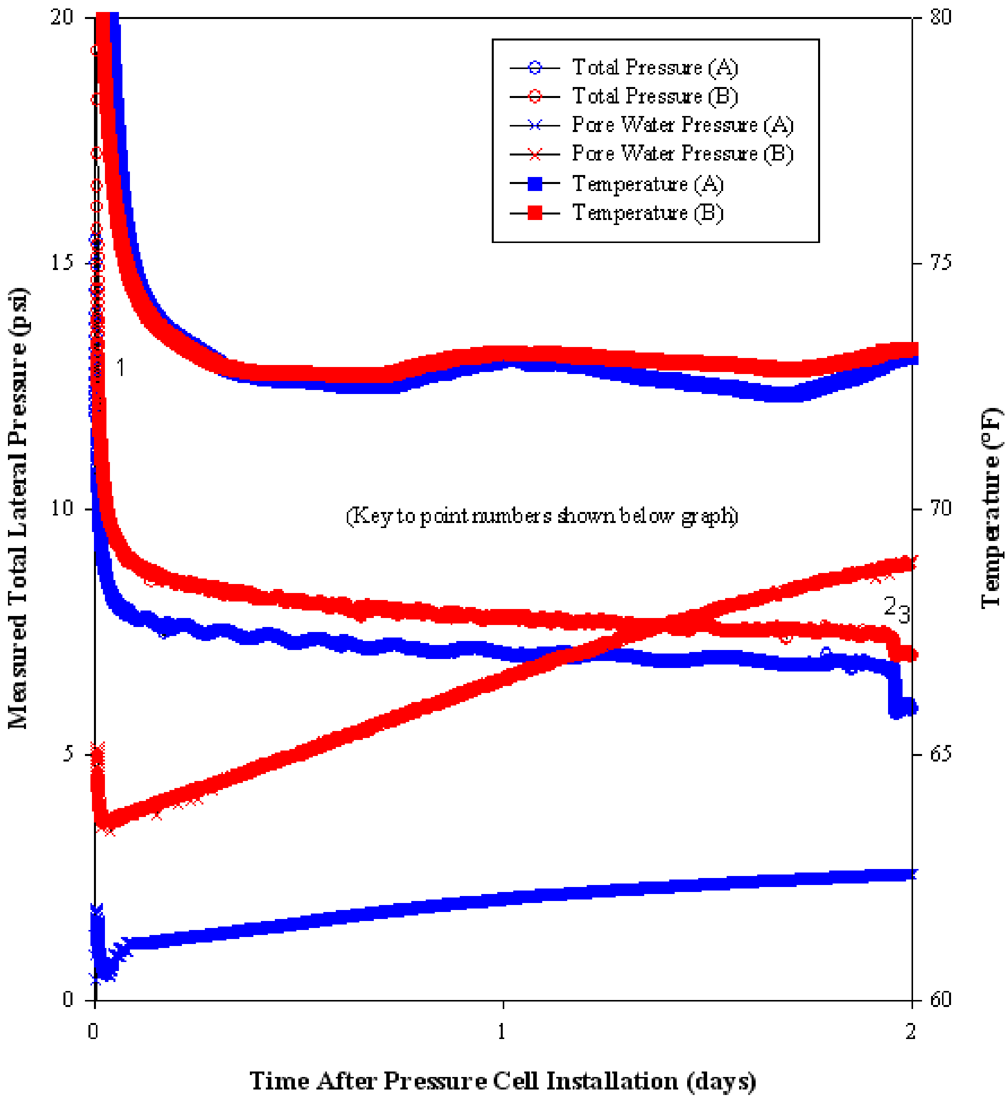


Figure 4.115. Profile showing pressure cells and borehole (looking north)

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 4.116). 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 fact that the mole displaces soil radially as it moved through the ground.



Key to Numbered Construction Events	
1)	Pressure cells A and B were installed to read at a depth of 3.3 feet.
2)	The 2.5-inch pneumatic mole passed 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 are recorded. Pressure cells were removed one day later.

Figure 4.116. Total lateral earth pressure, pore water pressure, and temperature measured 1.7 and 3.1 feet from centerline of bore

4.3.5.6 Soil Characterization

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

4.3.5.7 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 of 3.3 feet to the top of the pipe. The pipe was installed by forming a borehole with an impact mole and pulling the pipe into the hole afterward. The bore was successfully completed, and no significant problems were 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 bore proceeded past the pressure cells; however, the changes indicated a negative pressure change. Despite these negative changes, it is clear that the boring is affecting the surrounding soil pressures.

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

The installation was successfully completed 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.

4.3.6 State Avenue, Ames, HDD

4.3.6.1 Project Information

This project was constructed along State Avenue in Ames, Iowa, during September 2008. HDD 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. One location discussed in this report was where pipe was installed using HDD for a distance of 30 feet at a depth of 8 feet to the top of the pipe.

The pipe was installed by drilling and enlarging a single borehole running north-south parallel to State Avenue (see Figure 4.117). The borehole was drilled between points B and D. The research team installed two vibrating wire push-in pressure cells at point C in Figure 4.117 to observe changes in lateral earth pressure during installation.

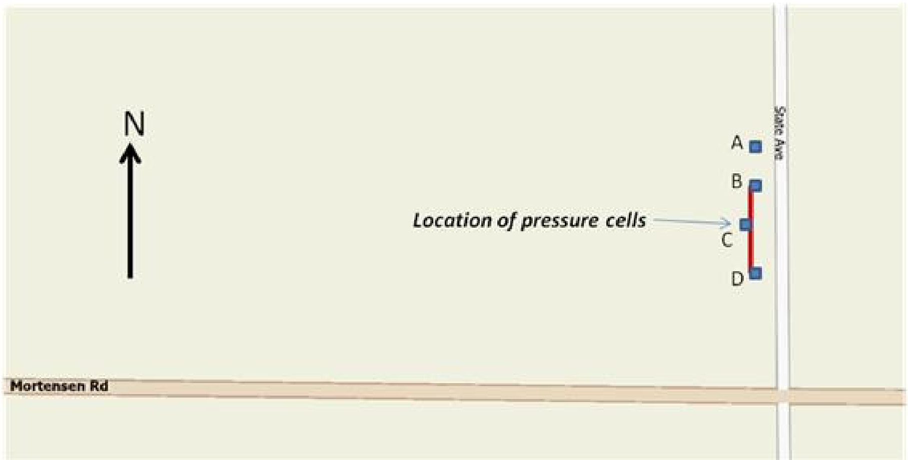


Figure 4.117. HDD project location (bore path in red) along State Avenue in Ames, IA

4.3.6.2 Trenchless Method Selection

The owner selected trenchless construction 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 was only required under the field entrance (see Figure 4.118).

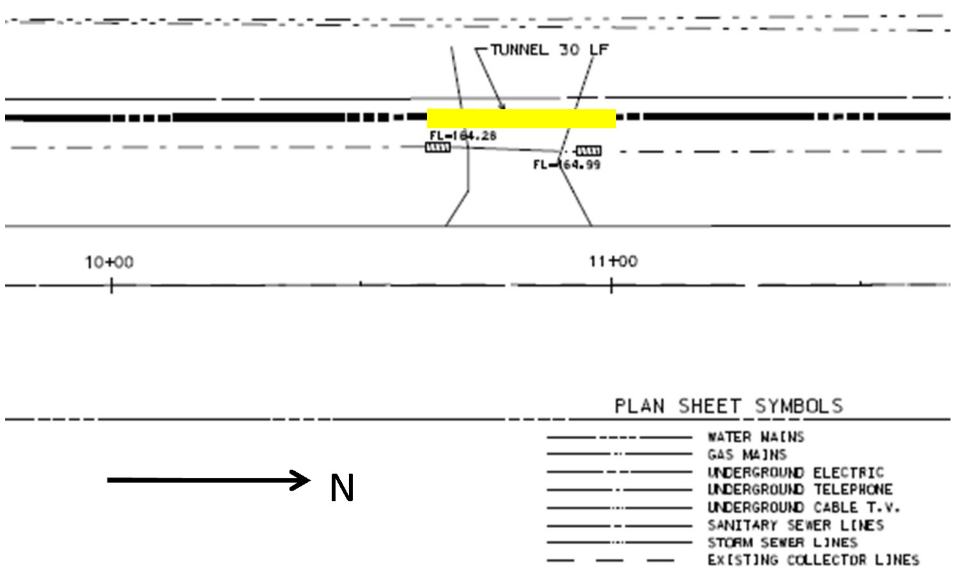


Figure 4.118. Plan view of project site taken from the plan set; bore path in yellow

4.3.6.3 Soil Conditions

As a City of Ames project, the option of soil testing was the contractor's responsibility, as stated in the project specifications. In this case, the contractor elected not to conduct any soil testing.

This decision was made because of familiarity with the clayey soils common to the area. The contractor also stated that it normally obtains information from others who have previously drilled in the area. Additionally, digging the entry and exit pits on either side of the field entrance (for the purpose of accessing the pipe ends after installation) also allowed visual confirmation of the soil types at the boring depth. 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 because the boring was performed above the water table. This soil condition was considered appropriate for HDD construction.

4.3.6.4 Trenchless Installation

The contractor made preparations for installation by 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-long pipe installed with trenchless construction would eventually be connected to the rest of the pipeline, which would be installed later with open-cut methods. The pit dug at point B is shown in Figure 4.119. The pit at point D served 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 potholing was not necessary to confirm existing utility locations.

Figure 4.119. Pit at point B

A drilling fluid consisting of sodium bentonite and a clay-inhibiting polymer was used during all boring stages. 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 4.120) and 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 drill rig's hydraulic machinery. 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 4.121).

Figure 4.120. Directional drilling machine

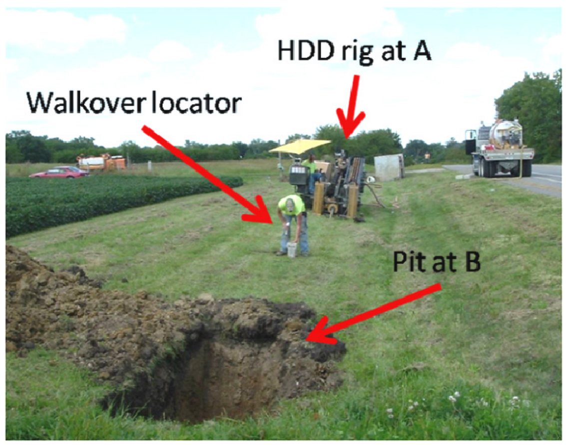


Figure 4.121. Truck with drilling fluid mixing tank (top right) worker with handheld locator (center) monitors advancing pilot bore depth and position

The pilot bore proceeded with periodic adjustments being made to the depth and direction to keep the bore online. 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 accommodate 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 around 30 minutes of boring (see Figure 4.122).

Figure 4.122. Pilot bore emerging from exit pit at point D

The 5.5-inch drill bit used for the pilot bore was removed from the drill string and replaced by a 16-inch reamer for the pre-reaming stage (see Figure 4.123). 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, enlarging the hole from 5.5 inches to 16 inches in diameter. This pre-reaming 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 reemerged at point D.

Figure 4.123. Reamer (16-inch diameter) attached to drill string

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 4.124). The pipe and reamer are shown being pulled into the borehole at point D in Figure 4.125. The reamer in front of the pipe cleaned out the borehole and ensured that the diameter was a full 16 inches. The pullback was subject to additional resistance because both the borehole and the bell had 16-inch diameters, but the pullback was only 30 feet, so it was deemed acceptable. The pipe was successfully installed 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 could later be connected to the open-cut portion of the pipeline installation.

Figure 4.124. Certa-Lok® bell (16-inch diameter) connecting pipe sections (12-inch diameter)

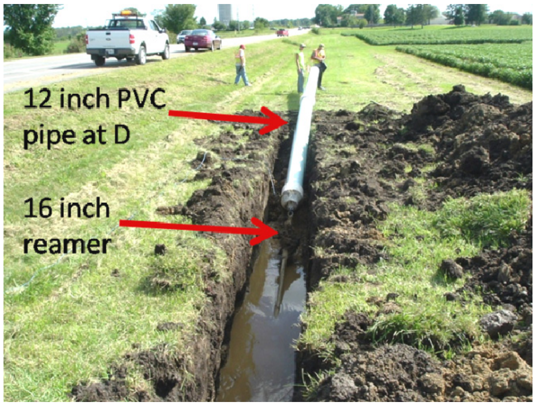


Figure 4.125. PVC pipe (12-inch diameter) being pulled into borehole from point D with a 16-inch diameter reamer leading

The contractor also followed the same procedure for two similar bores in the vicinity. Both of these bores were successfully completed with no problems.

4.3.6.5 Lateral Earth Pressure Monitoring

Two vibrating wire push-in pressure cells were installed at point C. To do this installation, an instrumentation hole was drilled a lateral distance of 2 feet away from the planned bore path. This borehole was drilled to a depth of 6 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 drill rig vertically pushed the pressure cell into the ground to a depth of 6 to 8 feet (see Figure 4.126). The cells were oriented so that the flat pressure-sensing side of the pressure cell was parallel to the planned horizontal borehole. Due to the configuration of the instrument, the sensors in the pressure cell were centered at a depth of 7.5 feet, matching the expected depth of the HDD installation. A similar process was used to install the second pressure cell, denoted “B.” Pressure cell B was installed in a new instrumentation hole to also take readings 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 feet away from pressure cell A so that they were located as close as possible while still leaving a 3-foot buffer (see Figure 4.127). This buffer was ensured that the soil deformations induced by the installation process for one pressure cell would not affect the other pressure cell’s readings.

Figure 4.126. Push-in pressure cells being installed

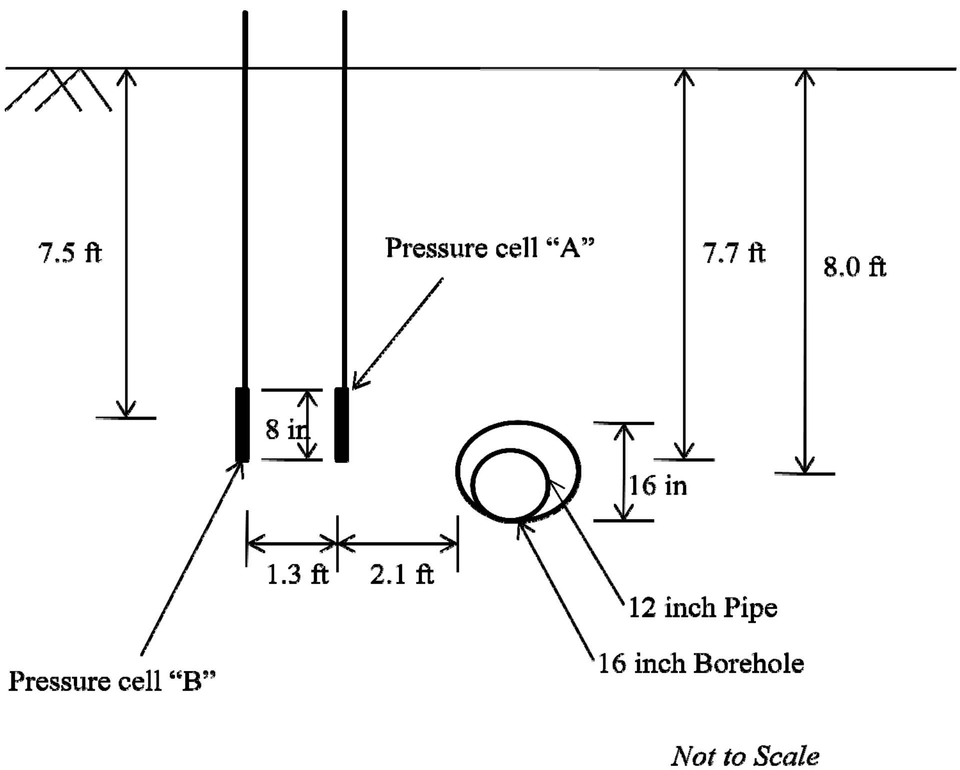


Figure 4.127. Profile showing pressure cells and borehole (looking south)

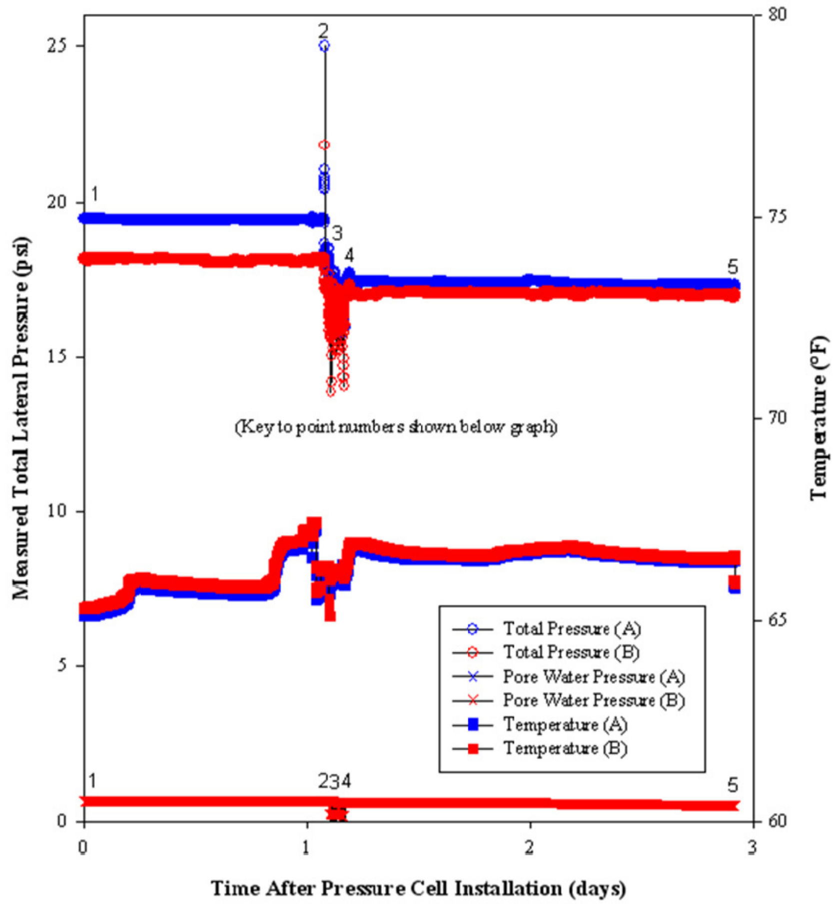
The two vibrating wire push-in pressure cells recorded the total lateral earth and piezometric pressures and the temperature during the 30-foot boring process from points B to D. The pressure cells were connected to a datalogger that was stored in a pickup truck and that recorded readings once every 10 seconds. This frequency of readings was considered sufficient to create a continuous record of the effect that the directional drilling process had on lateral earth pressure of the surrounding soil.

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

The pressure readings 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 4.128. 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 pilot bore as it passed, which was measured to be a lateral distance of 2.1 feet from the face of pressure cell A and 3.4 feet from pressure cell B. These actual distances were considered to be 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 feet instead of the expected 7.5 feet. This difference in expected depth was expected to lower the pressure readings.

Figure 4.128. Bore approaching two pressure cells

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 entire construction process. Figure 4.129 shows 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 of 2.1 feet to the center of bore, 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 of 3.4 feet to the center of bore, recorded a pressure increase of 3.7 psi. The pre-reaming and pipe installation phases created much smaller pressure increases. The 16-inch reamer in the pre-reaming 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 12-inch PVC pipe with the 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 in which the bell passed the pressure cells because a period of small pressure fluctuations was recorded in the time after the reamer and the beginning of the pipe had passed the pressure cells. It is estimated that about five minutes may have elapsed, although no single pressure spike was noticed. Small additional fluctuations after the installation was completed may be due to the operation of a backhoe that was very near the pressure cells.



Key to Numbered Construction Events	
1)	Pressure cells A and B were installed to read at a depth of 7.5 feet.
2)	The 5.5-inch pilot bore passed pressure cells with its center at a depth of 8 feet and a lateral distance of 2.1 feet to A and 3.4 feet to B.
3)	The 16-inch reamer passed pressure cells with its center at a depth of 8 feet and a lateral distance of 2.1 feet to A and 3.4 feet to B.
4)	The 16-inch reamer with the 12-inch product pipe passed the pressure cells with its center at a depth of 8 feet and at a lateral distance of 2.1 feet to A and 3.4 feet to B. The 16-inch-diameter bell passed about five minutes later, causing a slight fluctuation in the pressure readings. The pipe was successfully installed.
5)	Pressure cells A and B were removed.

Figure 4.129. Total lateral earth pressure, pore water pressure, and temperature measured 2.1 and 3.4 feet from centerline of bore

The soil samples that were recovered during pressure cell installation were analyzed to obtain a better understanding of the soil's properties and how they relate to the HDD process.

4.3.6.6 Soil Characterization

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

4.3.6.5 Key Findings

This project demonstrated an effective use of HDD to install PVC water pipe. The contractor installed 30 feet of 12-inch-diameter pipe at a depth of 8 feet to the top of the pipe. The pipe was installed by first drilling a pilot bore, followed by pre-reaming using a 16-inch reamer and pulling the 16-inch reamer attached to the product pipe through the hole. The bore was successfully completed with no important 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 each construction process proceeded past the pressure cells, especially during the pilot bore. These results indicate that the boring affected the surrounding soil.

The soil was visually observed to be sandy lean clay, which is common to the area. This soil is considered an appropriate soil type for HDD.

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

4.4 Discussion of Site Investigation Results

The research team visited trenchless job sites in Iowa with the goals of obtaining a better understanding of field activities, better identifying the risks involved in trenchless technologies, and determining how these risks can be minimized. A total of 19 projects were visited. The trenchless methods used on these projects included one pipe jacking, one tunneling, one impact moling, five auger boring, and eleven HDD.

The "Site Visits" portion of the fieldwork involved observing 13 trenchless construction projects. These projects involved the research team making visits to the jobsite and observing and documenting construction practices and the successes and failures experienced at these sites. 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 to 495 feet.

All of the "Site Visit" projects were successfully completed, although for one project, the HDD trenchless construction process caused a frac-out and some surface heave to occur. 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 when the hole was enlarged to 12 inches in diameter. Stiff soil caused 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 flowing out of the launching and retrieval pits. Because the drilling fluid was trapped, pressure increased, causing the borehole walls to rupture and frac-out so that drilling fluid flowed to the ground surface through the rupture. This frac-out occurred while boring through gravelly clay soil. The pre-reaming process was continued and, several hours later, surface heave of around 10 inches appeared directly over a portion of the bore path. This heave occurred approximately 10 feet after switching from gravelly soil to clay.

Difficulties matching the drilling fluid to the soil conditions may have contributed to this failure. A drilling fluid with bentonite was used because the contractor expected granular soils, which is an appropriate use. When clay was encountered, the drilling fluid may have formed a thin layer on the clay surface that later was carried with the reamer. 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. These combined actions may have caused the fluid pressures to rise until the soil eventually fractured along the planes of greatest weakness, allowing the fluid to flow to the surface through the fractures. A clay-inhibiting polymer in the drilling fluid may have prevented this from happening. This hypothesis is supported by the research team's later subsurface soil investigation in which permeable sandy soil was observed around the frac-out area and less permeable clay was observed around the heave area. If contractor personnel had better knowledge of subsurface soil conditions, they might have used a different drilling fluid mix and avoided this problem. This example suggests that additional field testing is advisable for HDD projects when there is significant uncertainty of the subsurface conditions.

The second project during which a frac-out was encountered was investigated as one of the "Field Monitoring" (Section 4.3) projects (sites where research included additional testing and measurement of lateral earth pressure). At the site designated as Osborn Drive, Ames, Domestic Water, HDD (Section 4.3.1), the research team recovered undisturbed soil samples for laboratory testing, measured soil stresses during the construction, and carried out an in-depth investigation of the soil properties. As part of this project, the contractor installed an 8-inch pipe at a depth of 6 feet for a distance of 485 feet. A frac-out first occurred while pre-reaming a 4-inch pilot hole to 14 inches in diameter. The contractor vacuumed the pooled drilling fluid and continued the bore. A second frac-out occurred while the pipe was being installed behind the 14-inch reamer. A push-in pressure cell was used to monitor soil pressure increases caused by the bore, but no increases were measured immediately before either frac-out. These results suggest that drilling fluid pressures never became extremely high and that 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 may 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, (3) a lack of stability of the borehole walls, and/or (4) the speed with which the reamer was pulled through the borehole.

The research team noted that, for the majority of projects that were observed, neither the designer (on behalf of the owner) nor the contractors utilized a soil testing program. The contractors generally felt that their previous experience in an area made soil testing an unnecessary expense.

An investigation that included lateral earth pressure monitoring using push-in pressure cells was performed at eight trenchless installations. Seven of these installations used HDD techniques in sandy clay or well-graded sand with silt and gravel. The pipe diameter installed using HDD techniques ranged from 3 to 12 inches. Push-in pressure cells were installed at a distance ranging from 1.3 to 11.5 feet. For the HDD installations of 6-inch-diameter pipe in well-graded sand, the increase in lateral earth pressure was measured during the pilot bore, pre-reaming, and combined pipe-pulling and reaming stages. During the pilot bore stage, the change in lateral earth pressure was 2.4 psi at a distance of 3.8 feet and 0 psi at a distance of 11.5 feet. During the pre-reaming stage, the change in lateral earth pressure ranged from 2.5 psi to 0 psi at distances of 3.8 and 11.5 feet, respectively, while the change in lateral earth pressure during the combined pipe-pulling and reaming stage ranged from 2.1 psi to 0 psi for the same distances.

For sites with sandy lean clay soil and pipe diameters of 3 inches, the change in lateral earth pressure measured at horizontal distances away from the pipe of 3.5 feet, 4.5 feet, 7 feet, and 8 feet ranged from 0.5 psi to 0 psi during the pilot bore. During the pre-reaming stage, the pressure ranged from 0.9 psi to 0 psi at the same distance.

In both cases cited above, the maximum change in lateral earth pressure was observed during the pre-reaming stage.

The measurements documenting increases in soil stress that were included in the “Field Monitoring” portion (Section 4.3) of this report are in Table 4.11. Stress measurements were taken in sandy lean clay, well-graded sand with silt and gravel, and fat clay. The pressure cells recorded stress readings at distances ranging between 1.3 and 11.5 feet from the edge of the borehole. Lateral stress changes as high as 5.6 psi were recorded. During the Osborn Drive Domestic Water HDD project, negative pressure increases were recorded during the passage of the HDD pilot bore, pre-reamer, and pipe pull-in. These negative pressures may have been caused by the formation of cracks that later resulted in frac-outs or from installing the pressure cells perpendicular to rather than parallel to the direction of drilling.

Table 4.11. Field monitoring results

Project	Trenchless Method	Soil	Pipe Dia. (in)	Pressure Cell	Distance from Bore (ft)	Pressure Differential (psi)		
						Pilot Bore	Pre-ream	Pipe + reamer
Ames, Osborn Drive Domestic Water	HDD	Sandy Lean Clay	8	A	1.3	-3	-6.3	-4.3
Ames, Seed Science Building #1	HDD	Well Graded Sand with Silt and Gravel	6	B	9.5	0	0.1	0.7
				A	11.5	0	0	0
Ames, Seed Science Building #2	HDD	Well Graded Sand with Silt and Gravel	6	B	3.8	2.4	2.5	2.1
				A	5.4	0.1	0.2	0.1
Ames, Hub #1	HDD	Sandy Lean Clay	3	B	4.5	0.3	0.9	-0.5
				A	3.5	0.5	0.9	0.5
Ames, Hub #2	HDD	Sandy Lean Clay	3	B	8	0	0	0
				A	7	0	0	0
Ames, Pammel Drive	HDD	Sandy Lean Clay	8	A	3	1.5	1.7	3.6
				B	3.8	0.7	0.5	1
Ankeny, Safe City Demo	Impact Moling	Fat Clay	0.75	A	1.6	0.9	-	-
				B	2.6	0.5	-	-
Ames, State Ave.	HDD	Sandy Lean Clay	12	A	2.1	5.6	1.0	1.6
				B	3.4	3.7	0.5	0.5

-Measurements Not Taken

For the same soil conditions, the increase in lateral earth pressure during the various construction stages as a function of distance are summarized in Figures 4.130, 4.131, and 4.132. Figure 4.130 shows the amount of lateral stress increase as a function of distance measured during the pilot bore. For example, at a distance of 3.8 feet the stress increase was 2.4 psi. The stress increase immediately decreased as a function of distance. Similar trends were observed during the pre-reaming and pipe pullout stages.

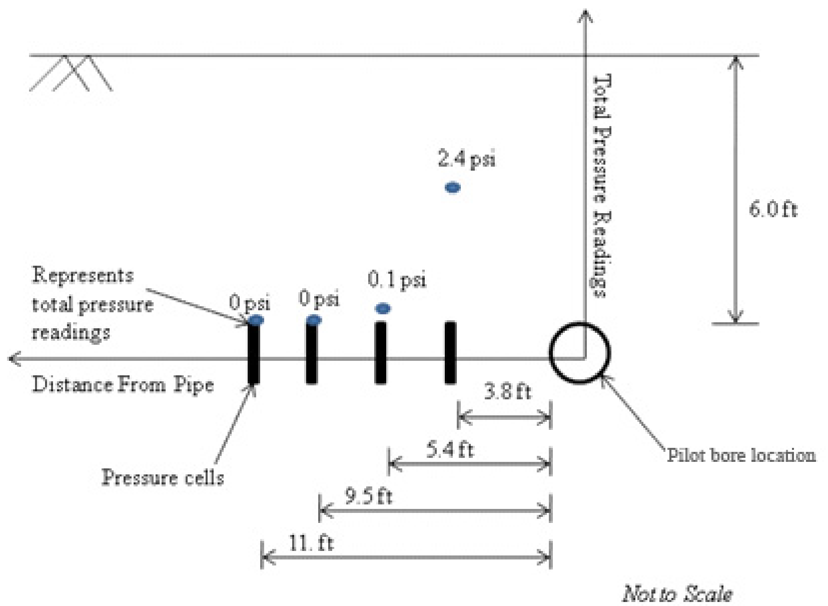


Figure 4.130. Total pressures measured during pilot bore for Ames Hub HDD project installing two 6-inch pipes in well-graded sand with silt and gravel

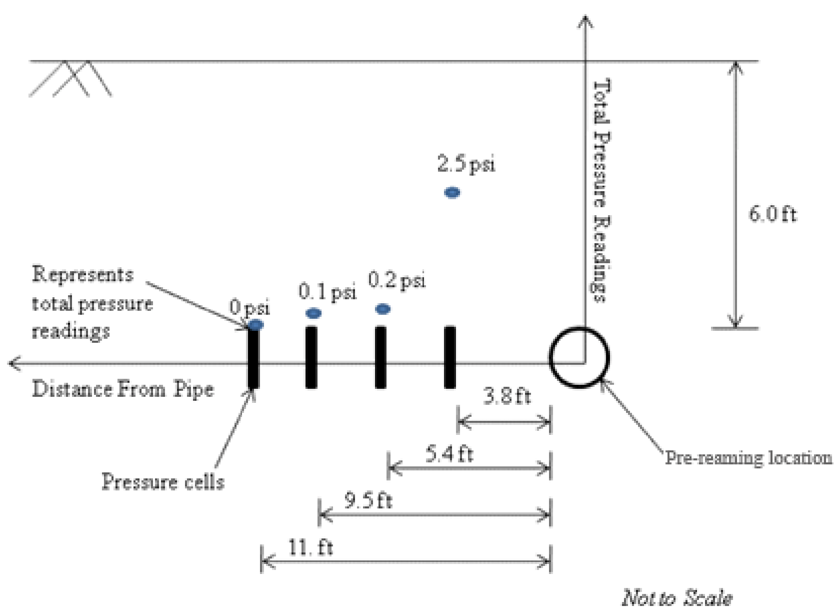


Figure 4.131. Total pressures measured during pre-reaming stage for Ames Hub HDD project

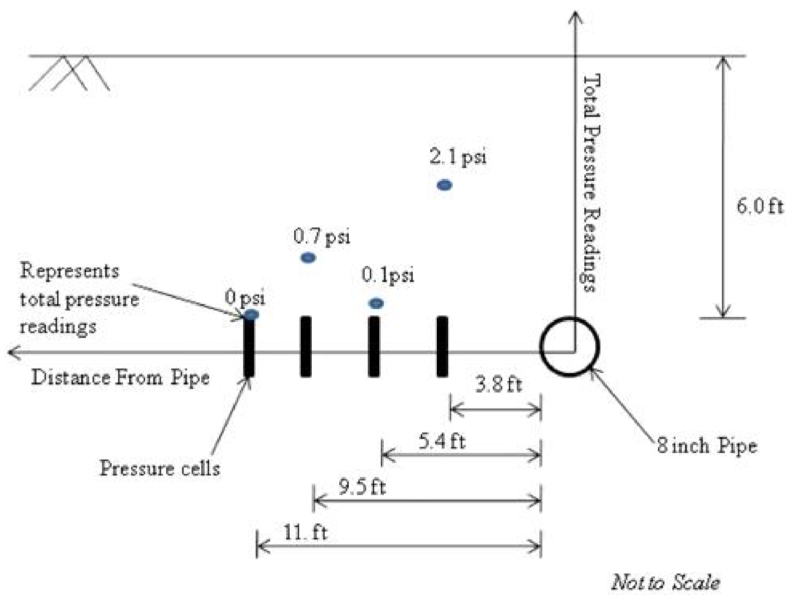


Figure 4.132. Total pressures induced by reamer and pipe pullback stage for Ames Hub project

Several researchers investigated the variation of radical stresses as a function of distance. Yu (2000) proposed the equation below.

$$\sigma_r = P_o + (P - P_o)\left(\frac{a}{r}\right)^2 \tag{1}$$

$$\sigma_r\sigma_r = p_0 + (p - p_0)\left(\frac{a}{r}\right)^3 \tag{2}$$

In the equation, p_0 is the external pressure, which was 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.

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

These calculations were done for a 4-inch pilot bore to compare calculated stresses with the measured values for the bores for the Ames Hub#1 and #2, Seed Science Building #1 and #2, and the Pammel Drive projects. These calculations allowed for direct comparison to the measured pressure results from projects that involved expanding a 4-inch pilot bore. The HDD projects at the Hub and Seed Science Building all fit this criterion. Figure 4.133 shows the calculation results with the measured readings overlaid on the graph.

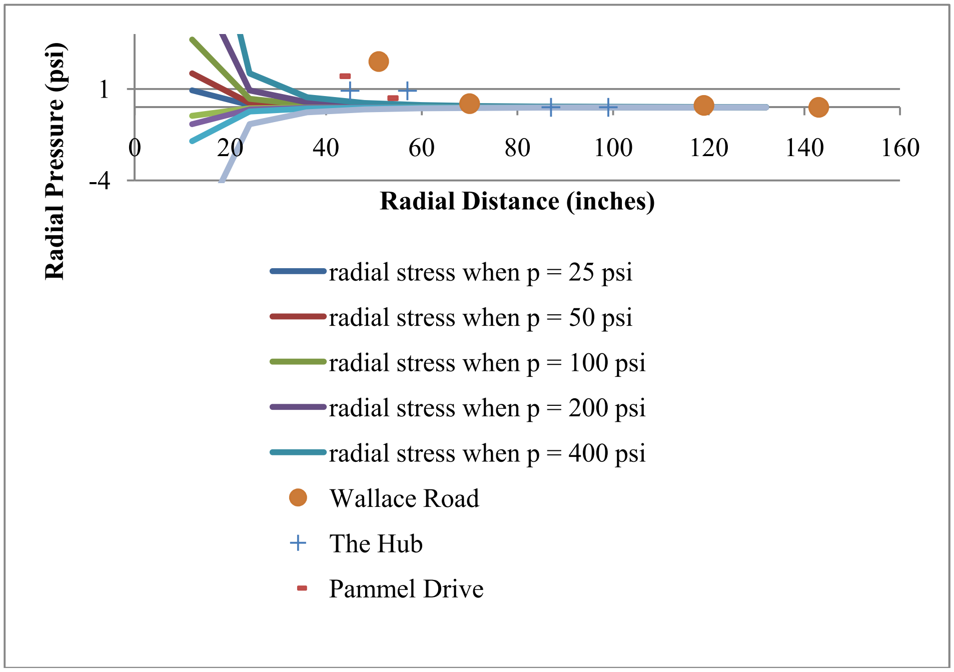


Figure 4.133. Comparison of stress calculations and actual measurements during 4-inch pilot bores

A similar comparison was also made for the State Avenue HDD project, which began with a 5.5-inch pilot bore. These results are shown in Figure 4.134.

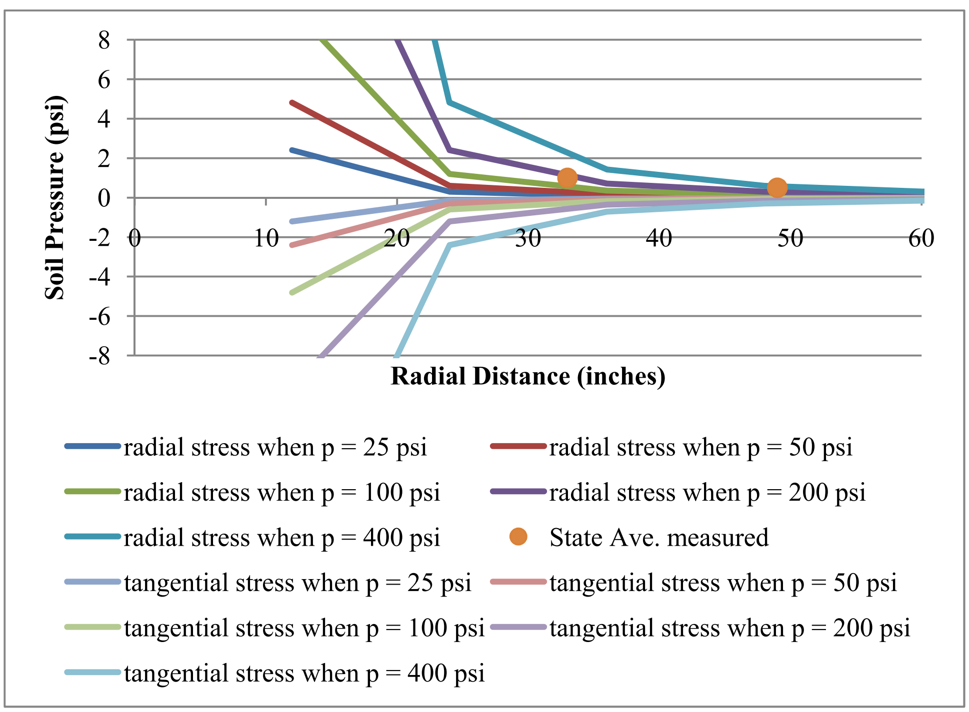


Figure 4.134. Comparison of stress calculations and actual measurements during 5.5-inch pilot bores

Figures 4.133 and 4.134 show a comparison between calculations based on the equation reported by Yu (2000) compared with measured stress. It should be noted that Yu's approach does not account for any variations of soil properties.

Using both a cylindrical cavity expansion analytical approach and a finite element numerical approach, Hunter (2005) investigated the stresses induced during the time of installation for trenchless techniques, especially HDD and impact moles. Assuming a linear elastic perfectly plastic soil model, Hunter (2005) used equation (4), which solves for the radial pressure at the borehole to soil interface.

$$p_r = p_0 + c_u\left[1 + \ln\left(\frac{G}{c_u}\right) + \ln\left(\frac{\delta A}{A}\right)\right] \tag{4}$$

where, c_u is the undrained shear strength, G is the shear modulus, and $\frac{\delta A}{A}$ is the change in cavity area (assuming a very small initial cavity with diameter of 0.01 meters) divided by cavity area after expansion.

Figure 4.135 shows 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.

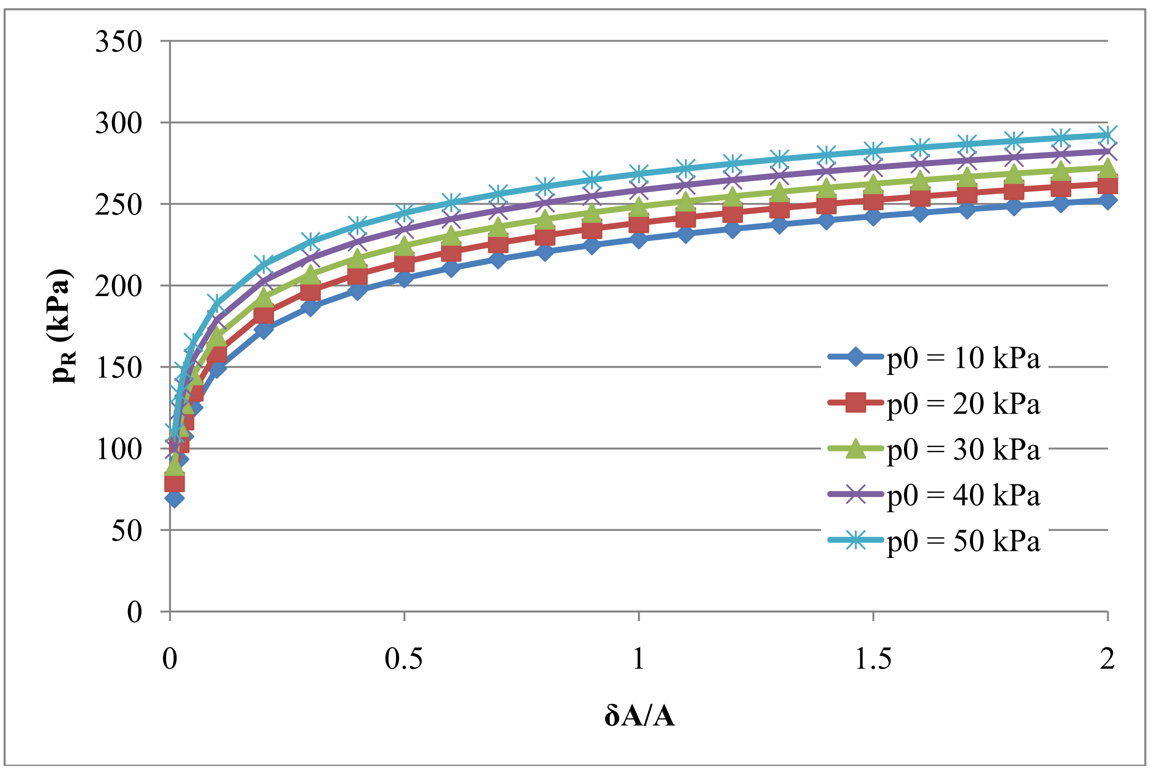


Figure 4.135. Calculation of radial pressure for a given change in area over area and external pressure

Using the computer program Plaxis v7.2, an initial finite element (FE) analysis was conducted in an attempt to model the lateral earth pressure increases that occurred during construction of the HDD project on Pammel Drive. Soil properties for the analysis were obtained from the project's geotechnical report and from the research team's investigation of the Ames Osborn Drive domestic water HDD project, which occurred several hundred feet away from the Pammel Drive project and in similar soil. The effect of the 14-inch reamer enlarging the 5-inch pilot bore at a depth of 6 feet in the sandy lean clay was modeled. After consulting the literature and several contractors, it was assumed that the reaming process in this scenario would have expanded 2 to 3 inches of soil into the surrounding borehole, while the rest of the spoil was removed in the drilling fluid. The exact amount of soil that is radially expanded depends on numerous factors, such as soil conditions, reamer size and type, and drilling fluid mix. Both the 2- and 3-inch soil displacements were axisymmetrically modeled. It was observed that at a distance of 3.6 feet away from the centerline of the borehole (corresponding to 3 feet away from the outer edge of the 14-inch borehole), a 2-inch displacement caused a 3 psi lateral pressure increase (from a baseline pressure of 3 psi) and a 3-inch displacement caused a 4 psi lateral pressure increase. These values were somewhat larger than the 1.5 psi increase measured by pressure cell A during the Pammel Drive project. At a distance of 4.4 feet away from the centerline (corresponding to 3.8 feet away from the outer edge of the 14-inch borehole), a 2-inch displacement caused a 1.7-psi lateral pressure increase and a 3-inch displacement caused a 2.5-psi lateral pressure increase. These values were also larger than those measured by pressure cell B, which measured a 0.5-psi lateral pressure increase.

CHAPTER 5. SUMMARY AND CONCLUSIONS

This research was undertaken with two primary objectives:

- Document the current practices and applications of trenchless technology in the United States and, particularly, in Iowa
- Evaluate the effects of trenchless construction on surrounding soil and adjacent structures

To fulfill these objectives, a literature review was first conducted to assemble information on the current practice of trenchless technologies. The literature review examined the rationale for trenchless technology and introduced the major trenchless construction and rehabilitation methods. Soil investigation methods for trenchless projects, QC/QA, the effects of trenchless technologies on surrounding soil, and design processes were all discussed.

To gain addition insight regarding trenchless technology, trenchless practitioners were surveyed and interviewed. Three separate surveys, with each survey targeting a different geographic region, were sent to professionals. The surveys targeted Iowa, the Midwest, and the entire United States. These surveys and interviews focused on the following four major topics:

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

The Iowa survey garnered 34 respondents, 60% of whom were public employees and 40% of whom were contractors and consultants. The survey results indicated that HDD, auger boring, pipe jacking, and cured-in-place pipe are considered to be the most common trenchless construction methods used in Iowa. The respondents also reported that pipe ramming and pipe bursting are the least favorably viewed of the common methods because of perceived risks associated with these methods. Respondents were asked if they had seen pavement distress or other problems occur as a result of trenchless installations, and 47% reported that they had.

A shorter survey was sent to professionals around the Midwest, and it garnered 32 respondents. Of these respondents, 22% were public employees and 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 observing pavement distress or other problems resulting from trenchless technologies.

Questions from the Midwest survey were included in a larger survey that was conducted by Dr. Mohammed Najafi of the University of Texas at Arlington and sent to state department of transportation employees across the United States. 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 in response to these surveys. Researchers also conducted interviews as part of field and office visits. As a result of these activities, the research team collected many comments related to trenchless technology. Professionals commonly expressed the following 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 because 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 and 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 that involved observing 19 trenchless construction projects. Research activities included documenting construction procedures, noting successes and challenges, interviewing personnel, obtaining soil samples for laboratory testing, and measuring stress changes in the soil near the borehole during construction. Soil samples were tested in the laboratory to better understand the types of soil involved in the trenchless installations. Finally, 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 (Section 4.2) of the field work involved observing 13 trenchless construction projects. For these projects, researchers visited the jobsites, observed and documented construction practices, and noted successes and challenges. Soil samples were tested in the laboratory to evaluate soil properties. "Field Monitoring," as described in Section 4.3, involved the same tasks as "Site Visits" but also included additional soil testing and measuring soil stresses during construction. This additional soil testing made these project investigations more thorough compared to the investigations in the "Site Visits" in Section 4.2. Undisturbed soil samples were recovered and tested in the laboratory. When the appropriate samples and testing equipment were available, samples were tested using confined and unconfined compression, consolidation, and multistage consolidated-undrained triaxial tests.

Soil stresses in the field were measured during the six "Field Monitoring" projects by installing push-in pressure cells in the ground near the bore path before the boring began. These pressure cells provided readings of soil pressure increases experienced as the boring equipment passed the instruments. Soil samples were analyzed in the laboratory 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-foot by 5-foot box culvert. Installation lengths ranged from 24 to 495 feet. These projects were all successfully completed but, in two projects, the HDD caused frac-out and surface heave. The trenchless methods used for the 19 total projects included 1 pipe jacking, 1 tunneling, 1 impact moling, 5 auger boring, and 11 HDD.

The first observed project to experience a surface heave and a frac-out involved an HDD installation of two 4-inch-diameter HDPE pipes in one borehole. The installation depth was 17 feet, and the bore length was 400 feet. Researchers summarized that the cause of this failure was probably incorrect mismatch between the drilling fluid mixture and the soil type encountered. A drilling fluid that was high in bentonite was used because of the contractor's concern that granular soils would be encountered; bentonite drilling solutions reduce the possibility of borehole collapse in granular soils. When clay was encountered, the lack of a clay-inhibiting polymer in the drilling fluid apparently caused the drilling fluid to stick to the gravelly clay soil so that the borehole sealed shut. Fluid pressures rose because drilling fluid was being pumped into the borehole and could not escape. The borehole eventually fractured along the planes of greatest weakness and drilling fluid seeped to the surface. This hypothesis was later supported by the research team's subsurface soil investigation in which permeable sandy soil was observed around the area of frac-out and less permeable clay was observed around the area of heave. A better knowledge of subsurface conditions may have encouraged the contractor to use a different drilling fluid mix and possibly avoid this problem. This example suggests that additional field testing is desirable for HDD projects in which the contractor is uncertain of the subsurface conditions.

The second project that experienced a frac-out was an HDD installation of 8-inch HDPE pipe at a depth of 6 feet and a length of 495 feet. The frac-out may 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 mismatch between drilling fluid mix and the soild and construction procedure, (3) a lack of borehole stability, and/or (4) the speed at which the reamer was pulled through the borehole (possibly too fast). Researchers could not definitely conclude the cause of the frac-out. The research team noted that most of the observed projects did not utilize a soil testing program. Contractors generally believed that they had sufficient experience in the local area and that soil testing was an unnecessary expense.

Calculations were made using cavity expansion theory to predict the pressure increases that could be expected.

Future research could provide trenchless project participants with a better understanding of several trenchless construction methods and of how to avoid pavement damage and other problems. Soil pressure monitoring of other trenchless methods in addition to HDD and impact moling could allow a better understanding of how these methods interact with soil to possibly cause pavement damage and other problems. Also, additional FE modeling of various installation procedures could improve understanding of the conditions that increase the risk of pavement damage and other problems. Additionally, an improved knowledge of the causes of HDD drilling fluid pressure buildups that lead to heave and frac-out could lead to a decreased risk during HDD installation. Based on survey comments and interview results, surface heave is considered to be the most common concern regarding the use of HDD.

Because the projects observed by the research team were successful overall, trenchless technologies appear to be effective methods for installing utility pipe in areas where open-trenching is undesirable. However, the contractor's experience level is very important, and it is also important to conduct soil testing in areas of uncertain subsurface conditions. It is expected that trenchless technologies will become more popular as project participants gain experience and technology improves.

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APPENDIX A. DETAIL FROM SURVEY AND INTERVIEW RESPONSES

Iowa, Midwest, 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 indicated that current methods are not always adequate. The following are individual comments:

- Problems experienced with CIPP include sags in the felt liner, poor adhesion, and difficulty telling if appropriate temperatures for curing are uniformly maintained
- 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 that are available
- Contractor shortcuts
- In some cases, companies reduce 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 bore route
- Soil borings can miss localized problem areas
- Okay for water mains
- Testing doesn't consider soil stability

Iowa survey respondents were asked about which soil investigation methods are currently used prior to trenchless construction. The following comments summarize their responses:

- 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 as follows:

- 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 of the work performed by utility companies needed by the community
- 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 as follows:

- 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 compared to open-cut projects
- Dewatering can be a challenge during tunneling and microtunneling
- Trenchless methods avoid disrupting the public and business owners and reduce the carbon footprint 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 his/her employer has recently started using pipe ramming as an installation option. The respondent 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 undertaken or made to trenchless technologies to make these technologies more feasible. The following were suggestions:

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

An interview with an Iowa trenchless technologies contractor yielded a large amount of information about the trenchless construction industry and the problems the industry has commonly encountered in the field. The following are suggestions and comments from the interview:.

- Given the necessary specifications, experienced contractors need to be given the flexibility to use whatever methods they deem necessary to complete a job. 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, which 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; looks for blow counts, water content, and location of water table; and interprets 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 greater than 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 error 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 deep 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 but has gotten settlement at 20-feet deep.
- 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 pullback and caused surface heave.

One city designer told researchers that a franchised utility company installed conduits approximately 4 to 6 inches in diameter at shallow depths. These diameters seem to be associated with 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 jobsites, designers, and equipment vendors.

- During HDD work on the ISU 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 signal transmission 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 rule is less than the 2 to 1 ratio that one contractor informed the research team 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 to 1 bentonite to water ratio is used in sand because it is a challenge to stabilize a borehole in sandy material. This ratio 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 accuracy 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 drilling 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 three times longer compared 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 compared to steering in clay.
- Most common problems encountered: existing utilities, rock, and water.
- If the 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 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 pre-ream.
- 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, 2 feet of clearance is necessary 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 compared to HDD
- A 2 by 2 foot 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.

APPENDIX B. IOWA SURVEY

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 in the US excluding Iowa) Which trenchless techniques do you think are most commonly used in the U.S.?

- [ ] 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 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?

- 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. AUGUR BORING AND HDD INSTALLATION PROJECTS

Des Moines, SE 6th Avenue and 64th Street, Auger Bore

Project Information

This project was located at the T-intersection of Southeast 6th Avenue and Southeast 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 D.1. No plan view from the bid documents was available.

Figure D.1. Location of auger boring project on 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 that method is not appropriate for rigid concrete pipes.

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 in comparison to other layers 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. 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 D.2). After the nearby existing utilities were located, the launching pit at point A was dug with an excavator. Shoring was not used, and instead the sides of the launching pit were terraced (see Figure D.3). 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 D.4) 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 D.5). 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 D.2. Potholing by edge of pavement

Figure D.3. Launching pit with terraced sides and a gravel base

Figure D.4. Twenty four inch concrete pipe

Figure D.5. Launching pit at point A being 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 information was obtained from conversations with contractor personnel.

Tests carried out on the disturbed samples included moisture content, gradation, liquid limit, and plastic limit. Results from these tests are summarized in Table D.1. 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 feet. The gradation curve is shown in Figure D.6. From 16 feet down to 20 feet, sandy clay was observed. The water table was located deeper than 20 feet.

Table D.1. SE 6th Avenue and SE 64th Street project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0 – 2.0	**Peat (Pt)**	-	-	-	-
0 – 16.0	**Sandy lean clay (CL)**	28.7	27.1	15.0	12.1
16.0 – 20.0*	**Sandy clay**#	-	-	-	-

*Top of the 32 inch pipe at depth of 17 feet; Location of water table is below this level
#Reported by the contractor
-Test not conducted

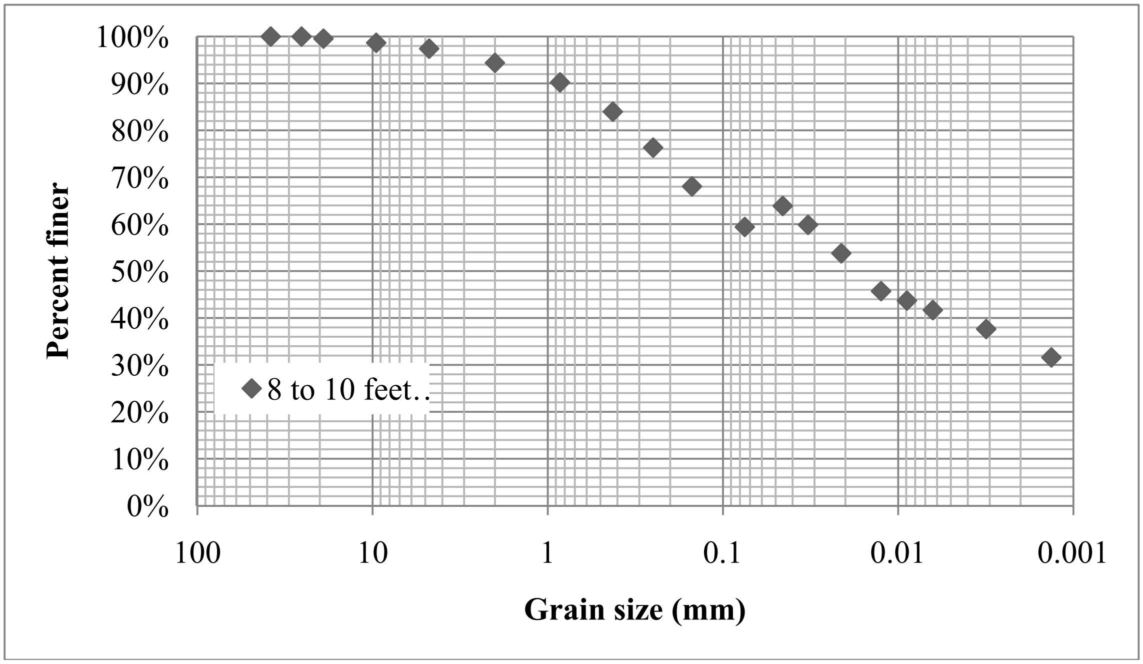


Figure D.6. Soil gradation curves for depths of 8 to 10 feet

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.

Des Moines, 62nd and Grand, Auger Bore

Project Information

This project was located at Grand Avenue, near its intersection with 62nd 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 a set of 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 D.7. Drawings for the project were not made available to the researchers.

Figure D.7. The location of the auger boring project at Grand Avenue in Des Moines, IA; bore path in red

Trenchless Method Selection

Trenchless construction was selected to avoid closing Grand Avenue and the Iowa Interstate Railroad tracks (see Figure D.8). This gave trenchless construction an important 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 method is not appropriate for installing rigid steel pipe.

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 type of soil to be expected. Cohesive soil was expected, and that proved to be the soil type encountered during the entire bore.

Trenchless Installation

- After nearby existing utilities were located, potholing was used to confirm the position of utilities that were close to the bore path (see Figure D.9). Next the launching pit at point

A and the receiving pit at point D were excavated 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 D.10). 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 D.11), taking only two days to complete. Part of the reason for the quick boring was because of favorable 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, approximately 30 minutes were required 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 depth was adequate (12 feet).
- The bore emerged at the receiving pit at point B after two days of boring. The installed pipe 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.

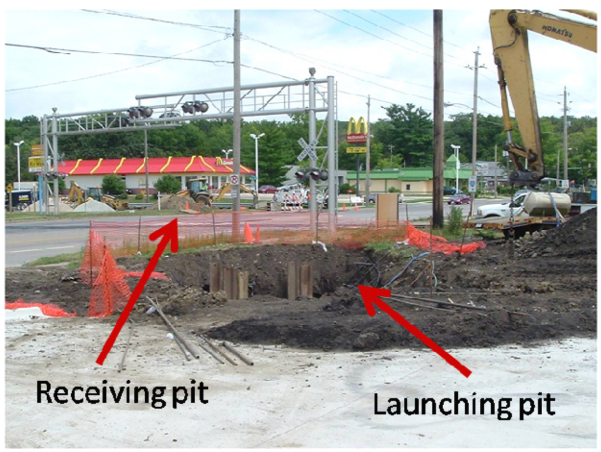


Figure D.8. View of the project looking from the launching pit at point A across Grand Avenue and the railroad tracks to the receiving pit at point B

Figure D.9. A potholing pit covered and marked

Figure D.10. A backhoe lowers a new 20 foot section of steel pipe into the launching pit

Figure D.11. Pipe and 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. Samples were not recovered from this project.

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 a street and railroad track. The bore covered a distance of 140 feet. The casing pipe was installed to shield a new water main that connected a newly built retail store with the city's water distribution network.

The soil was observed to be a clay, which is favorable for auger boring. The favorable 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.

Ames, West Osborn Drive, Ductile Iron Waterlines

Project Information

This project was located along Osborn Drive, on the campus of Iowa State University and was constructed 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 D.12. Drawings were not available to the researchers because the work was part of a change order.

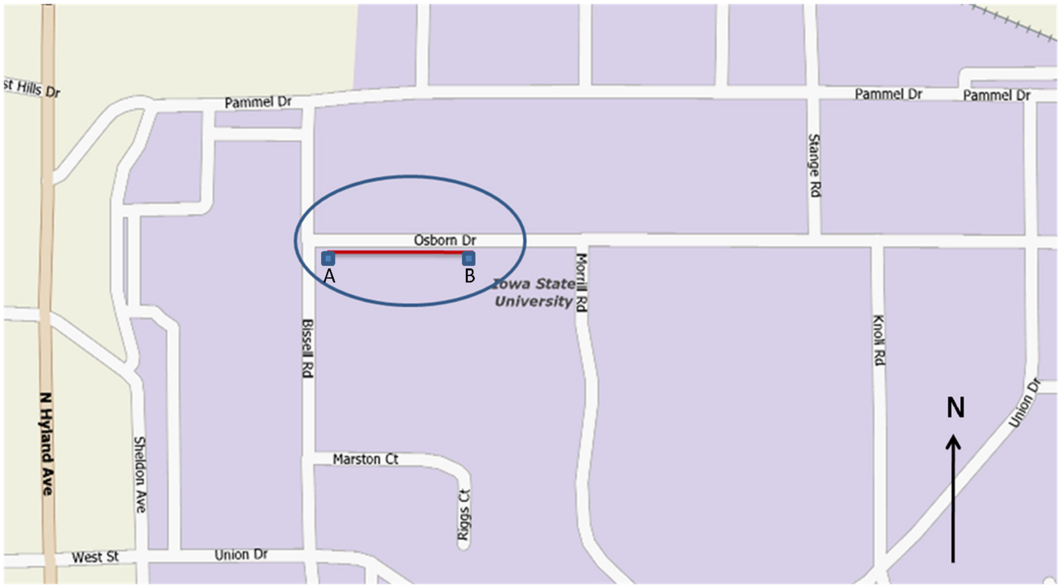


Figure D.12. The location of the auger boring project at Osborn Drive in Ames, IA; bore path in red

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 practical 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 layer of clay 20 feet deep that was underlain by a deep stiff clay layer. Dewatering was not necessary as the boring was performed above the water table and no problems were expected from sand seams, boulders, or pieces of debris. 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 excavated. 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 D.13). 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, and waterline system was not 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 D.14). 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 did not 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 necessitated 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 total distance of 80 feet. 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 D.15). 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 the 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 D.16. The two installed casings with the first ductile iron pipe installed is shown in Figure D.17. 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 D.18. This completed the pipeline. No additional problems were encountered on this project and no surface heave or settlement was observed.

Figure D.13. Launching pit with shoring and a gravel base

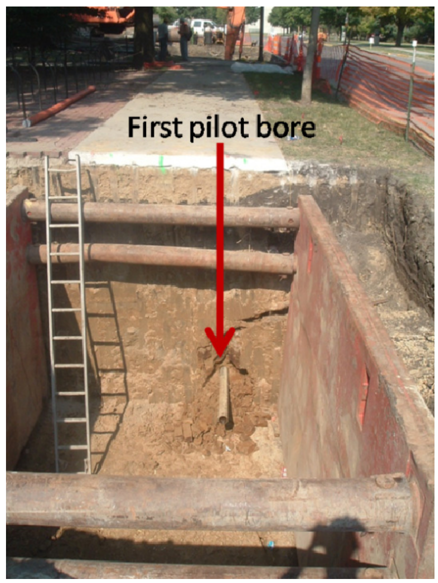


Figure D.14. First pilot bore emerging in the receiving pit

Figure D.15. First casing being bored from the launching pit

Figure D.16. The first ductile iron water pipe with casing spacers attached is placed by backhoe into the steel casing

Figure D.17. 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 D.18. The final connection of pipe sections in the launching pit

Research Team Actions

The research team recovered disturbed samples, which were taken from the middle of the thick, uniform clayey layer. The samples were taken 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 D.2. The gradation curves are shown in Figure D.19. The soil profile between the ground surface and the pipe locations consists of two soil layers. The first layer, between the ground surface and a depth of 1.5 feet, 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 D.2. Ames, West Osborn Drive auger boring project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0 - 1.5	**Peat (Pt)**	-	-	-	-
1.5 – 10*	**Sandy Lean Clay (CL)**	9.8	19.1	13.4	5.7

*Top of the 24 inch pipe at a depth of 10 feet,
Location of water table is below this level,
-Tests not conducted

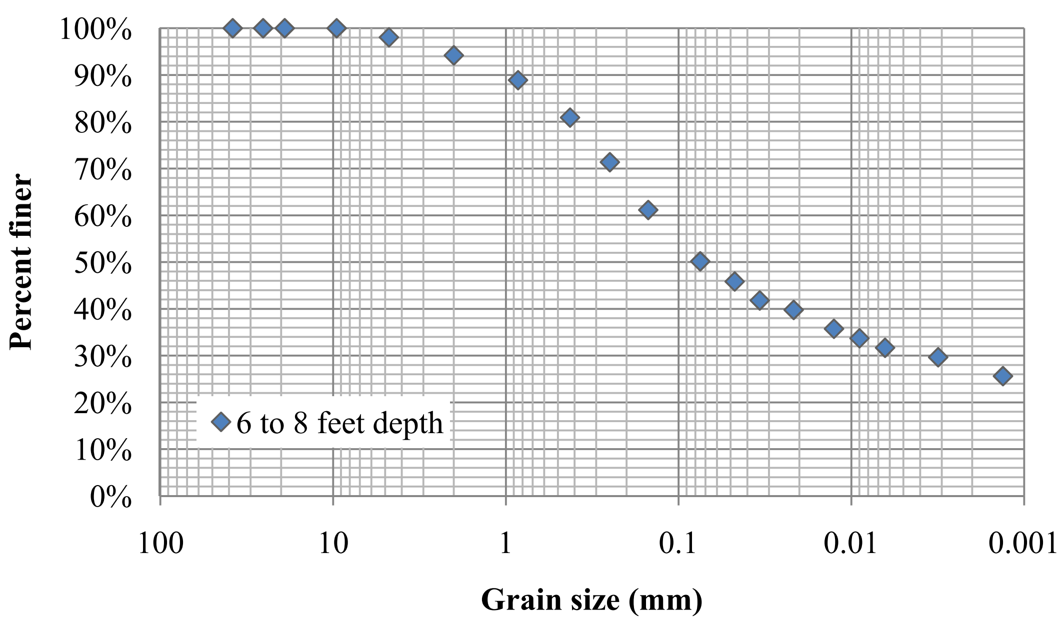


Figure D.19. Soil gradation curve for depths of 6 to 8 feet

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 at the time of the project 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.

Ames, Osborn Drive, Chilled Water, HDD

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 D.20. The boring ran east to 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 D.21.

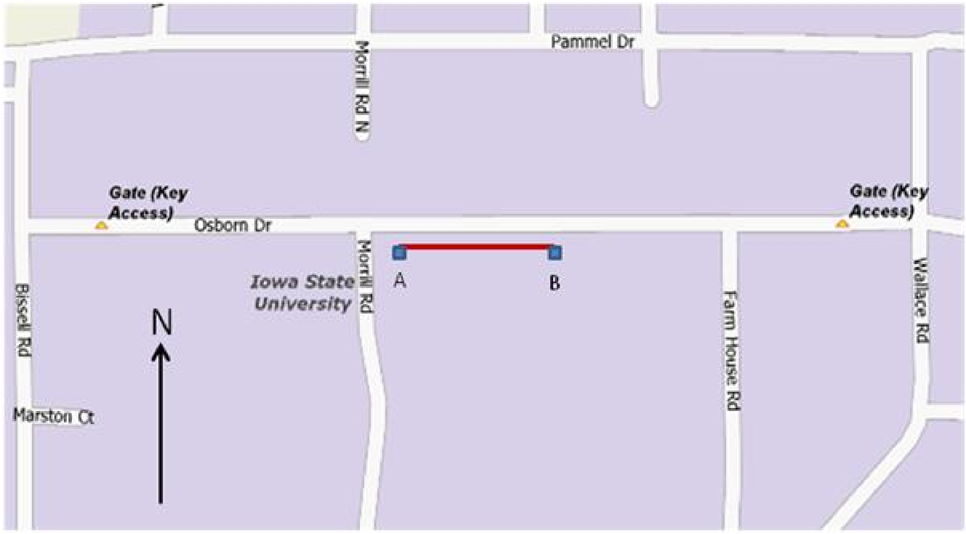


Figure D.20. The location of the HDD along Osborn Drive in Ames, IA; bore path in red

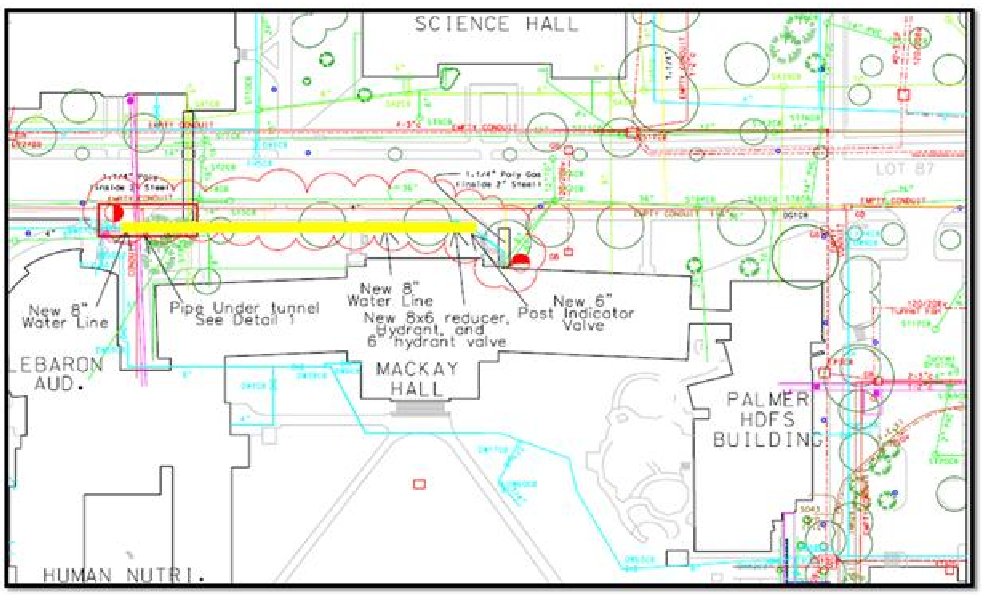


Figure D.21. Plan view of the project site; bore path in yellow

Trenchless Method Selection

Trenchless construction was selected by the owner to avoid damaging existing trees that had a protected status. Additionally, the closure, demolition, and reconstruction sidewalks and driveways in the bore path were also avoided.

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 to be too close to the bore path. 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 cooect and remove drilling fluid that contained 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 marketed as being capable of improving 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 D.22). The pilot bore depth was 6 feet below the surface except at a location where 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. 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 D.23) for the pre-reaming stage. The directional drilling machine was switched from push mode to pull mode. The reamer was then 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 point B with the 14 inch reamer still attached. The 8 inch HDPE pipe (see Figure D.24) was then attached to the reamer, and the reamer and pipe were pulled back from point B to point 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 capped until it was

connected to the rest of the pipeline. The installation was completed in four days, two of which were spent on the actual boring.

Figure D.22. Directional drilling machine

Figure D.23. Fourteen inch diameter reamer

Figure D.24. Eight inch HDPE pipe

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 taken to the laboratory in sealed plastic bags. Additional soil information was obtained by interviewing 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 D.3. 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 D.25. The water table was located deeper than 9 feet.

Table D.3. Ames, West Osborn Drive first HDD project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Strength (psf)
0 - 1.5	**Peat (Pt)**	-	-	-	-	-
1.5 – 9*	**Sandy Lean Clay (CL)**	14.8	27.8	14.9	12.9	-

*Top of the 8 inch pipe at a depth of 6 feet. Location of water table is below this level.
-No tests taken

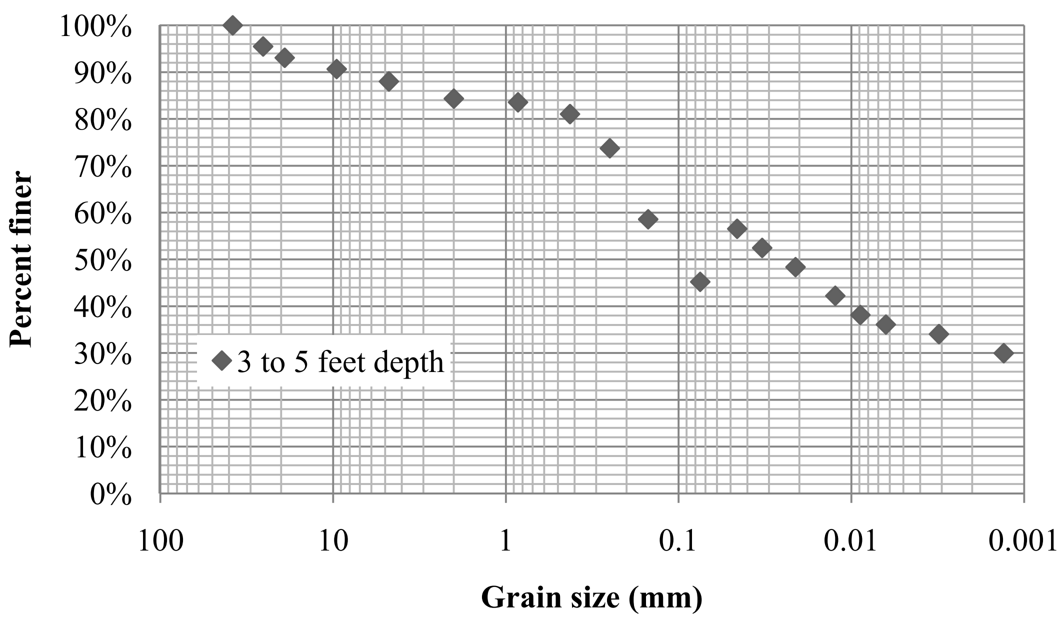


Figure D.25. Soil gradation curve for depths of 3 to 5 feet

Key Findings

This project involved the installation by horizontal directional drilling of a single HDPE chilled water pipe 330 feet long and 8 inches in diameter. 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 pre-reaming 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; this can happen if higher speed is used. Second, the drill rig had enough power to provide the necessary pullback forces of this bore. Third, the correct selection of a drilling fluid was also important. 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.

Tama, Railroad Crossing, Auger Bore

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 D.26 for each of three bores. No plan view from the bid documents was available.

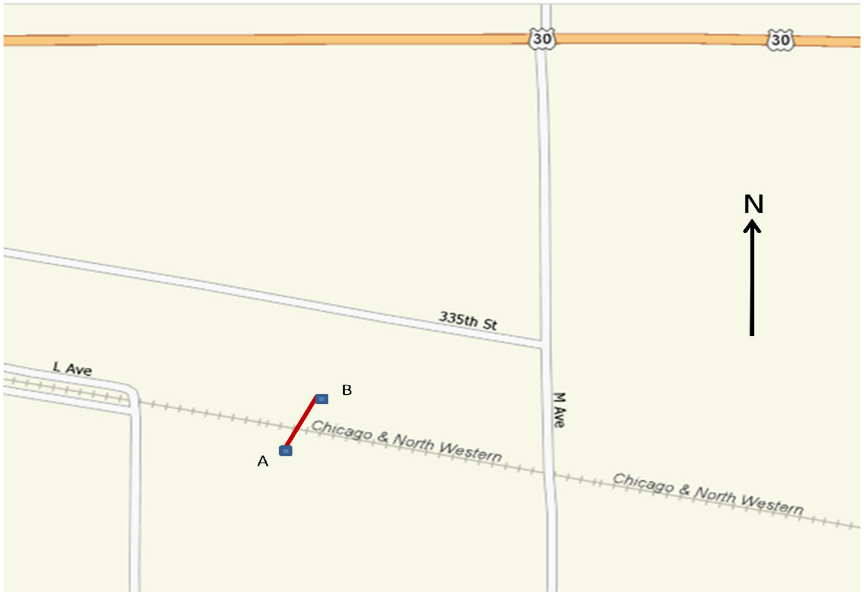


Figure D.26. The location of the auger boring project in Tama, IA; bore path in red

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 the 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 conducted by researchers.

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 moved into the launching pit and placed on its track (see Figure D.27).
- The initial placement of the boring machine tracks was very important, because the boring machine had no steering mechanism. The accuracy of the bore was important, because of the potential damage to the railroad tracks above, and because of the need for the drainage pipe to be gravity flow. Careful construction was important, because railroad tracks have a 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 D.28).

- 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 D.29). 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 D.27. Auger boring machine on its track against the sheet pile thrust block

Figure D.28. A new 20 foot long pipe section being welded to the pipeline

Figure D.29. The third pipe being installed

Research Team Actions

The research team observed the installation and interviewed crew members. Soil samples were not taken 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 the 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.

Ames, Forker Building, HDD

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 D.30. The bore path runs east and west, parallel to Beach Road on the Iowa State University campus (see Figure D.31 and Figure D.32). The research team observed the installation and interviewed crew members.

Figure D.30. The location of the HDD along Beach Road in Ames, IA; bore path in red

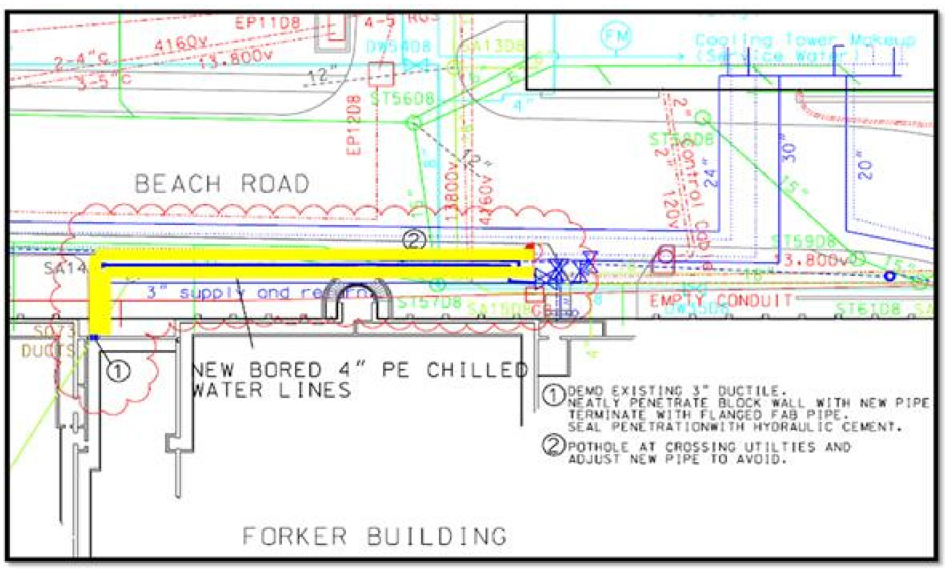


Figure D.31. Plan view of the project site; bore path in yellow

Trenchless Method Selection

Trenchless construction was selected by the owner to avoid damaging protected trees along 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 higher risk than HDD.

Soil Conditions

Soil testing was not 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 better understand 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 D.32) on the east side of the site (point A in Figure D.30) for the first 180 foot 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 was then removed from the drill string and replaced by a 5 inch reamer for the pre-reaming 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 D.30 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 5 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 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, the 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 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 D.33.

Figure D.32. Directional drilling machine

Figure D.33. The 4 inch diameter HDPE pipes connected to the building near point B

Research Team Actions

The research team recovered disturbed samples, which were obtained from the middle of the thick sandy layer. The samples were taken 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 D.4. 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 D.34. The water table was located deeper than 7 feet.

Table D.4. Ames, Beach Road project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Compressive Strength (psf)
0 – 1.0	**Peat (Pt)**	-	-	-	-	-
1.0 – 7.0*	**Clayey sand (SC)**	15.9	27.3	16.4	10.9	-

*Top of the 4 inch pipes at a depth of 6 feet,
Location of water table is below this level
-Test not performed

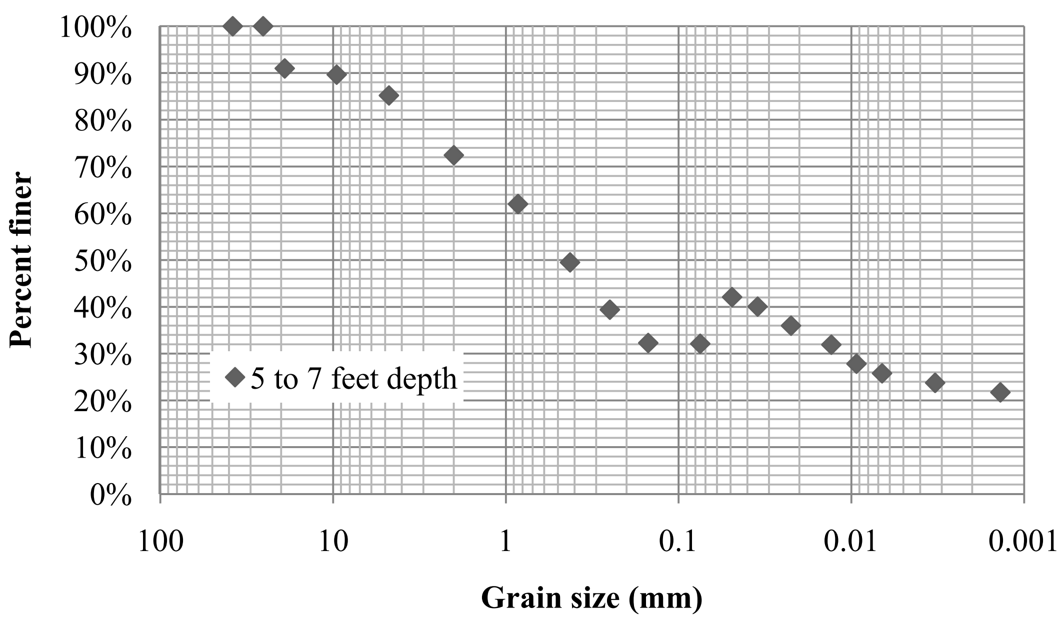


Figure D.34. Soil gradation curves for depths of 5 to 7 feet

Key Findings

This project involved the installation by horizontal directional drilling of two parallel heat welded HDPE chilled water pipes 180 feet long and 4 inches in diameter. 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 pre-reaming 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.

Boone, 210th and Quartz, HDD

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 D.35. The bore path runs north and south, parallel to Quartz Avenue (see Figure D.36). The research team observed the installation and interviewed crew members.

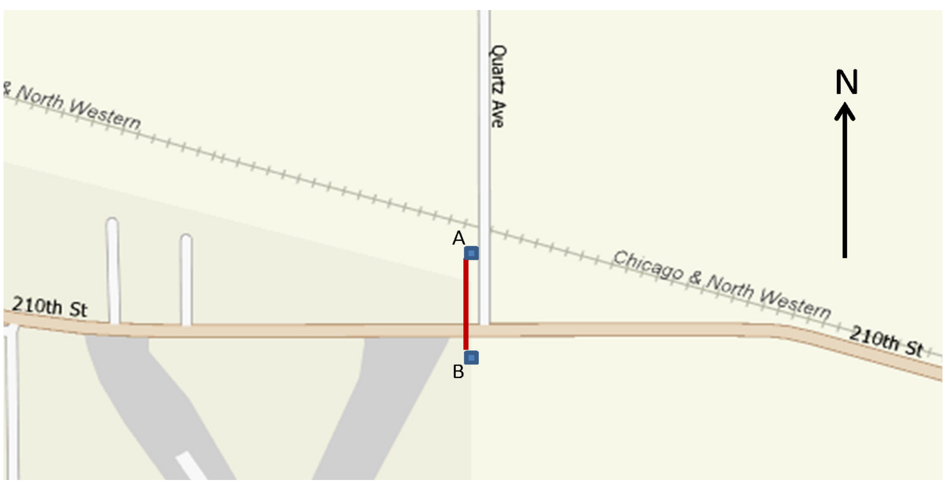


Figure D.35. The location of the HDD project across 210th Street in Boone, IA; bore path in red

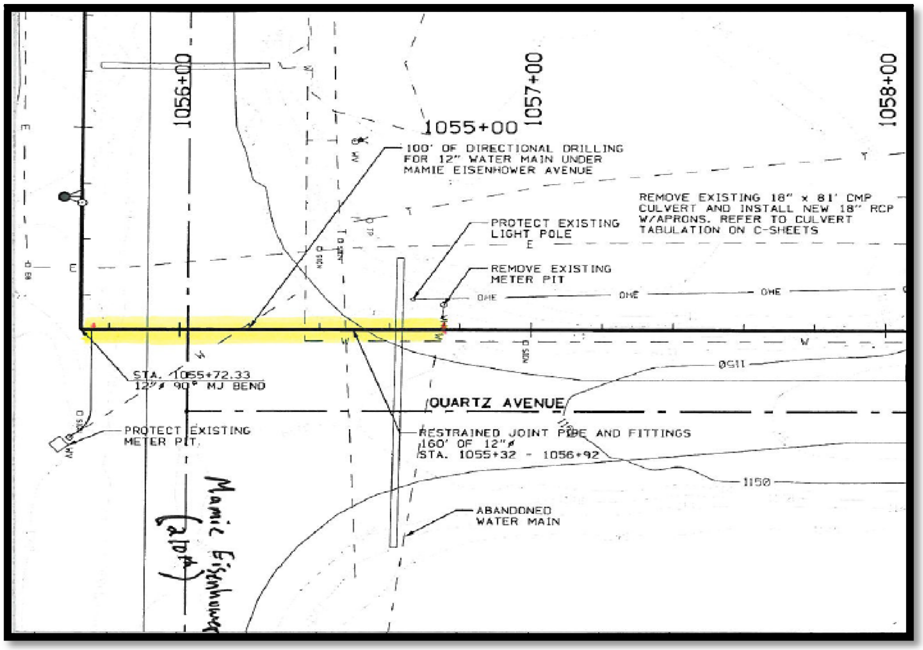


Figure D.36. Plan view of the project site; bore path in yellow

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-cutt construction. The owner considered the project to be appropriate for either HDD or auger boring; however, the bid for the HDD alternative was lower.

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 D.35) for the 100 foot bore (see Figure D.37). 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. Although the minimum depth of cover was specified to be 5 feet, most of the pipe was installed at a depth of about 8 feet. The boring crossed an abandoned water main, but no other utilities were present in the area.

- A drilling fluid consisting of MAXBORE HDD™ sodium bentonite, a clay-inhibiting polymer, was used during the boring (see Figure D.38). 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 D.39). 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 pre-reaming 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 substantially increase the borehole diameter without using incrementally larger reamers was due to the relatively short length of run (100 feet) and the fact that the contractor was using a relatively largedrill rig. The reaming step was completed successfully, and the reamer emerged by the drill rig at point A on Figure D.35. 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 was later connected to the rest of the pipeline. This installation was completed in less than a day.

Figure D.37. The directional drilling machine

Figure D.38. Drilling fluid being vacuumed from the bottom of the exit pit at B

Figure D.39. Five 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 D.5. The 12 inch pipe was installed between the depths of 5 and 8 feet. The samples recovered by the research team confirmed that low plasticity clay was found directly above the installation, from 5 to 8 feet. The gradation curve is shown in Figure D.40. From 2 feet down to 8 feet, sandy lean clay was observed. The water table was located deeper than 8 feet.

Table D.5. Boone 210th Street and Quartz Avenue project soil parameters

Depth (ft)	USCS Classification	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unconfined Compressive Strength (psf)
0 – 2.0	**Peat (Pt)**	-	-	-	-	-
2.0 – 10.0*	**Sandy lean clay (CL)**	15.8	23.8	15.5	8.3	-

*Top of the 12 inch pipe at a depth of 8 feet;
Location of water table is below this level
-Tests not conducted

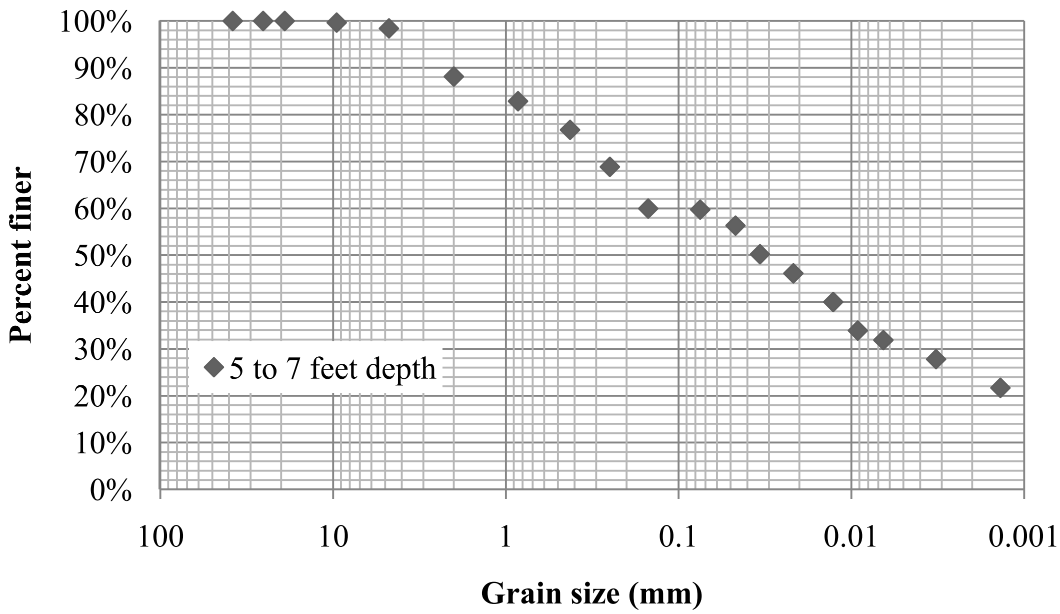


Figure D.40. Soil gradation curves for depths of 5 to 7 feet

Key Findings

This project involved the installation by horizontal directional drilling of a single PVC water pipe with a length of 100 feet and a diameter of 12 inches. 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 pre-reaming using an 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 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.

Boone, Marion Street, HDD

Project Information

This project was located at the intersection of West First Street and Marion Street in Boone, Iowa, and was constructed 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 D.41. The boring runs north-south, parallel to Marion Street (see Figure D.42). The research team observed the installation and interviewed crew members.

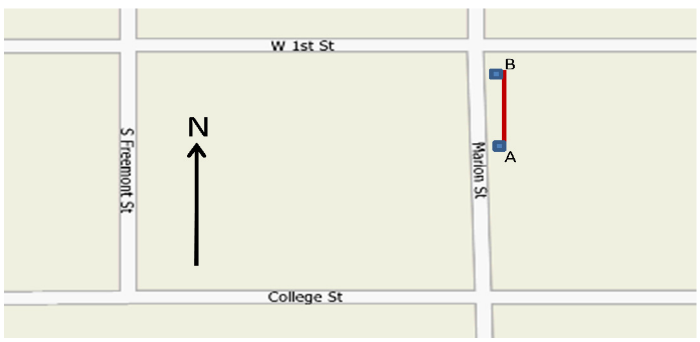


Figure D.41. The location of the HDD installation along Marion Street in Boone, IA; bore path in red

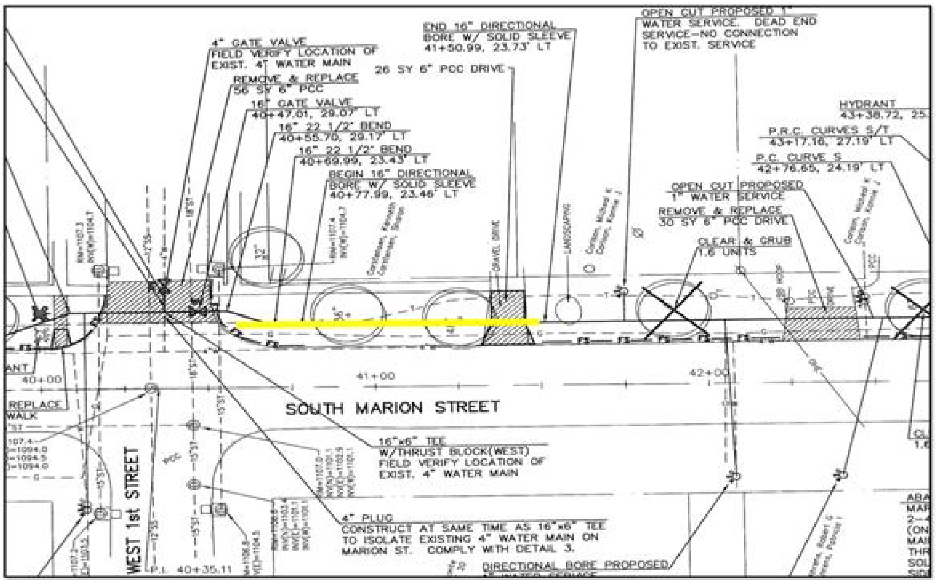


Figure D.42. Plan view of the project site; bore path in yellow

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 path. 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 be 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 D.41) for the 75 foot bore (see Figure D.43). 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 there was adequate distance 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, a clay-inhibiting polymer, was used during the boring (see Figure D.44). The bentonite was marketed to

provide suspension, bore stability, filtration control, and to help reduce drag (see Figure D.45). 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 pre-reaming stage (see Figure D.46). 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 length of run (75 feet) and because the drill rig had sufficiently high pull back force. The reamer successfully emerged by the drill rig at point A on Figure D.41. 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 D.47). 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 was later connected to the rest of the pipeline. The installation was completed in less than a day.

Figure D.43. The directional drilling machine

Figure D.44. Drilling fluid at the bottom of the exit pit at B

Figure D.45. Drilling fluid close-up showing texture

Figure D.46. Twenty-four inch wagon wheel reamer

Figure D.47. Sixteen 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

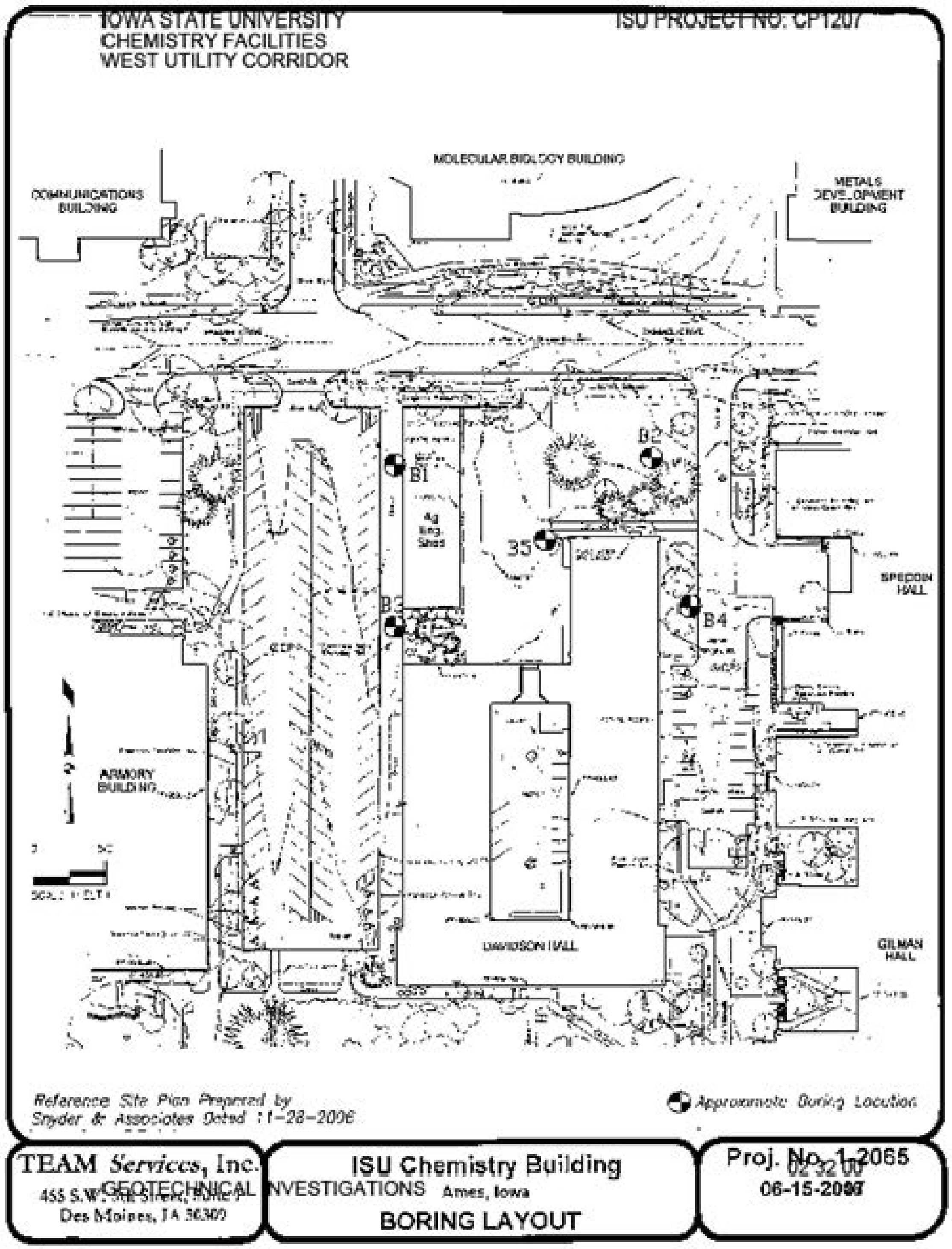
This project involved the installation by horizontal directional drilling of a single PVC water pipe 75 feet long and 16 inches in diameter. The depth to the top of pipe was about 5.5 feet.

The pipe was installed by first drilling a 5 inch pilot bore, followed by pre-reaming 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.

APPENDIX E. SOIL BORING LOGS FROM PAMMEL DRIVE HDD PROJECT



IOWA STATE UNIVERSITY
CHEMISTRY FACILITIES
WEST UTILITY CORRIDOR

ISU PROJECT NO. CP1207

LOG OF BORING NO. 1

OWNER	ARCHITECT/ENGINEER
IOWA STATE UNIVERSITY	
SITE: Ames, Iowa	PROJECT: Iowa State University Chemistry Building

Approx. Surface Elev.: 954.0 ft.

DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF
			SAMPLES				TESTS		
Topsoil--Organic CLAY, trace sand and gravel, very dark brown		OL	1	AS			23.2		
				HS					
3.0 — 951.0		OL	2	ST	10"		32.7		3500*
Sandy lean CLAY, trace gravel and ferrous staining, olive brown, very stiff	5			HS					
		CL	3	ST	18"		12.3	122	7500*
	10			HS					
--becomes hard @ 10.5'		CL	4	ST	14"		15.1	118	4920 8500*
	15			HS					
--ferrous ends, color change to dark gray and becomes very stiff @ 16'		CL	5	SS	19"	13	13.1		8000*
	20			HS					
		CL	6	SS	17"	17	14.2		8000*
	25			HS					
		CL	7	SS	16"	18	14.8		7000*
	30			HS					
		CL	8	SS	20"	16	14.7		
	35								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS			
WL	None	WD None	24 hrs.
WL	None	AB	
WL			

TEAM Services, Inc.
GEOTECHNICAL INVESTIGATIONS

BORING STARTED	6-5-07		
BORING COMPLETED	6-5-07		
RIG	Rig 112	FOREMAN	MG
APPROVED	RED	JOB #	1-2065

02 32 00
19

LOG OF BORING NO. 1

OWNER: IOWA STATE UNIVERSITY

ARCHITECT/ENGINEER:

SITE: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	TESTS: MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF
Sandy lean CLAY, trace gravel, dark gray, very stiff				HS					
		CL	9	SS	21"	15	15.1		7000*
	40			HS					
		CL	10	SS	17"	11	14.9		6500*
	45			HS					
--color change to dark olive gray and becomes hard @ 47'		CL	11	SS	16"	30	12.9		
50.0 — 904.0	50								
Bottom of Boring									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS

WL	None	WD	None 24 hrs.
WL	None	AB	
WL			

GEOTECHNICAL INVESTIGATIONS

TEAM Services, Inc.

BORING STARTED	6-5-07		
BORING COMPLETED	6-5-07		02 32 00 20
RIG	Rig 112	FOREMAN	MG
APPROVED	RED	JOB #	1-2065

IOWA STATE UNIVERSITY
CHEMISTRY FACILITIES
WEST UTILITY CORRIDOR

ISU PROJECT NO. CP1207

LOG OF BORING NO. 2

Page 1 of 2

OWNER	ARCHITECT/ENGINEER
IOWA STATE UNIVERSITY	
SITE: Ames, Iowa	PROJECT: Iowa State University Chemistry Building

Approx. Surface Elev.: 952.5 ft.

DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF
			SAMPLES				TESTS		
Topsoil--Lean CLAY, with sand, trace organic matter, dark brown		CL	1	AS			17.4		
2.0 — 950.5				HS					
Sandy lean CLAY, trace gravel, olive brown, stiff		CL	2	ST	24"		14.6	118	3500*
	5			HS					
--trace ferrous staining, color change to light olive brown and grayish brown and becomes very stiff @ 6'		CL	3	ST	17"		11.5	125	8000*
	10			HS					
--color change to dark gray @ 11'		CL	4	SS	13"	20	12.0		
	15			HS					
--ferrous staining ends @17'		CL	5	SS	15"	18	13.9		8000*
	20			HS					
		CL	6	SS	17"	15	14.7		8000*
	25			HS					
		CL	7	SS	17"	12	15.1		7000*
	30			HS					
		CL	8	SS	21"	15	15.2		
	35								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS			BORING STARTED	6-7-07
WL ∇ None	WD ▼ None	24 hrs.	BORING COMPLETED	6-7-07
WL None	AB	TEAM Services, Inc.	RIG Rig 112	FOREMAN MG
WL			APPROVED RED	JOB # 1-2065

GEOTECHNICAL INVESTIGATIONS

02 32 00 - 21

IOWA STATE UNIVERSITY
CHEMISTRY FACILITIES
WEST UTILITY CORRIDOR

ISU PROJECT NO. CP1207

LOG OF BORING NO. 2

OWNER: IOWA STATE UNIVERSITY	ARCHITECT/ENGINEER:
SITE: Ames, Iowa	PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG	DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	TESTS MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF
	Sandy lean CLAY, trace gravel, dark gray, very stiff				HS					
			CL	9	SS	18"	15	15.5		
		40			HS					
			CL	10	SS	18"	19	15.4		
		45			HS					
	--color change to olive and becomes hard @ 47'		CL	11	SS	19"	22	16.3		
	50.0 Bottom of Boring 902.5	50								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS			
WL	None	WD	None 24 hrs.
WL	None	AB	
WL			

TEAM Services, Inc.

GEOTECHNICAL INVESTIGATIONS

BORING STARTED	6-7-07		
BORING COMPLETED	6-7-07		02 32 00 22
RIG	Rig 112	FOREMAN	MG
APPROVED	RED	JOB #	1-2065

LOG OF BORING NO. 3

OWNER	ARCHITECT/ENGINEER
IOWA STATE UNIVERSITY	
SITE: Ames, Iowa	PROJECT: Iowa State University Chemistry Building

Approx. Surface Elev.: 954.0 ft.

Depth (ft.)	Description	USCS Symbol	Number	Type	Recovery	SPT-N Blows / Ft.	Moisture, %	Dry Density PCF	Unconfined Strength PSF
	Lean CLAY, with sand, trace organic matter, dark grayish brown	CL	1	AS			23.4		
2.0 (952.0)				HS					
	Sandy lean CLAY, trace gravel, olive brown, stiff	CL	2	ST	23"		15.1	115	3000*
5				HS					
	--trace ferrous staining and become very stiff @ 7'	CL	3	ST	27"		12.2	124	6500*
10				HS					
	--color change to light olive brown @ 12'	CL	4	ST	21"		16.0	119	4034 7000*
15				HS					
	--ferrous staining ends and color change to dark gray @ 17'	CL	5	SS	19"	18	16.5		
20				HS					
		CL	6	SS	16"	17	13.5		8000*
25				HS					
		CL	7	SS	17"	16	14.1		7500*
30				HS					
		CL	8	SS	18"	15	15.7		7000*
35									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS				
WL	49'	WD	None	24 hrs.
WL		AB		
WL				

TEAM Services, Inc.
GEOTECHNICAL INVESTIGATIONS

BORING STARTED	6-5-07		
BORING COMPLETED	6-5-07		
RIG	Rig 112	FOREMAN	MG
APPROVED	RED	JOB #	1-2065

02 32 00
23

LOG OF BORING NO. 3

OWNER: IOWA STATE UNIVERSITY

ARCHITECT/ENGINEER:

SITE: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG	DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY	SAMPLES: SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH PSF
	Sandy lean CLAY, trace gravel, dark gray, very stiff				HS					
			CL	9	SS	18"	15	15.1		
		40			HS					
	–color chang to dark olive gray, with gravel and becomes hard @ 42'									
			CL	10	SS	16"	23	9.1		
		45			HS					
	47.0 ▼ 907.0									
	SILT, trace sand, dark gray, hard ▽		ML	11	SS	17"	36	10.2		
	50.0 904.0	50								
	Bottom of Boring									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS			
WL	▽ 49'	WD ▼ None	24 hrs.
WL	GEOTECHNICAL INVESTIGATIONS 48	AB	
WL			

TEAM Services, Inc.

BORING STARTED	6-5-07		
BORING COMPLETED	6-5-07 02 32 00		
RIG	Rig 112	FOREMAN 24	MG
APPROVED	RED	JOB #	1-2065

IOWA STATE UNIVERSITY
CHEMISTRY FACILITIES
WEST UTILITY CORRIDOR

ISU PROJECT NO. CP1207

LOG OF BORING NO. 4

OWNER	ARCHITECT/ENGINEER
IOWA STATE UNIVERSITY	
SITE	PROJECT
Ames, Iowa	Iowa State University Chemistry Building

Approx. Surface Elev.: 954.5 ft.

Depth (ft)	Description	Elev. (ft)
0.3	Asphalt	954.3
1.0	Concrete	953.5
3.0	Fill–Lean CLAY, with sand, trace gravel, olive brown, very dark gray	951.5
5.0	Lean CLAY, trace sand and organic matter, very dark brown, stiff (buried topsoil)	949.5
	Sandy lean CLAY, trace gravel and ferrous staining, olive gray, stiff –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'	

	SAMPLES					TESTS		
USCS SYMBOL	NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF	
CL	1	ST	9"		17.3		2000*	
CL	2	ST	8"		24.6	90	3000*	
		HS						
CL	3	ST	13"		16.2	105	2477 3500*	
CL	4	ST	22"		15.4	116	4500*	
		HS						
CL	5	ST	20"		12.7		9000*	
		HS						
CL	6	SS	15"	19	14.1			
		HS						
CL	7	SS	17"	17	12.1			
		HS						
CL	8	SS	16"	13	13.3		8000*	
		HS						
CL	9	SS	17"	11	14.4		7500*	
		HS						
CL	10	SS	13"	14	14.7		8000*	

Depth (ft.): 5, 10, 15, 20, 25, 30, 35

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS				
WL	None	WD	None	24 hrs.
WL	None	AB		
WL				

TEAM Services, Inc.

GEOTECHNICAL INVESTIGATIONS

02 32 00 25

BORING STARTED	6-7-07		
BORING COMPLETED	6-7-07		
RIG	Rig 112	FOREMAN	MG
APPROVED	RED	JOB #	1-2065

LOG OF BORING NO. 4

OWNER: IOWA STATE UNIVERSITY

ARCHITECT/ENGINEER:

SITE: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG	DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY	SAMPLES: SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH PSF
	Sandy lean CLAY, trace gravel, dark gray, very stiff				HS					
			CL	11	SS	14"	13	13.3		
		40			HS					
			CL	12	SS	17"	17	15.4		
		45			HS					
			CL	13	SS	19"	21	14.8		
	50.0 — 904.5	50								
	Bottom of Boring									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS				TEAM Services, Inc.				
WL	None	WD	None 24 hrs.		BORING STARTED	6-7-07		
WL	None	AB			BORING COMPLETED	6-7-07		
WL					RIG	Rig 112	FOREMAN	MG
					APPROVED	RED	JOB #	1-2065

GEOTECHNICAL INVESTIGATIONS

02 32 00 26

LOG OF BORING NO. 5

OWNER: IOWA STATE UNIVERSITY

ARCHITECT/ENGINEER:

SITE: Ames, Iowa

PROJECT: Iowa State University Chemistry Building

Approx. Surface Elev.: 954.5 ft.

Depth to base (ft)	Description	Elev. (ft)
0.3	Asphalt	954.3
1.0	Crushed limestone	953.5
2.0	Sandy lean CLAY, trace gravel, very dark gray (buried topsoil)	952.5
	Sandy lean CLAY, trace gravel and ferrous staining, grayish brown, stiff	
	–color change to olive brown and becomes very stiff @ 7'	
	–ferrous ends and color change to dark gray @ 17'	

DEPTH (ft.)	USCS SYMBOL	NUMBER	TYPE	RECOVERY	SPT - N BLOWS / FT.	MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH PSF
	CL	1	AS			16.5		
			HS					
	CL	2	ST	17"		15.5	113	2500*
5			HS					
	CL	3	ST	19"		13.4	122	8000*
10			HS					
	CL	4	SS	14"	15	15.5		
15			HS					
	CL	5	SS	13"	18	13.0		
20			HS					
	CL	6	SS	3"	21	12.6		
25			HS					
	CL	7	SS	17"	14	15.1		8000*
30			HS					
	CL	8	SS	16"	13	14.4		8000*
35								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS		
WL	None	WD None 24 hrs.
WL	None	AB
WL		

TEAM Services, Inc.

GEOTECHNICAL INVESTIGATIONS

BORING STARTED	6-7-07		
BORING COMPLETED	6-7-07		
RIG	Rig 112	FOREMAN	MC
APPROVED	RED	JOB #	1-2065

02 32 00 27

IOWA STATE UNIVERSITY
CHEMISTRY FACILITIES
WEST UTILITY CORRIDOR

ISU PROJECT NO. CP1207

LOG OF BORING NO. 5

Page 2 of 2

OWNER	ARCHITECT/ENGINEER
IOWA STATE UNIVERSITY	
SITE: Ames, Iowa	PROJECT: Iowa State University Chemistry Building

GRAPHIC LOG	DESCRIPTION	DEPTH (ft.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY	SAMPLES: SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH PSF
	Sandy lean CLAY, trace gravel, dark gray, very stiff				HS					
			CL	9	SS	21"	19	13.5		
		40			HS					
			CL	10	SS	19"	16	15.5		
		45			HS					
			CL	11	SS	18"	15	15.1		
	50.0 Bottom of Boring 904.5	50								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL AND ROCK TYPES: IN-SITU, THE TRANSITION MAY BE GRADUAL.

Calibrated Hand Penetrometer*

WATER LEVEL OBSERVATIONS			TEAM Services, Inc.		
WL	None	WD None	24 hrs.	BORING STARTED	6-7-07
WL	None	AB		BORING COMPLETED	6-7-07
WL				RIG: Rig 112	FOREMAN: MG
				APPROVED: RED	JOB #: 1-2065

GEOTECHNICAL INVESTIGATIONS 02 32 00 - 28