The National Utility Contractors Association's

TRENCHLESS CONSTRUCTION AND NEW INSTALLATION METHODS

5th Edition

NUCA TRENCHLESS

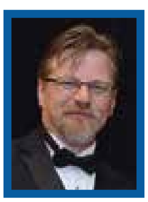
Produced by the NUCA Trenchless Technology Committee

PREFACE

NUCA's Trenchless Construction and New Installation Methods 5th Edition is a manual written by experts in the industry. A manual written from the contractors' and manufacturers' perspective with the goal to educate and guide others in choosing the best method to complete the project. The manual has been four years in the making, with countless hours put in by the contributing sub-committee authors and an initial re-write of the original manual by the Trenchless Technology Center at Louisiana Tech University.

Trenchless new installation methods have been a growing segment of the utility construction market for the past 20 years. Advancements and applications have driven the need to develop the 5th Edition. The primary intention of the manual is to help utility owners and engineers specify the right trenchless method that meets their specific need.

A special thank you goes to Jeff Rumer, NUCA Trenchless Technology Committee Chair and Tom Olson, Committee Co-Chair. Jeff's passion for trenchless construction was the driving force behind the development of the manual. Tom's diligent review and detailed compilation of Chapter 13 covering the legal rights and obligations in trenchless construction helped bring the manual to fruition.



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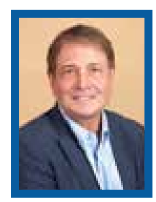

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Courtesy of Laney Directional Drilling

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CHAPTER 1 INTRODUCTION

The trenchless technology industry began to form in the mid-1980s as a result of the first No-Dig conference held in Great Britain. Trenchless technology (TT) can be defined as the family of equipment, materials and methods that permit the installation of new underground pipelines and cables and the renewal of existing underground pipelines and cables with minimum disruption and/or destruction to society and the environment. However, many trenchless methods existed long before the industry started to become organized. For example, pipe jacking can be traced back to about 1900 in the U.S. and auger boring to about the mid-1930s.

As the utilization of these methods expanded, it became apparent that industry guidelines were needed to assist designers and contractors. In response to this industry need, the National Utility Contractors Association (NUCA) formed the Auger Boring (AB) and Pipe Jacking (PJ) Committee during the late 1970s. This committee published the AB and PJ manual in 1981 with a revision in 1986.

Five individuals representing NUCA, WEF, NASSCO, TTC and AWWA were instrumental with organizing NASTT and the first No-Dig Conference held in the United States in the early 1990s. During this time, the NUCA AB&PJ Committee transitioned to the Trenchless Technology Committee to recognize the comprehensive family of trenchless methods used for new installation and renewal of pipelines and cables. The NUCA

TT Committee has continued to provide industry leadership through additional manuals and workshops.

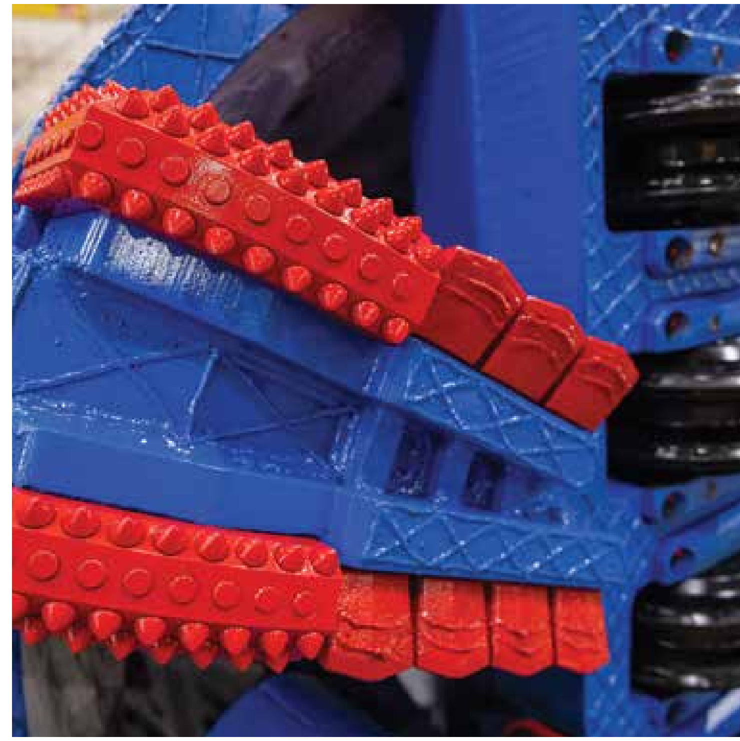
The trenchless industry continues to expand at a rapid rate. It is a challenge for designers and contractors to keep informed of the industry's dynamic nature. Even though much has been accomplished for the trenchless industry through many organizations, workshops, seminars, conferences, manuals, etc., the industry is still experiencing unacceptable high rates of project failures, claims, litigation, delays, and cost overruns.

The NUCA TT Committee became concerned that there is a lack of information and understanding of what is needed to produce successful trenchless projects from the contractors' perspective. This manual has been prepared by the Trenchless Technology Center (TTC) at Louisiana Tech University (LA Tech) under the supervision of NUCA's TT Committee to fill this gap.

In previous NUCA TT manuals, both new installation methods and renewal methods were presented. The TT Committee decided that the focus of this manual needed to be just on the new trenchless installation methods but does include pipe bursting since it results in the installation of a new pipe. There are numerous methods used to classify the various trenchless methods. The following classification of methods will be used for this manual with the method definitions being focused on the process and not diameter size.

- Methods that require personnel inside the pipe during installation:
    - o **Pipe Jacking (PJ)** – A system of using hydraulic jacking from a drive shaft to directly install pipes behind a shield machine so that they form a continuous string in the ground. (ASCE 36-15)
    - o **Utility Tunneling (UT)** – A construction method for excavating an opening beneath the ground without continuous disturbance of the ground surface. The excavation is of sufficient diameter to permit personnel access and to allow excavation, transport of spoils, and erection of a ground support system. (ASCE 36-15)

- Methods that do not require personnel inside the pipe during installation:
    - o **Auger Boring (AB)** – A technique for forming a bore from a jacking or drive shaft to a receiving shaft by means of a rotating auger with cutting tools. The casings are jacked forward sequentially in a cyclic process while the auger is turned. Spoils are removed back to the drive shaft by helically wound auger flights rotating in the steel casing. The equipment may have limited guidance and steering capability. (ASCE 36-15)



Courtesy of Akkerman

    - o **Microtunneling (MT)** – A trenchless construction method for installing pipelines. The North American definition of microtunneling describes a method but does not impose size limitations on that method; therefore, a tunnel may be considered a microtunnel if all of the following are used during construction: (i) Remote control, (ii) Accurate guidance, (iii) Pipe jacking, and (iv) Continuous face support.[1]
    - o **Pipe Ramming (PR)** – A trenchless installation method whereby a percussive hammer is attached, via an adapter, to an open-end casing, which is then driven through the ground. To create an open casing, the spoils within the casing are removed after the drive is completed, or periodically during the drive. (ASCE 36-15)
    - o **Horizontal Directional Drilling (HDD)** – A surface-launched trenchless technology for the installation of pipes, conduits, and cables. HDD creates a pilot bore along the design pathway and reams the pilot bore in one or more passes to a diameter suitable for the product, which is pulled into the prepared bore in the final step of the process.
    - o **Soil Compaction (SC) Methods** – This method consists of several techniques for forming a bore hole by in-situ soil displacement using a compacting device. The compacting device is forced through the soil, typically from a drive shaft to a reception shaft, by

applying a static thrust force, rotary force and/or dynamic impact energy. The soil along the alignment is simply displaced rather than being removed. This is a two-stage process.

- o **Direct Pipe® (DP)** – In this method the machine is deployed at the front end of the pipeline and pushed into the ground together with the pipe. The excavated material removed by the slowly rotating cutterhead at the tunnel face is mixed inside the machine with the drilling fluid (bentonite suspension) and then fed through a discharge line through the entire pipeline to the separation plant on the surface (through a separate discharge line). After treatment, the material is pumped back into the circuit via a feed line. The drilling fluid thus not only discharges the excavated material but also supports the tunnel face. Comparable with the jacking frame used for standard pipe jacking with short concrete pipes, the Pipe Thruster acts as the thrust unit in Direct Pipe®. It hydraulically grips the prefabricated and outlaid pipeline and pushes it into the ground in strokes of 17 ft (5 m). Coated product pipes can be pushed directly. The required borehole is excavated by the Direct Pipe® machine, which is based on an AVN micromachine. Thruster can deliver average thrust force of 15 t to maximum 28 t to the Direct Pipe® machine above it.[2]
- o **Pilot Tube (PT)** – The PT method, also known as the Guided Boring Method (GBM), is a hybrid technique that combines the characteristics of other trenchless techniques, such as HDD, AB, and MT to accurately install a pipe. The PT method first appeared in Germany in the 1980s for the installation of sewer laterals with a diameter of 4-6 in. (102-52 mm). It was used in the United States in the late 1990s for the installation of pipes with a diameter of 4-12 in. (100-305 mm). The PT technology has improved and is able to install pipes up to 48 in. (1.2 m) in OD with drive length in the 300 ft (91 m) range.[3]
- o **Pipe Bursting (PB)** – Pipe bursting is a well-established trenchless method that is widely used for the replacement of deteriorated pipes with a new pipe of the same or larger diameter. Pipe bursting conventionally involves the insertion of a cone shaped bursting head into an old pipe. The base of the cone is larger than the inside diameter of the old pipe and slightly larger than the outside diameter of the new pipe to reduce friction and to provide space for maneuvering the pipe. The back end of the bursting head is connected to the new pipe and the front end is attached to a cable or pulling rod. The new pipe and bursting head are launched from the insertion shaft and the cable or pulling rod is pulled from the pulling shaft. There are three basic methods of pipe bursting: pneumatic, hydraulic, and static pull.[4]

The scope of this manual is not intended to be an exhaustive detailed analysis of each of the methods defined above. There exist excellent manuals of practice, best practice guidelines, standards, technical papers, etc. produced by a wide range of professional and trade organizations world-wide. For example, the American Society of Civil Engineers (ASCE) recently published the "Standard Design and Construction Guidelines for Microtunneling."[1] This standard was developed by a consensus standards development process that has been accredited by the American National Standards Institute (ANSI). Most of the method definitions were extracted from this document to ensure consistency and reduce the likelihood of confusion.

The intent of this manual is to enhance the body of knowledge which already exists by providing input from the utility contractors which work every day utilizing these methods to build our underground infrastructure with the overall objective to produce safe and successful projects. Therefore, the main focus is on constructability. This requires identifying, allocating and managing risks associated with the projects. Underground construction is characterized with risk, and trenchless projects can be even more risky since work is being accomplished without direct vision of the installation process. It is not acceptable to simply pass the risk to others without regard of which parties are best qualified and experienced to manage the risks. The owner of the project needs to understand the risk and ensure that the party best trained and knowledgeable has been assigned responsibility to manage the risk.

A major way to understand and reduce the risk of trenchless projects is to provide accurate and complete information on the subsurface conditions along the project alignment. The major components of understanding the subsurface conditions are the location of existing underground utilities and other obstacles and the geological conditions. Chapter 3 is dedicated to discussing geotechnical considerations.

NUCA has been a national leader in supporting regulations related to the One-Call laws and the accomplishments of the Common Ground Alliance. Encountering unknown and/or mismarked existing underground utilities is a major concern that utility contractors deal with every day. It has been reported that over 379,000 utility hits occurred in the U.S. during 2017, costing the industry billions of dollars and numerous loss of lives. This is up from about 340,000 hits in 2014. This trend is moving in the wrong direction. It has been estimated that about 70% of project delays and cost overruns are due to conflicts with existing underground utilities. The best insurance that can be obtained for the project is being informed.

NUCA with ASCE UESI (Utility Engineering and Surveying Institute) are sponsoring partners of the Utility Investigation School (UIS) conducted by the Trenchless Technology Center (TTC) at Louisiana Tech University. UIS is a five-day school based on the principles and practices of the ASCE 38-02 Standard Guideline on the Location and Depiction of Underground Utilities. It is significant that the nation's largest utility contractors' organization and the nation's largest civil engineering organization are partnering sponsors of the TTC UIS.

ASCE 38-02 provides the guidelines for the Subsurface Utility Engineering (SUE) professional discipline. This document clearly establishes the industry's standard of care, which should be followed during the design process. It describes the following four Quality Levels (QL):

- **QL D** – Obtain as much information as possible from existing records, maps, documents, etc., and depict on a map/drawing.
- **QL C** – Visit the proposed jobsite to identify any features that can verify the accuracy of the QL D drawing. This would include valves, meters, fire hydrants, manholes, cleanouts, etc.
- **QL B** – Utilize appropriate geophysical techniques to better define the location accuracy of underground utilities.
- **QL A** – Requires data to be properly verified via intrusive efforts such as 'potholing' under the supervision of a licensed PE with the final utility map bearing the seal of the PE.

Owners need to be aware that independent studies have shown that when these guidelines are followed for every $1.00 spent results in a $4-7.00 savings. In addition to the cost savings provided to the project owners, project safety is enhanced, risk is reduced, and likelihood of utility hits are reduced, including catastrophic events such as a gas explosion.

The remainder of this manual will provide more detailed information on the trenchless methods utilized for the installation of new pipelines and cables.

REFERENCES

1. *American Society of Civil Engineers (ASCE/CI 36-15). Standard Design and Construction Guidelines for Microtunneling. 2015.*
2. *Direct Pipe® is a registered trademark of Herrenknecht AG.*
3. *ASCE Manual and Reports on Engineering Practice No. 133. Pilot Tube and Other Guided Boring Methods. American Society of Civil Engineers, 2017.*
4. *Plastics Pipe Institute® Handbook of PE Pipe. Second Edition. Chapter 16, Pipe Bursting.*

Courtesy of McLaughlin

ABBREVIATIONS:

AB: Auger Boring
ANSI: American National Standards Institute
AWWA: American Water Works Association
DP: Direct Pipe®
HDD: Horizontal Directional Drilling
MT: Microtunneling
NASSCO: National Association of Sewer Service Companies
NUCA: National Utility Contractors Association
PB: Pipe Bursting
PJ: Pipe Jacking
PR: Pipe Ramming
PT: Pilot Tube
SC: Soil Compaction
SUE: Subsurface Utility Engineering
TT: Trenchless Technology
TTC: Trenchless Technology Center
UESI: Utility Engineering and Surveying Institute
UIS: Utility Investigation School
UT: Utility Tunneling
WEF: Water Environment Federation

CHAPTER 2
TRENCHLESS TECHNOLOGY:
AN OVERVIEW OF THE NEW INSTALLATION METHODS

The trenchless methods which this manual will focus on were defined in Chapter 1. Chapter 2 will move forward and provide a comparative analysis between the various methods. The objective is to provide some basic parameters to assist the user with determining which methods are best suited for specific applications. As seen from Chapter 1, there are a broad family of methods for installing pipelines and cables with minimum disruption to the environment and society. Owners, designers, and contractors need to be knowledgeable about potential methods which can be utilized for their project. This chapter will provide general guidelines for method selection. This chapter will provide a decision-making process to assist the user in selecting appropriate methods based on various site and project conditions.

For this manual, the term “Owner” will be used to identify the organization which owns the pipeline or cable system. These pipelines or cables may be privately, publicly, or cooperatively owned. The function of the pipelines, cables, facilities, or system may be for private use, for producing, transmitting, or distributing communications, cable tele-

Photo courtesy of Midwest Mole

vision, power, electricity, light, heat, gas, oil, crude products, water (drinking, wastewater, and storm), and steam to the public, or for transporting sanitary sewage for treatment and disposal. Near surface underground space is increasingly congested, especially in urban environments. It has been estimated that over 150,000 miles of new pipelines and cables are installed every year increasing this congestion. Owners of underground infrastructure must know what assets (pipelines and cables) they have and where these assets are located. Typically, this is done through the aid of GIS mapping systems.

Figure 2.1 is a basic classification system for trenchless methods used to install a wide range of types and sizes of new pipelines and cables. The origins of this classification system are rooted in the Purdue University-Indiana Department of Transportation (DOT) research project conducted in 1987/88.[3] Modified versions of this classification system have been used in a technical paper for the American Society of Civil Engineers (ASCE) Journal of Construction Engineering and Management[7] and for a National Utility Contractors Association manual.[8] This system has been accepted as a standard by both design and construction professionals. Other classification systems and detailed descriptions of the techniques can be found in the literature.[9-11] The classification system depicted in Figure 2.1 is based on a key principle of operation (i.e., whether the process requires people to be working inside the conduit as it is being installed underground.) If the process does require people to be inside the conduit, it is classified as either a utility tunneling (UT) operation or a pipe jacking (PJ) operation. If the process does not require people to be inside the conduit, it is classified as a horizontal earth boring (HEB) operation.

The definitions for the methods provided in Figure 2.1 were provided in Chapter 1. More detailed information on these methods will be provided in Chapters 4 through 9.

Table 2.1 on page 14 provides some of the basic characteristics associated with the various trenchless alternatives. These, too, are described in more detail in Chapters 4 through 9.

FACTORS AFFECTING THE SELECTION OF TRENCHLESS TECHNOLOGY ALTERNATIVES

During the planning, design, and installation phases of a trenchless technology (TT) project, it is important for decision makers to be knowledgeable about the capability and limitations of TT alternatives. Constructability issues need to be addressed early in the project planning and design phases. The alignment, depth of cover, and profile of the pipeline or cable may vary depending on the installation technique. It is important that the design provide the contractor with adequate surface and subsurface information so that compatible methods, materials, and equipment can be selected. Adequate information can only be provided if planning and design professionals are sensitive to and knowledgeable about constructability issues.

Table 2.2 (page 16) identifies significant factors that must be evaluated during the selection of TT alternatives. Many of these factors are not significant for a traditional open-cutting (trench) technique, but are extremely important for a TT project. For example, a TT project in wet sand versus clay could require a different type of machine, whereas with a trench excavation, the choice of machine might not be as important. Figure 2.1 provides

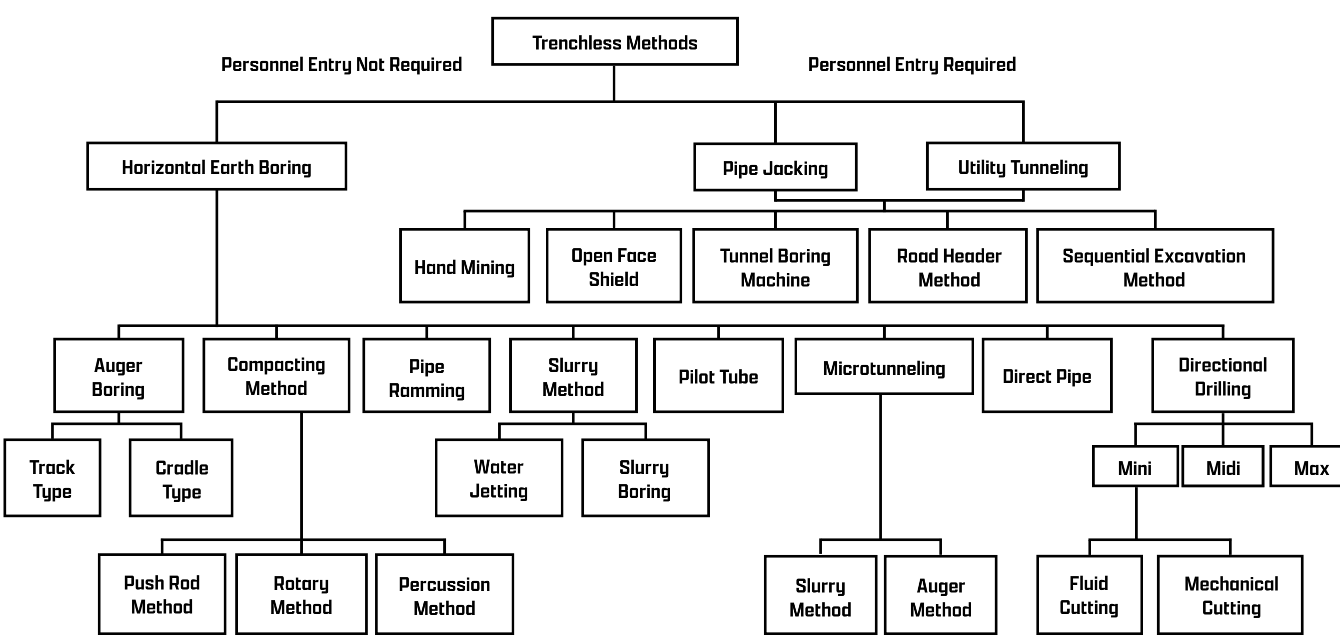


Figure 2.1 Classification for trenchless methods.

Table 2.1 Characteristics of trenchless construction methods.

Type	Pipe/Casing Installation Mode	Suitable Pipe/casing	Soil Excavation Mode	Soil Removal Mode
Auger Boring (AB)	Jacking	Steel Casing Pipe	Mechanical	Augering
Slurry Boring (SB)	Pulling/Pushing	All Types	Mechanical and Hydraulic	Hydraulic, Mechanical Reaming and Compaction
Microtunneling (MT)	Pipe Jacking	Steel Casing Pipe, Reinforced Concrete Pipe, Glass-Fiber Reinforced Plastic Pipe, Polymer Concrete Pipe, Vitrified Clay Pipe, Ductile Iron Pipe	Mechanical	Augering or Hydraulic (Slurry)
Horizontal Directional Drilling (HDD)	Pulling	Steel Casing Pipe, Polyvinyl Chloride Pipe, High Density Polyethylene Pipe	Mechanical and Hydraulic	Hydraulic, Mechanical Reaming and Compaction
Pipe Ramming (PR)	Hammering/Driving	Steel Casing Pipe	Mechanical	Augering, Hydraulic, Compressed Air, or Compaction
Soil Compaction (SC)	Pulling	Steel Casing Pipe, Polyvinyl Chloride Pipe, High Density Polyethylene Pipe	Pushing	Displacement (in-situ)
Pipe Jacking (PJ)	Jacking	Steel Casing Pipe, Reinforced Concrete Pipe, Glass-Fiber Reinforced Plastic Pipe	Manual or Mechanical	Augers, Conveyors, Manual Carts, Power Carts, or Hydraulic
Utility Tunneling (UT)	Lining	Steel or Concrete Liner Plates, Ribs w/ Wood Lagging, Wood Box	Manual or Mechanical	Augers, Conveyors, Manual Carts, Power Carts, or Hydraulic
Direct Pipe® (DP)	Pushing	Steel Pipe	Mechanical	Hydraulic (Slurry)
Pilot Tube (PT)	Jacking	Vitrified Clay Pipe (VCP), Reinforced Concrete Pipe (RCP), Polymer Concrete Pipe (PCP), Steel Casing Pipe	Mechanical	Augering and Reaming

a flow diagram of a seven-step process for selecting a TT alternative suitable for a particular application. A more detailed discussion of each of these steps follows. It is essential that construction input be provided during the selection of alternatives. For example, at what point should a steerable system be provided? A 15-m (50-ft) drive under a roadway in homogeneous soil conditions for non-gravity sewer applications may need no steering, but steering may be required for a 90-m (300-ft) crossing.

STEP 1: Develop an Understanding of Trenchless Technology Alternatives

Individuals involved in planning, designing, and constructing a TT project should have access to TT expertise. Keeping abreast of TT, which is constantly evolving, is a continuous learning process. Numerous trade and professional organizations provide a wide range of training and professional development. This manual has been developed by the NUCA TT Committee to provide the trenchless contractors' perspective on factors which should be considered to produce a successful project and minimize risk.

TT expertise requires an understanding of the benefits provided by TT and the performance characteristics of the techniques (i.e., what methods are appropriate for a given set of conditions). Most of this manual addresses construction issues; however, the following itemizes some social and environmental benefits that could be included in the selection process.[12]

Public Impact

Traffic – If minimizing disruption to local traffic is important, use of TT often results in minimum or no interference with traffic. Because traffic impact is a major concern for many projects, shaft location should be considered carefully during the design phase to identify locations where traffic impact will be minimized. For example, if a drive shaft is located in a main intersection, it could have a significant impact on traffic because it may be in use for several months.

Pavement – Pavement cutting, followed by the usual quality of restoration, may significantly reduce pavement life. This results in more frequent pavement repairs, additional traffic impacts, and increased maintenance costs.[14] A study conducted for the city of Burlington, Vermont, found "streets without utility cut patching have a life of 18.5 years while streets with utility cut patching have a life of 10.9 years."[15] The use of TT alternatives may result in no or minimal pavement cuts, primarily for shafts. However, field construction problems could lead to major pavement cuts under emergency conditions. Proper site investigation, planning, design, selection, and use· of compatible construction methods, materials, and equipment can minimize these types of problems.

Commercial – If there are businesses along or near the work area that will suffer significant losses because of lane or road closures, use of TT could minimize commercial impact.

Residential – Reducing delays, inconvenience and congestion for area residents, access for emergency services, and minimized disturbance to yards, driveways or sidewalks provide additional justification for TT. Residential customers of River Oaks, in Houston, Texas, thought these factors were extremely important, and because of their demands, 7,000 m (20,000 ft) of gravity sewer lines ranging from 250 mm (10 in.) to 530 mm (21 in.) in diameter were installed by microtunneling (MT) in 1987.

Government Income – By minimizing business disruption, local governments can prevent decreased sales tax revenue, parking meter revenue, and other sources of government income using TT.

Accidents – Often, when TT is used, the number of workers is reduced, but the workers are more specialized. Specialized workers can reduce the risk of accidents to project personnel. Also, when TT is used to minimize traffic disruption, the risk of accidents involving the public may be reduced.

Environmental Impacts

Noise – During TT projects, most major activity takes place underground. The problem of surface noise can be isolated to access shafts where it can be managed to acceptable levels (e.g., by use of hospital-type generators).

Air Pollution – Fine soil particle dispersal in the air because of soil excavation, handling, and backfilling can be an important issue for projects near hospitals or other sensitive areas. With proper planning and design, the use of TT can limit the potential for airborne particles by minimizing the amount of excavation. The location of the excavation can be controlled by locating the shafts as far away from sensitive areas as possible.

Soil Disposal – The handling and disposal of hazardous and contaminated materials is a serious and expensive issue, requiring special equipment and specially trained personnel. If the project will involve disposal of excavated asphalt concrete or chemically contaminated soil or soil contaminated with sanitary sewer exfiltration, the use of TT alternatives can minimize the volume of contaminated soil and groundwater that needs to be disposed of or treated. For example, when 915 m (3,000 ft) of relief sewer 1,372 mm (54 in.) was installed parallel to the Nimitz Highway for the city and county of Honolulu, Hawaii, the project faced extensive soil contaminated with oil. The contractor stated that with microtunneling, only soil equal to the volume of pipe had to be dug out and that only the slurry had to be treated.[18] Shaft location and design are important in the contaminated area. Often it is possible to keep shafts out of contaminated zones. By using watertight shaft construction techniques, the risks associated with the removal, disposal, treatment, and migration of contaminated materials can be minimized.

Surface Defacement – With use of TT impact on grass, trees, wetlands, and other environmental components can be significantly minimized.

STEP 2:

Develop an Understanding of the Trenchless Technology Project Ground Conditions

Individuals involved in the planning, design, and installation of a TT project must develop a clear understanding of the characteristics of a particular project. These individuals must be able to identify which aboveground and underground factors will affect the complexity of the project and will be significant in the selection of the proper TT equipment. An understanding of these factors requires that adequate ground data be provided. The importance of adequate ground information cannot be overemphasized. After all, the TT contractor will be navigating through ground without seeing the excavation and conduit installation process.

The contractor must be informed of anticipated ground conditions to select the proper equipment and design the proper installation process. NUCA and ASCE both recommend the use of Geotechnical Design Summary Reports (GDSRs).[19] Nothing can eliminate the risk of encountering differing subsurface conditions. However, the potential for costly disputes and litigation over what constitutes differing conditions can be greatly reduced, if not eliminated, with well-defined geotechnical baselines. The overall risk associated with an underground project is inversely proportional to the extent of subsurface investigation. The GDSR sets forth the geotechnical conditions anticipated by the designer and establishes clear and concise baselines for identification of differing site conditions. The GDSR should be incorporated as part of the contract, with no exculpatory language disclaiming responsibility for accuracy or completeness.

Ground information needed for construction may differ from what is needed for design. The civil engineering design community typically uses the Unified Soil Classification System (USCS), which was developed for the U.S. Corps of Engineers by Casagrande in 1953 and adopted by the American Society for Testing and Materials (ASTM) in 1969. The owner must provide adequate ground information to the contractor. This information should not be limited to USCS data. The contractor is more concerned with the behavior of the ground; therefore, ground information should be communicated to the TT contractor, with terms and concepts clearly ex-

Table 2.2 Factors affecting the selection and use of trenchless technology (TT) alternatives.

Factors	Description
Diameter of Drive	Need to identify which methods are suitable to install the pipe required for the drive from project scope. As the diameter increases, the complexity and risks associated with the project also increase. Some methods are unsuitable for some diameters.
Length of Drive	Need to identify which methods are suitable for installing the pipe for the drive lengths required by the project scope. As the length increases, the complexity and risks associated with the project also increase. Length of drive may rule out certain methods or result in cost penalties for mobilization for short distances.
Existing Underground Utilities	Need to determine location of all existing underground utilities and underground structures so that the likelihood of obstruction or damage can be addressed for each TT alternative. Actions needed to avoid obstructions should be identified for each prospective method.
Existing Above Ground Structures	The likelihood of ground movement caused by the proposed TT alternatives should be evaluated. A possibility of heaving the roadway or causing ground subsidence should be evaluated. The parameters to be monitored to ensure minimum effect on adjoining structures must be identified.
Obstructions	The likelihood of encountering obstructions (either naturally occurring or manmade) should be evaluated. The proposed equipment must be able to handle the anticipated obstructions. For example, some techniques might permit steering around or crushing obstacles up to a certain size.
Casing	Identify whether a casing pipe is required. Identify whether the product pipe can be installed directly. If a casing pipe is required, identify whether the annular space between the product pipe and casing pipe need to be filled, and with what materials. Identify whether the casing pipe needs internal and/or external coatings? Identify how far the casing should extend beyond the pavement edge.
Ground Conditions	Need to accurately determine the actual ground and ground water table conditions at the site, as well as whether the proposed TT equipment is compatible with the anticipated conditions. Identify whether the equipment can operate properly under the water table. Identify whether the equipment can function in unstable ground conditions or whether the ground conditions need to be stabilized prior to the trenchless process being employed. If stabilization is needed, identify how this will be accomplished and the feasibility of stabilization methods (dewatering, etc.) Identify whether contamination will be an issue for the ground/groundwater. Identify the likelihood of ground heaving or settlement. Need to establish allowable limits for ground movement and need to determine how ground movement will be measured. What about ground behavior? With all forms of tunneling, ground behavior is the key to the choice of means and methods.
Drive/Reception Shafts	Need to make sure that adequate space is available at the project site to provide the required space for the shafts. The working room available may limit the length of pipe segments that can be handled. For example, using 12 m (40 ft) steel pipe segments will minimize field welding time and may be desirable from a construction perspective, but may not be achievable due to site constraints. These constraints need to be identified early in the process.
Tolerances	Need to determine alignment and grade tolerances desired for the installation. Typically, the tighter the tolerance, the higher the cost of installation will be. Need to determine how tolerance accuracy will be measured. This normally starts with whether you are constructing a pressure pipeline where tolerances can be very wide and still function or a gravity flow line where tolerances can be critical to the function of the pipeline.
Steerability	Identify the level of sophistication needed to steer and track the leading edge of the cutting head. Identify limits needed on steering corrections to prevent overstressing the drill stem or pipe.
Bulkheads	Bulkheads are used to provide end seals between the casing and product pipe. Identify whether your project requires bulkheads. If bulkheads are required, identify what they should be made of.
Materials	Identify what materials the casing and/or product pipe should be (i.e., Steel, RCP, PVC, GFRP, HDPE, etc.) and joint requirements. Selection must be based on use, environmental conditions, and compatibility with the trenchless method.
Ventilation/ Lighting	Identify what conditions will require ventilation and/or lighting and what level of ventilation and lighting will be adequate.
Abandonment	Identify what conditions will require work stoppage and abandonment. Identify abandonment procedures.
Measurement/ Payment	Identify conditions of payment, including who will measure work and how work will be measured.
Submittals	Identify what information the contractor is required to supply, and the process and people involved in submittal review. Identify the construction risks and the party responsible for each construction risk.

plaining characteristics and anticipated behavior. The tunneling industry uses ground classification terms that are different from those a TT contractor typically uses.[5, 20] The tunneling contractor is concerned with the behavior of the ground at the tunnel face. The terms commonly used in the tunneling industry are[1] running ground,[2] flowing ground,[3] raveling ground,[4] squeezing ground, and[5] swelling ground.[21, 22] TT contractors are more familiar with ground classification terms such as[1] wet running sand;[2] wet stable sand;[3] dry sand;[4] dry clay;[5] wet clay;[6] soil with small gravel;[7] soil with large gravel, cobbles, and boulders;[8] hard pan;[9] soft or hard rock; and[10] fill and mixed face conditions.[4,5] Numerous references provide detailed information on the criteria for providing adequate subsurface information. [10,11,21-24] This is explained in more detail in Chapter 3 – Geotechnical Considerations.

STEP 3:

Determine the Factors Affecting the Selection of Trenchless Technology Alternatives

After Steps 1 and 2 are accomplished, individuals should have a clear understanding of the capability and limitations of available TT options, as well as a clear understanding of the nature of the project. The objective in Step 3 is to direct more attention to the appropriate method for a particular project. Table 2 contains a list of factors that can affect the selection of TT alternatives. Once the conditions of a specific site are known, these factors can be evaluated. For example, casing under roadways is not always required. Therefore, the need for casing needs to be determined because it could have a significant impact on the complexity of the project. In general, state DOT regulations require encasement of mains that cross under pavement. However, DOTs are not unified on this policy. About one-third of the states require encasement of all types of line crossings, whereas two-thirds allow crossings without encasement under certain conditions.[1]

Reasons for requiring encasement include the following:

- To avoid roadway excavation for repair or replacement of the pipeline,
- To ensure structural integrity of the roadbed and pipeline, and
- To detect and remove leaking fluids and gases from the vicinity of the pipeline with proper venting.

Following are reasons for not requiring encasements:

- It is more difficult to protect the pipeline from corrosion, and
- Procedures have been developed that ensure adequate wall strength in the pipeline to handle anticipated stresses.

STEP 4:

Evaluate the Effectiveness and Design of Trenchless Technology Alternatives

After determining which factors apply to a specific project, each TT alternative should be evaluated to determine compatibility and to identify any special conditions. For example, installing a 600-mm (24-in.) steel casing that is 16 m (50 ft) in length under a roadway in firm silty sand may not require the use of locating equipment and a steering head for the leading end of the casing. However, if the crossing is 90 m (300 ft) in length, the locating and steering capability should be specified. These types of technology selection alternatives should not be left up to the contractor. Otherwise, the knowledgeable and prudent contractor will bid knowing that it is important to know where the end of the steel is at all times, but his or her bid will be higher than the contractors who elects to risk public safety to obtain a lower bid. If left to chance, the probability of field emergencies, delays, and extra costs during construction will be high.

STEP 5:

Determine the Cost of Trenchless Technology Alternatives

After the appropriate TT alternatives have been determined based on technical capabilities, the cost-effectiveness of the alternatives should be analyzed. It is important to evaluate the total and life-cycle costs of a utility project. The total project construction cost is the sum of all real costs incurred as the direct or indirect result of the project. The life-cycle cost takes into consideration the total construction cost and the operation and maintenance cost incurred during the life of the project.

It is a common practice for TT alternatives to be selected based on the lowest direct cost only (i.e., the lowest qualified bidder). This practice is being evaluated by some DOTs. For example, Minnesota DOT has completed a study, Indirect Costs of Utility Placement and Repair Beneath Streets, which emphasizes that “the purpose of an analysis of indirect cost of utility work is to minimize the total economic costs to the community. In a situation where the indirect costs are significant, the method of work which is most cost effective for the community may not be the method with the lowest first cost. Basing the selection of construction technique on both direct and indirect costs does not increase the total cost to the community of the project. Instead, it avoids one segment of the community being unfairly penalized with the imposition of the social costs while another group pays less than the true cost of the work”.[33] For example, pavement cuts decrease the service life of the pavement and increase operation and maintenance costs. These costs may not be reflected in the direct construction cost, but the community will eventually have to pay the extra cost for pavement repairs and replacement.

Typically, the procedure to address social costs is simply for the owner to limit the options that can be proposed by bidders to those that will result in acceptable social costs. This approach avoids the necessity of quantifying social costs, which is very difficult. The following is a list of potential costs for a project. These potential costs should be considered when selecting methods to install utilities beneath roadways. The list was taken from the Minnesota study[33], which is based on the work of the Institute of Science and Technology at the University of Manchester in the UK[34]. Direct cost is the amount of money the owner pays for items that are necessary for, or are a direct result of, accomplishing the project. Direct costs include:

- Permitting and easements
- Site investigation
- Legal and administrative
- Project engineering
- Field construction (original bid amount plus change orders)
  - o Excavation and backfill
  - o Pipe and pipe installation
  - o Pavement reinstatement
  - o Temporary utility service diversions
  - o Traffic diversions and traffic control
  - o Treatment of contaminated soils, slurry, and groundwater.

Indirect cost is the real cost incurred as an indirect result or impact of the construction operation on the normal service and operation of public and private facilities in the vicinity of the project area. Typical items in this category include:

- Traffic
  - o Traffic diversions and delays
  - o Increase in vehicle operating costs
  - o Loss of accessibility and parking spaces
  - o Delays to public transport
- Environmental
  - o Increased noise
  - o Increased air pollution
  - o Increased construction debris
  - o Increased visual intrusion
- Safety
  - o Decreased safety for motorists
  - o Decreased safety for pedestrians
  - o Decreased safety for the workers
- Economic
  - o Loss of trade to local businesses
  - o Damage to other utilities
  - o Damage to street pavement
  - o Increased workload on other government agencies or utilities
- Service Life
  - o Decreased live and dead loads on the pipe, resulting in the likelihood of extended service life
  - o Increased pipe quality because of construction load requirements, resulting in the likelihood of extended service life.

The cost of public transport disruption, noted above, can be further broken down as the cost of the following:

- Additional route mileage
- Delay time
- Shuttle/relief
- Extra walk time
- Information and inspectors' time
- Loss of revenue
- Impact of bus traffic on diversion routes.

STEP 6:

Determine Time Requirements and Other Considerations

After the cost-effective, appropriate TT alternatives have been identified, the next step is to determine a· construction schedule for the project. This schedule should include the following:

- Time to make necessary modifications to the equipment
- Mobilization requirements
- Project set-up time
- Time required to construct shafts
- Time required to install the conduit
- Time required to remove the equipment
- Demobilization.

STEP 7:

Selection of the Appropriate Trenchless Technology Alternative

The appropriate TT alternative is the one that provides the highest probability of success. Following are criteria for a successful project:

- Ensuring the safety of workers, the public, and property
- Providing the required quality end product
- Minimizing the impact to society and the environment
- Meeting time requirements
- Meeting budget requirements.

NOTE: Much of the material of this chapter was developed by one of the primary authors of this NUCA manual for the National Cooperative Highway Research Program (NCHRP), Synthesis of Highway Practice 242, "Trenchless Installations of Conduits Beneath Roadways," Transportation Research Board, National Research Council, National Academy Press, Washington, DC, 1997.

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ABBREVIATIONS:

ASCE: American Society of Civil Engineers
ASTM: American Society for Testing and Materials
DOT: Department of Transportation
GDSRs: Geotechnical Design Summary Reports
GFRP: Glass Fiber Reinforced Polymer
GIS: Geographic Information System
HDD: Horizontal Directional Drilling
HDPE: High-Density Polyethylene
HEB: Horizontal Earth Boring
NUCA: National Utility Contractors Association
PR: Pipe Ramming
PVC: Polyvinyl Chloride
RCP: Reinforced Concrete Pipe
SB: Slurry Boring
USCS: Unified Soil Classification System

CHAPTER 3 GEOTECHICAL CONSIDERATIONS

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INTRODUCTION

This manual has been developed to provide utility contractors recommendations on how to manage the likelihood of construction problems. NUCA supports the American Society of Civil Engineers (ASCE) position on identifying the hazards associated with an underground construction project, and allocating that risk of encountering the hazard and associated project impacts to the most qualified and experienced party to manage the risk. The geotechnical consulting engineer is certainly the most qualified and experienced party to characterize and manage the subsurface conditions. The key factor in the management of subsurface conditions relative to a specific project is to determine subsurface conditions and properties that are relevant to at least one way of constructing a project and providing cost effective operation/maintenance of a specific project. Results may then be communicated and documented to the contractor through the development of geotechnical data reports, geotechnical interpretative reports, and geotechnical baseline reports. The reports provide project-specific ground-to-project interaction and data necessary for the designer and contractor to manage the specific issue. This chapter does not address issues associated with underground utilities.

Selected subsurface construction means and methods have a limited range of subsurface conditions where the equipment is cost effective. Open cut construction has multiple access paths to a problem location and the problem can be observed. Access and observation allow the contractor to select or easily change means and methods based on common observations by all parties. Trenchless means and methods have a limited range of subsurface conditions that may be mined by a specific process, have a single access to the excavation face for both servicing the excavation process and removal of spoils, and often does not expose the issue thus relying on contractor skill to describe the possible problem based on the performance of the excavation equipment.

NUCA supports the generally accepted industry practice that the owner of the project owns the underground conditions. However, this concept is not necessarily legally binding in all states thus each state should be assessed by the Contractor's Council to determine legal risk for a particular project in a specific state.

Subsurface conditions are the product of geologic processes. These processes can cause ground conditions to change significantly over a short distance with respect to a project footprint. Factors such as groundwater level and fluctuation patterns, bedrock location, existence of perched water tables, cobbles, boulders, gravels, etc. are often critical when selecting subsurface construction means and methods. If the subsurface conditions are not properly characterized, the appropriate means and methods for controlling the ground are likely to not be selected or cost effective.

When the professional geotechnical engineer has utilized an appropriate standard of care as recognized by the industry in interpretation of available data, circumstances can still evolve during the construction process where the subsurface characterization as presented to the contractor was not encountered, resulting in a changed condition. Resolving this development can be a challenge for any underground construction project; however, it can be even more challenging for projects utilizing trenchless technologies since the contractor is installing the product without being able to see the excavation process, thus removing the common ability to observe, qualify, and quantify the problem.

Production of an Owner approved geotechnical baseline document developed by a qualified and experienced geotechnical engineer that represents the interpreted subsurface conditions in clear contractor understood language, makes it easier for all parties to agree to a defined baseline condition to assess if conditions are actually different from the anticipated conditions. Additionally, the baseline allows all bidders to base their bid on the same anticipated subsurface conditions. This provides the owner with the lowest cost for constructing the project based on the given characterization. It prevents the contractor from having to increase the bid price to cover risk due to unknown conditions.

Initially, no one knows definitively what will be encountered in the subsurface. The geotechnical engineer is retained typically by the Owner to utilize standard subsurface evaluation and investigative techniques to provide an accurate depiction and statement for the design and construction of the project. The purpose is to define when changed conditions are encountered, and where the owner will be responsible for paying for the additional costs associated with the modifications required to deal with the challenge. Therefore, it is critical that the geotechnical baseline be a fair and accurate statement.

Historically, larger underground construction contract documents include baseline reports as part of the bid package while smaller projects do not include such reports. Noteworthy is the fact that many trenchless projects provide only data with no interpretation, data for reference only, or no data. If any information were provided, disclaimer statements make it clear that the contractor would be responsible for obtaining the geotechnical information needed to select the appropriate means and methods. This is often an unrealistic expectation as there is limited time to obtain the necessary permits and property access and retain appropriate contractors to do the work.

Many versions of clauses requiring contractors to obtain their own data require the bidder to obtain this information prior to having a contract or any means to recover the cost. In addition, obtaining the information and adapting new but appropriate means and methods place design responsibility and risk on the contractor and can relieve Owners and Engineers any design or construction responsibility though they were paid by the Owner to take on that responsibility. Another factor when working without a contract is that a contractor cannot get a permit to work on property owned by others and in fact is a project betterment to the Owner for which the Contractor is not getting paid.

The Spearin Doctrine developed in response to unreasonable contract clauses that shift the constructability warrantee from Owner to Contractor. Basically, the Spearin Doctrine implies that the project offered for bidding by a responsible Owner must be constructable with respect to contract documents. Contractors are responsible for selecting appropriate means and methods based on the information provided in the bid and contract documents. However, applicability to a specific project in a specific state will differ thus contractors should always have corporate counsel advice regarding this risk. It is for this reason that case law has tended to support the contractor when it was clear that it was unrealistic to expect the contractor to understand the subsurface conditions and the behavior of the ground on the construction process when the owner and their design engineers did not know. This has moved the industry to embrace the concept of risk-sharing. This has validated the principle that an appropriate amount money and study invested on the frontend of the project can result in a more realistic total project cost to the owner. Basically, every project has a fair cost, this cost may be realized by proper project definition and characterization or realized by claims or court actions.

This chapter provides a basic understanding of the technical approach and planning for a typical trenchless construction geotechnical characterization program and discussions regarding the communication available to the Owner team to accurately convey the appropriate level of subsurface characterization clearly to the Contractor as a risk management tool including geotechnical data reports, geotechnical interpretation reports, and the geotechnical baseline reports. In the summary, an overview of trenchless methods will be provided with an indication of the appropriate soil types.

This chapter is not intended to be an exhaustive study of the professional practice of geotechnical engineering. It will focus on the essential components of the interaction between subsurface conditions which are important to successful construction by an experienced utility contractor and operation of the project by the Owner. The geotechnical exploration program may then be planned to address with data the interaction of the project with specific subsurface conditions.

This chapter provides owners and designers of projects with lessons learned from contractors from across North America who build trenchless projects every day. They have witnessed and experienced what works and what does not work, and they have joined together to provide their recommendations to produce projects with no or minimum problems so that the owner can obtain the project quality specified, within the budget and on schedule.

3.1 TECHNICAL APPROACH

The dominant axiom of all subsurface construction is to control the ground, or it will control you. Consideration is an act of careful thought or deliberation. The act includes determination of relevant facts understanding potential outcomes, and knowledge of models that may be applied to determine pertinent stress paths that will allow control of the ground. The facts are provided from a planned geotechnical exploration program consisting of both field and laboratory data acquisition used to characterize subsurface conditions.

Planning a geotechnical exploration program requires the understanding of initial conditions, understanding of the final conditions, and the ability to create a path using the scientific process to achieve the final conditions while not losing control of the ground. The materials of the trade are soil and rock deposits, or formations created by a wide assortment of geologic processes. Deposits and formations do not care about man defined boundaries. Instead, a deposit or formation may occur over a wide areal distribution, have variable thickness, and variable properties throughout. A deposit is a single series of soils placed by a singular geologic process.

Deposits often contain units of similar materials with relatively similar properties. However, deposit materials are manufactured by Mother Nature and do not comply to any standard like engineered manufactured products like steel. Thus, each site requires uniquely definition. The quality of the work is dependent on the skill of the engineer and the quality of the data. But more importantly, in the quality of the geologic interpretation (model) of the extent, thickness, and properties of various units that will interact with the project based on strategically placed explorations to test and verify the geologic model. It is equally important to reassess the model and data as the project progresses and recognize if base assumptions about the model remain valid. Should risk assessment using probability of occurrence and elevated cost be unacceptable then additional explorations may be required to better define the model.

Geotechnical consideration is the act of careful thought applied to geology and data required for characterizing ground conditions, and the impacts of the project. Project impacts typically involve ground stress and strength changes and the mechanics of the changes that result from the project. Impacts are a function of time thus study must include long-term, short-term, and dynamic changes in ground stress and strength. Geotechnical outcomes caused by the redistribution of ground stress and changes may include: Settlement, Differential Settlement, Ground movement, Slope movements, Groundwater pressure changes, or unequal bearing capacity among many. Various geotechnical models are used by the engineer to quantify and qualify these impacts. The data required to assess project impacts by the predictive models is provided by a planned geotechnical exploration program.

Determination and deliberation of impacts from the ground reaction is based on models that incorporate the principals of stability and strength based on the properties of the ground and principals of applied physics. Should results be controlled to prevent exceeding ground strength indicate then there is little need for further planning phase exploration or actions. The next step would be observation during construction to verify planning assumptions with field facts. Should ground strength be exceeded, then geotechnical engineering is applied by the engineer by using predictive models to develop mitigation measures to control the stress redistribution or strength change. The quality of this engineering study is strongly related to the reliability of the data required by the planned geotechnical exploration program and the quality of the geologic model of the site.

Ground behavior models are based on understanding of various states of stability. There are three states of stability: Stable, Unstable, and Neutral. Stable means that an applied force will encounter a resisting force that will restore a stable condition even if some strain occurs. Unstable means that an applied force will not encounter sufficient resisting force to restore balance thus the system will continue to strain until stabilizing forces are applied. Neutral stability means that an applied force can result in continual constant translational motion until stabilizing force is applied or the object encounters a Threshold. In all cases, the task is to define which version of stability is being used to evaluate the site and is the selected stability condition acceptable to the team.

The final site characterization is strongly influenced by moral judgement, not a cost judgement, for Civil Geotechnical Engineers. The final site characterization provides a contract base for estimating project costs based on the available data. The conditions are often documented in Geotechnical Baseline Reports which are part of a contract and are defined and structured for purposes contained in ASCE guideline documents. It is an important to understand that construction provides new data. If the new data contradicts with the final site characterization, then the change needs to be recognized and handled in accordance with contract documents. For this reason, the assumptions and associated risk of occurrence and cost should be clearly provided to the Owner by the geotechnical engineer so the Owner may manage the risk.

A key underlying component to the geotechnical consideration process is he need for the geotechnical engineer to develop moral and ethical judgements. Specific criteria impacting final site characterization decisions are best stated in the American Society of Civil Engineers (ASCE) Code of Ethics that govern the behavior and judgements developed by the engineer with the foremost requirement:

1. ***Society***
   Engineers:
   a. ***First and foremost, protect the health, safety, and welfare of the public.***

PLANNING PROCESS

A typical technical approach for planning a geotechnical exploration program is to determine initial and final conditions, then determine a path to the end that maintains stable or controlled ground conditions. Understanding these tasks will identify the geotechnical considerations that may impact the project to some degree. Considerations are geotechnical hazards to the project that may be assessed by risk analyses based on probability of occurrence and cost, schedule, safety, environmental impacts should the hazard be realized. It sounds simple to assess stability in terms of say ratio of strength divided by actual stress and an appropriate factor of safety. However, determination of outcomes is dependent on accurate consideration and actual data-based determination of properties within soil or rock can vary by factors such as 1,000 times and these properties are also impacted by natural dynamic events such as rainfall and erosion, then you may begin to understand the complexity of subsurface material properties and their non-linear behavior. It is these ranges, dynamics, and required model inputs that create the considerations for planning a geotechnical exploration program. Simplified, garbage in provides garbage out and often leads to court where nobody wins.

Initial conditions are developed using Desk Top Studies to establish site geologic processes, types and extent of deposits, and preliminary properties of units within the deposits. Desktop studies rely on original project design criteria to define the location of subsurface conditions potentially impacted and the final planned loads and geometry. This process is based on Owner project objectives for a successful project along with development of an initial site subsurface condition model based on publicly available data sources along with local experience of the geotechnical engineer, local contractors, and local public works departments.

Spatial variation of relatively similar materials within a project footprint

Subsurface materials can vary widely over a given area distributed by geologic processes such as deposition, weathering, water, glacial, and man. Materials found within a specific area may have been acted on and formed and reformed by multiple geologic processes over time thus may vary widely such as: rock, soil, grainsize distribution, mineral and organic content. A particular material may have wide variations in density and groundwater content and flow. All these factors and more impact the properties of a particular material AND on the possible areal distribution of that specific material. It is these properties that are required by soil mechanics models to make predictions of stress redistribution used to assess impacts to a proposed project.

Distribution of similar property subsurface materials can provide a fact-based geologic model to allow extrapolation of point data over a specific area. Geotechnical engineers have developed simplified models to help characterize subsurface materials into similar reacting layers and form a mathematical model to predict ground stress and strain changes resulting from a project. However, a key factor in the analyses is the distribution and thickness of units within the Owner provided project footprint. The geologic model can provide a sound basis for extrapolation of point data from an exploration but requires exploration-based data to define the area extent and thickness of units.

Development of the consideration based geotechnical Exploration Program

The geotechnical engineer uses this information to develop a geologic model of the site that includes preliminary subsurface conditions and properties. The geologic model is a critical element of the site characterization as it provides the logic necessary to extrapolate point data from explorations to a particular geologic unit. Based on the model and the experience of the geotechnical engineer, geotechnical considerations are developed based on anticipated ground response to the change form initial to final stress and geometry conditions. These considerations form the purpose and objectives for development of the geotechnical exploration and laboratory characterization program. The geotechnical engineer then determines which models will be needed for predicting project impacts assigns exploration types, sampling, and laboratory testing appropriate for quantifying the areal extent and properties of geologic units by using the selected predictive models.

Predictive models are used to develop mitigation procedures by assessing how to best control the ground which does not mean preventing stress redistribution. It does mean understanding the time-rate process of the stress redistribution. Often this assessment requires a reliable geologic model, specific geotechnical data, and experience. Some of the best examples of presenting a data-based time-rate process coupled with engineering judgement and experience is the "Tunnelman's Classification System" for soil reaction to a specific excavation process and the RQD, Q-System, RMR systems for rock behavior under tunnel excavation conditions. All mitigation to control the ground must first understand the ground reaction to the project to determine what properties need to be managed to maintain control of ground stress to predictable magnitudes.

There is a need to understand ground reaction and the types of models necessary to assessing and managing these conditions. Each model requires site specific subsurface data and material properties for model input to achieve meaningful results. A few examples are provided for geotechnical consideration in project specific subsurface exploration requirements.

Stress and Strength Changes

Stress changes in the ground or applied by the structure occur when strength is exceeded which triggers strains within the ground/structure system necessary to rebalance to a stable configuration. In general, construction activity changes the stress distribution in the ground. If the resulting stress exceeds the strength, movement will occur over some time interval to redistribute stress until the stress is less than the strength. The accepted practice for assessing stability is the use of Factor of Safety, FS.

$$FS = Strength/Stress$$

Peck (1969) also references an assessment technique developed by Broms and Bennermark (1967) used to determine construction stability of the ground for caisson design in plastic clays:

$$FS = (Pz - Pa) / Su < 6 \text{ for stable conditions.}$$

Pz = total vertical pressure at depth *z* of center of tunnel
Pa = Air pressure above atmospheric (May include slurry pressure above groundwater pressure)
Su = Undrained Shear Strength of clay = qu/2

There are multiple methods for calculation of in situ stress and strength or soil and rock materials. Each has an industry practice factor of safety range that may only be changed by engineering judgement based on facts.

Geotechnical study task starts with determination of initial ground conditions. The next task is preliminary assessment of project short- and long-term impacts to in situ stress and strength. Should problems be anticipated, perform geotechnical studies to determine actual properties that may be used in models to determine mitigation efforts necessary to control the ground. Thus, data requirements include information to determine existing conditions and data used to develop methods to control the ground. Movement of ground, whether soil or rock, is generally in a direction from higher stress to lower stress. The main source of stress is the force of gravity. Other sources include hydraulic or pneumatic fluid force, buildup of avalanche or talus mass over time and a derivative of gravity, or rarely electric and magnetic forces. All forces have a direction, and the direction of the force generally indicates the direction of movement. However, different materials have different properties which can interact and redirect the force and movement.

The study of the initial stress state and changes in stress with possible resulting strains has developed into the practice of geotechnical engineering – soil and rock mechanics. The initial stress change is caused by the relatively rapid interaction of the excavation/fill process with applied new loads or unloads. To control the material, you must have a sufficient understanding to know the initial condition and amount of change that may be possible before the strength is exceeded. Exceeding strength can be accomplished providing the rate is controlled. If control is lost, then failure occurs.

Ground reaction to changes in stress and strength

Stress redistribution can ultimately result in failures. Failures may be classified by the use of stiffness analyses as brittle stress-controlled failure or flexible strain-controlled failure. Note that stiffness is a function of both material properties and geometry. A stable small excavation does not mean that a scaled up larger excavation will also be stable. It is important to seek where possible flexible or ductile failure conditions occur by design then provide sufficient time in the failure process for early detection to protect the safety of the public.

Failure mechanisms may be stress or strain controlled. The result determines which model boundaries need to be applied to determine stability and specific observations used to assess short and long -term stability. As an example of design based on stiffness tunnel stability relating to soil loads and resisting lining loads may be assessed using the Flexibility ratio. This tool was presented by Peck, Hendron, and Mohraz (1972 RETC) based on work by Burns and Richards (1964) and Hoeg (1968) for determination of the Flexibility Ratio, F. The liner is considered to be flexible if F > 10 and rigid if 1 < F < 10.

$$F = [E/(1+n)]/\{[6E_L I_L/(1-n_L^2)]*[1/R^3]\}$$

Where:
IL = the moment of inertia of the cross section per unit length of the tunnel along axis
E = Youngs Modulus of Medium
n = Poisson's ratio of medium
EL = Youngs modulus of the liner ring
R = Nominal radius of the liner ring
t = Thickness of the ring

3.2 CLASSIFICATION SYSTEMS

The purpose of classification systems is to have a definition of a specific unit that has relatively common properties within the deposit to allow analyses using predictive models for a specific purpose. There are several classification systems each developed to communicate soil descriptions for a specific purpose. These classification systems are used to communicate a description of the site subsurface materials on exploration logs and in data reports. Understanding the soil type sufficiently to apply a classification allows assessment of anticipated behavior of the soil during construction. The classification system and description must be both disclosed in reports or logs to properly communicate the description. For detailed information on these classification systems, the reader is referred to the references on soil mechanics at the end of this manual.

Classification systems commonly separate a specific material into either a plastic material called cohesive material, or a non-plastic material called granular. Regarding construction, the main difference is that a plastic material requires longer time than typical construction durations to readjust to newly applied stresses while non-plastic material reacts over a relative short time interval typically less than construction duration.

Various standard and guideline documents have been developed that provide specific definitions to classify common subsurface materials and are used in the trenchless technology field. ASTM D288 "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) offers a procedure to enhance the soil description from multiple classification systems to include anticipated soil behavior. Several of the systems in common use and developed for a specific purpose include:

- Burmister System
- Unified Soil Classification System (USCS)
- American Association of State Highway and Transportation Officials (AASHTO)
- Occupational Safety and Health Administration (OSHA)
- Tunnelman's Ground Classification System (TGCS)
- Terzaghi, Peck, and Mesri System

There is not unanimous agreement on the exact size division between the major soil types of clay, silt, sand, and gravel. Gravel and sand are usually considered coarse-grained, since the individual particles are large enough to be distinguished without magnification. Silts and clays are considered fine-grained soil because their small particles cannot be seen with unaided eyes. The most commonly used divisions for classifying soils for engineering and construction purposes are shown in Table 3.5. On a comparative basis, gravel size is between 0.08 in. (2 mm) and 8 in. (200 mm), whereas particle size for sand range from 0.003 in. (0.074 mm) to 0.08 in. (2 mm). Silt particle sizes range from about 0.002 to 0.075 mm. Clay particles sizes are those less than about 0.002 mm.

Table 3.1 Soil Classification Based on Grain Size. (Liu and Evett, 1998)

Agency	Coarse-Grained			Fine-Grained	
	Gravel	Coarse Sand	Fine Sand	Silt	Clay
AASHTO millimeters (Sieve Size)	75.0-2.00 (3-in.-No. 10)	2.00-0.425 (No.10-No.40)	0.425-0.075 (No.40-No.200)	0.075-0.002	<0.002
USCS millimeters (Sieve Size)	Coarse: 75.0-19.0 (3-in.-3/4 in.)	4.75-2.00 (No.4-No.10)	0.425-0.075 (No.40-No.200)	Fines<0.075 mm (Silt or Clay)	
	Fine: 19.0-4.75 3/4in.-No.4	Medium Sand: 2.00-0.425 (No.10-No.40)			

Burmister Soil Classification System

The Burmister system is in common usage throughout the United States since it's development in 1950. This system is based on observation of soil samples recovered from field sampling and is a relatively simple system. The following table provides guidance for developing a soil description based on the Burmister System,

3.2.1 American Association of State Highway and Transportation Officials (AASHTO)

The purpose of The American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) soil classification system is to service highway engineering projects. The AASHTO System has seven soil classes including A-1 to A-7. The soil is placed in the classes based on performance characteristics. A-1 is the highest and A-7 is the lowest. A-1 to A-3 are sands and gravels, A-4 to A-7 are silts and clays. Table 3.1 illustrates AASHTO soil classification system.

Table 3.2 Burmister Soil Classification Naming System. *(Source: Dunn Geoscience Corp.)*

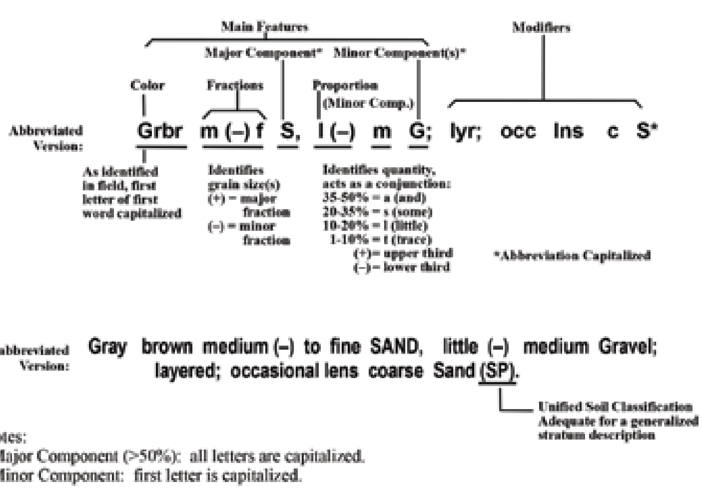


Table 3.3 Burmister Soil Classification System Coarse-Grained Soils, Gradation of Components.

Course to fine	cf	All sizes
Course to medium	cm	Less than 10% fine
Medium to fine	mf	Less than 10% course
Coarse	c	Less than 10% medium and fine
Medium	m	Less than 10% course and fine
Fine	f	Less than 10% coarse and medium

Table 3.4 Bumister Soil Classification System Fine-Grained Soils, Plasticity of Components.

Silt	$	Non-plastic	0 to 1
Clayey Silt	Cy$	Slight	1 to 5
Silt & Clay	$ & C	Low	5 to 10
Clay & Silt	C & $	Medium	10 to 20
Silty Clay	$yC	High	20 to 40
Clay	C	Very High	over 40

3.2.2 Unified Soil Classification System (USCS)

Unified Soil Classification System (USCS) was established by A. Casagrande in 1948 for the purpose of airfield construction. It was modified in 1952 by the Bureau of Reclamation the U.S. Army Corps of Engineers to adopt this classification system for foundations, dams, and other construction projects. USCS requires the use of sieve analysis and Atterberg limits. There are four main group of soils including 1. course-grained, 2. fine-grained, 3. organic soils, and 4. Peat. ASTM D2487 provides a detailed description of the system.

The USCS system recognizes the value of descriptions that provide information describing soil behavior under changing stress and strength conditions and has included both grainsize distribution index and Atterburg Limit index values into the classification system. The grainsize index values provide behavior information for granular soil and Atterberg Limits index values for cohesive soils.

Index values for granular soil used in the USCS include the coefficient of uniformity (Cu) and the coefficient of curvature (Cc) which are defined in Figures 3.2, 3.3 and 3.4. If Cu = 1 than all the soil particles are the same size thus the soil is poorly graded. Cu values of 2 – 3 indicate a poorly graded material. Poorly graded

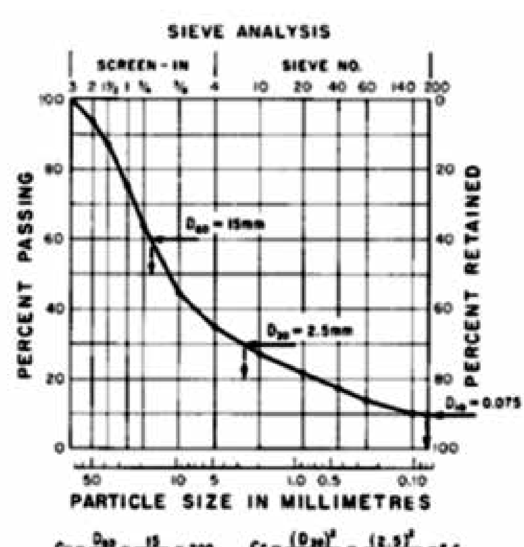


FIG. 5 Cumulative Particle-Size Plot

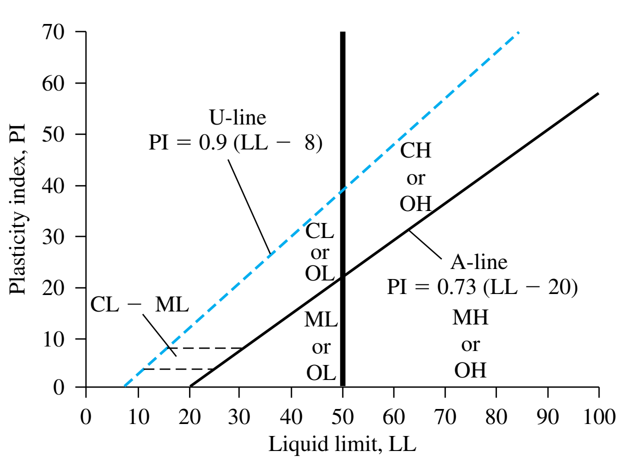
Table 3.5 AASHTO Soil Classification System 9 (from AASHTOL M 145 or ASTM D3282).

Genral Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Sit-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 max	...	...	...	...	...	...	...	...
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[1]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Note (1): Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index A-7-67 subgroup is greater than LL - 30

granular soils are considered to have poor stability during the excavation process and are sensitive to very sensitive to groundwater conditions when encountered. A typical poorly graded material would be Beach Sand. The coefficient of curvature provides an indication of a well graded soil. When Cc is between 1 and 3 the soil is considered to be well graded. A well graded soil with more than 18% material by weight captured on the US #200 sieve is generally considered a relatively stable material when subjected to construction processes. In both cases, the term stable refers to the relative potential stand up time of the material during the construction process time interval and not an absolute condition.

Granular soils tend to fail very quickly and continue to fail relatively quickly over time and until stabilizing conditions have been applied. Silts and fine sand have the highest degree of instability potential when water is present. Applying density information developed from field explorations with the USCS system provides an additional lever of soil behavior, compaction, and dilation. A disturbed loose granular material will quickly collapse as there is sufficient room with the matrix for a denser packing of the particles. Failures of loose granular soils can very quickly develop into chimney shaped features that extend to the ground surface or until a more stable material is intersected. However, a dense granular material must increase in volume before it can fail. Failure progress will be limited by soil arching unless the construction process is continued.



The behavior of cohesive soils changes significantly with the moisture content (w%). Atterberg Limits (ASTM D4318 "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of soils") provide an indication if a soil is a silt (granular material) or a clay (cohesive material). Limits are based on four behavioral conditions of a cohesive soil based on the moisture content: Solid, Semi-Solid, Plastic, and Liquid. The limits are defined as the soil moisture content at the transition between each of the four soil behavioral conditions and are called, Shrinkage Limit (SL), Plastic Limit (PL), and Liquid Limit (LL), and one of the original Atterberg Limits the Adhesion limit (AL). The adhesion limit is the moisture content when the soil just starts to stick to a steel spatula. Typical laboratory testing of recovered field soil samples only provides the Plastic and Liquid limits. However, by knowing the natural water content of the soil, the Atterberg Limits will provide an indication of how the soil will behave when stresses change including a qualitive indication of how much standup time may be available during construction.

Index values for a soil have been developed that indicate various behavioral attributes of a cohesive soil and include: Plastic Index (PI), Liquidity Index (LI), and Consistency Index (Ic), Activity Number (AN).

PI = LL – PL: Higher PI indicates higher clay content and PI >25 may indicate swelling/shrinking Clay unit.

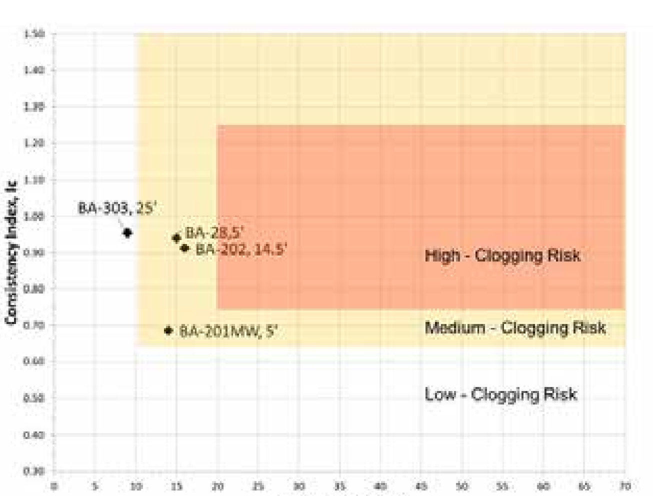
LI = (w% - PL)/PI: As LI approaches 1 the material behaves more like a liquid while LI <=0 is harder and behave more brittle like.

CI = (LL-w%)/PI: Higher CI indicates higher shear strength where w% is the in situ water content. This index has been shown to be an indicator of clay stickiness or the ability of clay to clog or inhibit spoils removal systems associated with trenchless excavations. The following diagram provides interpretation of index data for clogging by Thewes.

AN = PI/% by weight finer than 2 micrometers: AN > 1.25 are considered to be active and will higher volume changes with respect to moisture content. They expand and shrink.

3.2.3 Tunnelman's Ground Classification System (TGCS)

Terzaghi at 1950[11] for the first time provided the Tunnelman's Ground Classification system which became famous as a common and functional tunneling classification of soft ground, and later Heuer in 1974 refined it.[12] Tunnelman's Ground Classification System was designed to explain different ground performances and their effect on larger, conventionally constructed soft ground tunnels. This system is also a powerful tool to evaluate the soft ground behavior applicable for underground construction projects. Table 3.6 provides a general description of various ground performances.



Terzaghi, Peck, and Mesri System

The following is a list of soil types used for field classification excerpted from Terzaghi, Peck and Mesri (1996) and based on excavation soil reaction behavior or commonality of properties for subsurface projects.

- **Sand and Gravel:** Sand and gravel are cohesion less aggregates of rounded sub-angular or angular fragments of more or less unaltered rock or minerals. Particles with size up to 0.08 in. (2 mm) are referred to as sand, and those with a size from 0.08 in. (2 mm) to 7.9 in. (200 mm) are gravel. Fragments with a diameter of more than 7.9 in. (200 mm) are defined as boulders.
- **Hardpan:** Hardpan is a soil that has an exceptionally great resistance to the penetration of drilling tools. Most hardpans are extremely dense, well-graded, and somewhat cohesive aggregates of mineral particles.
- **Inorganic Silt:** Inorganic silt is a fine-grained soil with little or no plasticity. Because of its smooth texture, inorganic silt is often mistaken for clay, but it can be distinguished without laboratory testing. If shaken in the palm of the hand, a part of saturated inorganic silt expels enough water to make its surface appear glossy. If the pat is bent between the fingers, its surface again become dull. This procedure known as shaking test. After the pat has dried, it is brittle and dust can be detached by rubbing it with the finger. Silt is relatively impervious, but if it is in a loose state, it may rise into a drill hole or shaft like thick viscous fluid. The most unstable soils of this category are known locally under different names, such as bull's river.
- **Organic Silt:** Organic silt is a fine-grain more or less plastic soil with an admixture of finely divided particles of organic matter. Shells and visible fragments of partly decayed vegetable matter may also be present. The soil ranges in color from light to very dark clay, and it is likely to contain a considerable quantity of H_2S, CO_2 and various other gaseous products of the decay of organic matter. The permeability of organic silt is very low and its compressibility very high.
- **Clay:** Clay is an aggregate of microscopic and submicroscopic particles derived from chemical decomposition of rock constituents. It is a plastic within a moderate to wide range of water content. Dry specimens are very hard, and no power can be detached by rubbing the surface of dried pats with the fingers the permeability of clay is extremely low.
- **Gumbo:** The term "gumbo" is applied, particularly in the western United States, to clays that are distinguished

Table 3.6 Tunnelman's Ground Classification for Soils.

Classification		Performance	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above the water table; hard clay, marl, cemented sand, and gravel when not highly overstressed.
Raveling	Slow raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed. This is caused by loosening or overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, as opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon the degree of overstress.
	Fast raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow are caused by overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination with raveling at excavation surface and squeezing at depth behind surface.
Running	**Cohesive-running**	Granular materials without cohesion are unstable at a slope greater than their angle of repose (from ±30 to 35 degrees). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with a plasticity index in excess of about 30, generally containing significant percentages of montmorillonite clay.

in the plastic state by soapy or waxy appearance and by great toughness. At higher water contents, gumbo is very sticky.

- **Organic Clay:** Organic clay is material that owes some of its significant physical properties to the presence of finely divided organic matter. When saturated, organic clay is likely to be very compressible. When dry, its strength is very high. It is usually dark gray or black and it may have a noticeable odor.
- **Peat:** Peat is somewhat fibrous aggregate of microscopic and microscopic fragments of decayed vegetable matter. Its color ranges between light brown and black. Peat is so compressible that is almost always unsuitable for supporting foundations.
- **Till:** Till is an un-stratified glacial deposit of clay, silt, sand, gravel, and boulders.
- **Tuff:** Tuff is fine-grained water- or wind-laid aggregate of very small mineral or rock fragments ejected from volcanoes during explosions.
- **Loess:** Loess is a uniform, cohesive, wind-blown sediments, and is commonly light brown. The size of most of the particles ranges between the narrow limits of 0.01 and 0.05 mm. the cohesion is due to the presence of a binder that may be predominately calcareous or clayey. Because of the universal presence of continuous vertical root holes, the permeability in vertical directions is usually much greater than on horizontal directions. Moreover, loess has the ability to stand on nearly vertical slopes. True loess deposits have never been saturated. On saturation, the bond between particles is weakened and the surface of the deposit may settle.
- **Marl:** Marl is a loosely used term for various fairly stiff or very stiff marine calcareous clays of greenish color.
- **Lake Marl:** Lake marl or bog lime is a white fine-grained powdery calcareous deposit precipitated by plants in ponds. It is commonly associated with beds of peat.
- **Sedimentary Rock.**
- **Shale:** Shale is a sedimentary rock mainly composed of silt-size and clay-size particles. Most shales are laminated and display fissility or splitting, since the rock tends to

split along relatively smooth and flat surface parallel to the bedding. When fissility is completely absent, the classic sedimentary deposit is called mudstone or clay rock. Depending on the mineralogy, void ratio, and degree of diagenetic (the process of chemical and physical change in deposited sediment during its conversion to rock) bonding or weathering, compressive strength of shales may range from less than 362 psi (2.5 MPa) to more than 14,504 psi (100 MPa).

- **Granite Rock.**
- **Adobe:** Adobe is a term applied in the southern United States and other semiarid regions to a great variety of light-colored soils ranging from sandy silts to very plastic clays.
- **Caliche:** Caliche refers layers of soil in which the grains are cemented together by carbonates deposited as a result of evaporation. These layers commonly occur at a depth several feet (meters) below the surface, and their thickness may range up to a few feet (meters). A semiarid climate is necessary for their formation.
- **Varved Clay:** Varve is a layer or a series of layers of sediment deposited in a body of still water. Varved clay consists of altering layers of medium gray inorganic silt and darker silty clay. The thickness of the layers rarely exceeds 0.4 in. (10 mm), but occasionally very much thicker varves are encountered. The constituents were transported into freshwater lakes by melt water at the close of the ice age. Varved clays are likely to combine the undesirable properties of both silts and soft clays.
- **Bentonite:** Bentonite is a clay with a high content of montmorillonite. Most bentonites were formed by chemical alteration of volcanic ash. In contact with water, dried bentonite swells more than other dried clays, and saturated bentonite shrinks more on drying.

3.2.4 Occupational Safety and Health Administration (OSHA)

The Occupational Safety and Health Administration (OSHA) publishes federal standard to establish requirements for the protection of employees involved with excavations. The standard, titled 29 CFR, Part 1926, Subpart P, became effective in 1990. Included in this standard are crucial guidelines for analyzing soils and subsurface conditions with respect to minimum allowable parameters that may be applied for subsurface safety concerns. These values may not be exceeded without an engineer's recommendation based on sound engineering data. These safety considerations are typically applied to slope stability, shaft stability and occupied tunnel stability calculations. The OSHA soil classification divides soils into three main categories (Mickle, 1996):

Type A: Type A soils are soils with an unconfirmed compressive strength of 1.5 tsf (144 kPa) or greater. Examples of cohesive soils are: clay, silty clay, sandy clay, clay loom and sandy clay loom. Cemented soils such as caliche and hardpan are also considered Type A. However, no soil is Type A if:

- The soil is fissured
- The soil is subject to vibration from heavy traffic, pile driving, or similar effects
- The soil has been previously disturbed
- The soil is part of sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H: 1V) or greater
- The material is subject to other factors that would require it to be classified as a less stable material

Type B: Type B soil is:

- A cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa)
- Granular cohesionless soils including angular gravel (similar to crushed rock), silt, silt loom, sandy loom, and in some cases, silty clay loom and sandy clay loom
- Previously disturbed soils, except those which would otherwise be classed as Type C soils
- Soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration
- Dry rock that is not stable
- Material that is part of the sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H: 1V), but only if the material would otherwise be classified as Type B

Type C: Type C soil means:

- Cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less
- Granular soils including gravel, sand, and loamy sand
- Submerged soil or soil from which water is freely seeping
- Submerged rock that is not stable
- Material in a sloped, layered system where the layers dip into the excavation or a slope of four horizontal to one vertical (4H: 1V) or steeper

3.3 MATERIAL STRENGTH

Strength typically refers to the shear strength capacity of a specific material under specific conditions. Strength of subsurface materials is a combined function of material type, density, grain size and shape, degree of saturation, rate of load change, and fluid drainage capacity among other properties. Values have a wide range and are influenced by the existing stress state and historical stress path at the specific position of the strength determination within the subsurface. Changes in any of these factors for any reason will change the strength of the material at the specific location.

The most common method used to determine shear strength (S) of a subsurface material is known as the Mohr-Coulomb model. This model is represented by the following equation:

$$S = c + s'*\tan(f)$$

Where

S = shear strength that may be further defined and undrained Strength (Su) or drained Strength (Sd). Undrained strength usually applies to short term conditions while drained conditions usually apply to long term conditions. Short-term and long-term conditions relate to water drainage rate from the system to the time of the condition change.

c is the cohesion intercept of a cohesive material based on field in situ vane or penetrometer testing.

s' = st – u is the effective stress at the measurement location. Effective stress is the buoyant weight of overlying materials to the measurement point in the ground or the total pressure applied by the ground minus the groundwater pressure at the calculation point. Total pressure is the unit weight of each soil layer or overlying free water times the thickness of the soil layer or free water layer above the calculation point. Groundwater pressure is the unit weight of water times the height of groundwater above the calculation point.

f is the angle of internal friction of the material and usually applies to granular materials.

Strength determination requires field exploration, laboratory index testing along with site geology and geometry of the calculation location.

3.3.1 Strength of Cohesion-less Soil (Silt, Sand, and Gravel)

The strength of a cohesion-less soil depends on confining stress, unit weight or density, grain size, shape, mineralogy, and fines content. Primarily, it is controlled by density and confining stress. Strength arises from intergranular friction and interlocking controlled by these factors. Variations in strength are indicated by variations in the angle of internal friction. It may vary from about 28 degrees to about 45 degrees for a range of conditions from loose to very dense.

3.3.2 Strength of Cohesive Soil (Clay)

The strength of clay soils depends greatly on their drainage conditions. If the soil is saturated, and sheared without draining, the angle of internal friction is zero and the strength is dependent on cohesion, referred to as the undrained shear strength (Su) for this condition. If clay soils are sheared under completely drained conditions, the strength is dependent on friction angle and cohesion intercept. In many cases, some drainage occurs and clays exhibit both cohesion and friction.

3.4 ROCK QUALITY DESIGNATION INDEX (RQD-VALUE)

The Rock Quality Designation Index (RQD-Index) is a classification method indicating rock mass stability based on measured lengths of rock core pieces obtained by field explorations. This method was introduced in 1967 by Deere.[10]

The RQD index value can be calculated through following equation:

$$RQD = (L10/L) * 100\% \quad (1)$$

where,

L10, is the accumulative length of core pieces > 4 in. (10cm) length for NX sized core,

L, is the total length of core run (typically over a 60-in. core run)

L10, for the intact pieces with the length of less than 4 in. (10cm) will be assume zero. Based on the calculated RQD percentage and by using Table 3.7 on page 32, the RQD-Index is dependent on the orientation of the core. As rock joint patterns are dependent on rock formation and post deposition stress history, then the RGD can vary with sample orientation to the formation development and post formation stress history.

Several other classification systems including Bienowski's RMR system and Barton's Q system have been developed for prediction of rock stability and design of various shaped excavations.

Cost associated with rock excavation is very sensitive to prediction of the production rate and the amount of tool ware. There is a considerable amount of science that may be applied to determine penetration rate and tool ware. However, all prediction methods rely on basic formation properties that require both field and laboratory data. Detailed explanation of analyses is beyond the scope of this chapter however, the following field and laboratory data is basic for completing any analyses and predictions.

- Unconfined Compressive strength
- Slake stability test as developed by Washington State DOT
- Cherchar
- Brazilian tensile
- Punch test for rock toughness
- Mohs hardness test for the mineral composition of the rock
- Minerology of the rock as determined by thin section geologic analyses
- Geometry and geology of formation units

An effective index of penetration rate can be developed by recording the subsurface exploration penetration rate per foot of drill advance along with a description of the drill bit, hole size, and downhole pressure applied to the drill bit.

3.5 SUBSURFACE INVESTIGATION

The purpose of subsurface investigations is to generate project specific data that quantifies and qualifies subsurface conditions for specific short- and long-term interaction between project and ground. Long-term interactions are generally the concern of the designer. Short-term impacts generally involve construction and Contractor's means and methods. This chapter focuses on Contractor means and methods by providing understanding of what is pertinent data and how is that data presented and interpreted for managing contractor related issues. Basically, how should geotechnical field and laboratory data be interpreted for contractor-controlled issues.

Table 3.7 RQD value description.

RQD value	Description of the rock quality
0 to 25	Very poor
25 to 50	Poor
50 to 75	Fair
75 to 90	Good
90 to 100	Excellent

Contractor objectives include: assess the feasibility of completing the project using certain tools with appropriate means and methods, manage the likelihood of damaging existing facilities from uncontrolled subsurface movements caused by construction, and provide the information needed to allow selection of the most appropriate construction technique and equipment. The extent of site investigations and laboratory testing depends on the location and complexity of the project and the hazards with associated risks associated with the project.

Assessment of site-specific issues requires qualification and quantification of subsurface conditions. It is necessary to qualify the actual soil stratification and quantify the areal extent and thickness of deposits using field explorations along with determining the physical properties associated with a deposit using laboratory test results of the soil samples obtained from various depths and deposits. Data is developed in phases each based on what you know then verification by field data and laboratory testing of samples.

Planning subsoil investigation include:

Desk Top Study for Trenchless Projects

1. Define the project potential footprint
2. Develop a site-specific geologic model for the site based on publicly available sources that extends to a potential minimum depth of 3 times the potential project depth.
3. Visit to the site by an experienced geotechnical engineer, construction specialists, designer, environmental specialists, and permitting specialists.
4. Develop initial understanding of geologic conditions within the work area such as:
   a. Is project in soil, rock, or both?
   b. What historical and current geologic processes may be associated with the site deposits?
   c. Is groundwater within the project work area or not?
   d. Can and which materials are sensitive to groundwater and how are they sensitive?
   e. Are Soils granular or cohesive or both?
   f. Is rock sedimentary, metamorphic, or igneous?
   g. Are deposits anticipated to be wide-spread or localized and limited in extent?
   h. Are deposits anticipated to be thick or thin relative to size of excavation?
   i. Are deposit contacts expected to be level, undulating, gradational, convoluted?
   j. Is surface topography level or undulating?
   k. Are there indications of how stable is the ground?
   l. What are the seismic magnitudes and probability?
   m. Is there surface water or not within say 1 mile of the work site?
   n. Has the site been subjected to historical disturbance by construction?
   o. Are there chances of encountering hazardous materials (contamination)?
   p. What is the present usage of the work site?
   q. What are potential social and environmental exposure conditions at the site?
5. Develop construction means and methods that may be possible for building the project.
6. Develop a list of geologic hazards that if encountered within the geologic model will impact the project such as:
   a. Short- and long-term stability of the deposits,
   b. Production rates
   c. Tool ware
   d. Create or change groundwater or surface water natural routes.
   e. Require control of say groundwater for anticipated construction means and methods
   f. Require modification of properties to allow anticipated means and methods of construction.
   g. Require long term modification to stabilize the site with the new construction.
7. Documentation consisting of a report.

Phase 1 Subsurface Explorations and Laboratory Testing to verify and provide initial qualification and quantification estimates of actual subsurface conditions that can impact the project.

1. Engineering assessment of Desktop data to determine which subsurface properties will be needed to assess the geologic hazards.
2. Determine locations and planned depths for explorations at the site to assess actual geometry of subsurface conditions such as:
   a. Verification of anticipated geologic conditions based on the model,
   b. Initial magnitude of depth below existing grade and thickness of deposits,
   c. Sampling to acquire appropriate material to complete laboratory testing that would provide property data necessary for engineering analyses.
   d. Depth of explorations to extend to typically 50 ft below planned construction to produce data that allows for engineering the best space for the construction in the ground without data limitations.
   e. Determine critical project design features that require immediate data to assess the viability of a specific construction at a specific location due to project Owner defined issues or property right of way issues.
   f. Determine laboratory testing necessary to develop appropriate engineering properties required by engineers to assess stability and premium cost issues that may be associated with the project.
   g. Develop survey data to determine actual property constraints such as ownership, environmental restricted zones, and required construction easements for anticipated construction means and methods
3. Site assessment regarding access for exploration equipment to exploration locations
4. Site survey plan to accurately determine surficial topography and conditions within a universal system such as State Plane and vertical datum and to determine planned and as-constructed explorations.
5. Documentation of data to provide a data base for engineering associated with the project premium cost issues, verification of the potential magnitude of issues identified in the Desk Top study, and assessment of the risk of encountering hazards that will impact project design, cost, and schedule.

Engineering/Preliminary Design Assessment – Do we know enough to manage risk?

1. Assessment to determine if and what are the important issues that could impact project cost, schedule, design, environment, and safety associated with the project.
2. Using the assessment results indicating the selected important issues should they exist, plan a second exploration program to enhance the areal and vertical extent of the issue and acquire additional data to better define subsurface properties via engineering studies.
3. Determine exploration locations and objectives.
4. Document the objectives for the exploration program

Phase 2 Subsurface Explorations and Laboratory Data

1. Based on engineering recommendations, develop a second exploration program with similar criteria to the initial program.
2. Exploration depths may be limited to 20 ft below the invert of anticipated maximum depth of construction as redefined by the Preliminary Design Assessment.
3. Document the data from the exploration and laboratory program as the basis for final design.

Engineering/Design Assessment – Do we know enough to manage risks

1. The engineer's job is to assess the given subsurface conditions as they may impact the project construction and long-term operation and determine what are you going to do about managing the issues.
2. Develop final design recommendations and engineering cost estimates to qualify and quantify project cost and schedule.
3. Assess with the Owner the project hazards and associated potential cost and chance of occurrence. Owner to determine if the associated risk is acceptable or too broad to accept and determine best way to manage the risk. Often this requires additional subsurface data to better quantify the magnitude of the cost associated with encountering a hazard. This would be the purpose in an additional subsurface exploration program.
4. Document the final basis of design data base and risk with associated contingency funding along with recommendations for additional data to better manage the risks associated with encountering specific hazards.

Phase 3 Subsurface Explorations and Laboratory Data

1. Develop the program to address qualification and quantification data necessary to manage a contingency cost and schedule program-based owner Risk tolerance and contract allocation of risk cost and schedule.

Communication of data to Contractor

1. Produce documentation to disclose all data to Contractor and data interpretations as they impact project specific construction and design risk management.

Field Observations to assess new field data vs. expected data and construction interaction

1. Construction provides new data. This new data may be as anticipated or different form the anticipated data already acquired for project design. If the new data indicates different materials that change either construction cost or schedule or long-term project performance, then the new information needs to be communicated to the engineer and owner for consideration and impact to construction cost and schedule. A trained experienced construction observer is often used to determine and document possible changes and to initiate the reevaluation process. This person is critical to impartial determination of changes from anticipated conditions and associated impacts to project cost, schedule, environment, and safety.

3.6 COMMUNICATION: GEOTECHNICAL REPORTS

Data and data interpretation specific for the project must be effectively communicated to the designers and the contractors so that they understand the character of the subsurface and how this will impact the design and at least one viable method of construction. Proposed alternative construction processes require reevaluation by the design engineer of the database to determine if designed mitigation of specific issues remain valid or if alternative mitigation will be required. This communication is done through geotechnical reports.

Geotechnical reports will form the database that will be used to assess if disclosed site conditions based on subsurface explorations fairly represent conditions encountered during construction or not. If subsurface conditions are different and the difference significantly impacts the Contractor means and methods approved by the Owner and their representative, then the Contractor may be entitled to a Differing Site Condition (DSC) request or claim. The DSC concept was developed to limit subsurface conditions to defined conditions for a defined construction means and methods thereby reducing the bid risk contingency pricing resulting from Contractors forced to take on subsurface risk. The basic Contractor concept for these earlier projects was: 'Contractors do not take on unknown risk. They process them'. This concept frequently resulted in high bids for projects. A contract DSC clause is required in Federally funded projects and in most state and some private projects.

The function of the DSC is two-fold. First, it relieves the contractor of assuming the risk of encountering conditions differing materially from those indicated or ordinarily encountered. Second, it provides a remedy under the contract, so the matter is handled as an item of contract.

The ease of administering the DSC clause depends on how well the anticipated conditions are defined. The more clearly the anticipated conditions are defined, the more easily the encountered conditions can be evaluated as being materially different or not. Clear, precise baselines enhance the benefits and use of the DSC clause, because they provide a more straightforward basis for its determination.

NUCA supports the practice of risk-sharing with each party being assigned the risks they are most qualified and experienced to manage. The professional geotechnical engineer is the party most qualified to conduct the field investigation and provide the data and information needed by the designer and the contractor. They are the most qualified party to develop the interpretation of their investigation and analysis. The contractor is most qualified to develop the means and methods when provided with accurate and complete characterization of the subsurface conditions.

Geotechnical reports can vary in format depending on the owner's requirements and the nature of the project. NUCA supports the practice of full and accurate disclosure. This includes reporting the results of all subsurface investigations such as soil and rock types, the exploration logs, in situ and laboratory test results and groundwater conditions. The report needs to include the analysis and accurate interpretation. In general, the common format for the geotechnical report should include the following parts:[3]

- Scope and purpose
- Introduction
- Geologic setting
- Field studies
- Laboratory tests
- Analysis
- Conclusions and recommendations
- Appendix including all actual data

There are three (3) types of geotechnical reports in common usage: Geotechnical Data Report (GDR), the Geotechnical Interpretative Report (GIR), and the Geotechnical Baseline Report (GBR). It is the owner's responsibility to determine which reports are appropriate for the specific project and the contractual weight of each report. Often the interpretative report can be merged into the data report to become the projects geotechnical report of record. A GBR is recommended for projects as a mitigation tool for projects which anticipate complex geological conditions or high-risk circumstances that impact cost, schedule, environment, social, or safety related issues. The GBR is typically considered to be a contract document or at least a definitive document that a Contractor is allowed to rely upon to develop appropriate means and methods for supporting a Contractor Bid. This document is not developed or provided as 'For Information Only.'

A microtunneling example is provided as an example of condi-

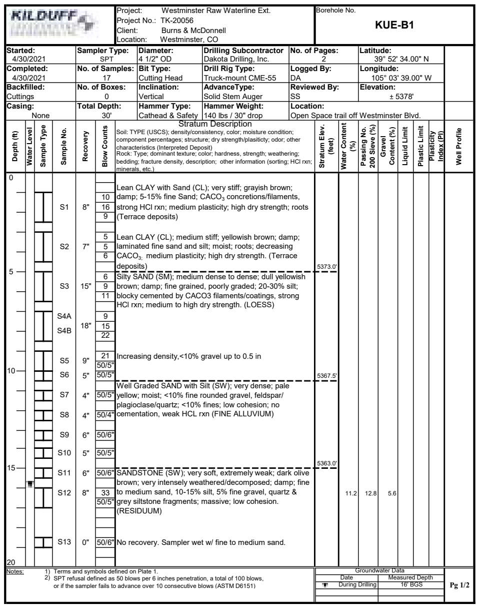


Figure 3.1 Sample of Borehole Log.

tions where a GBR is an effective tool to manage risk for trenchless projects. Slurry microtunneling has high setup and variable costs but limited mitigation measures that can be applied should conditions change. Additionally, though a microtunnel machine could handle a wide range of subsurface conditions, a specific machine setup can only handle a relatively specific range of material conditions. Changes to initial machine setups may require complete removal of the machine to retool machine operations or open excavation at the face of the machine to adjust the excavation tooling. Once the MTBM is launched, it must complete the drive or suffer enormous cost overruns resulting from retooling for different conditions or from very slow progress resulting from using the machine out of the initial ground setup tolerance. These overruns often are 3-4 times the bid unit price. Differing site conditions claims for microtunneling projects often exceed 7 figures. The typical cost of a GBR is miniscule in comparison and so is a dispute review board consisting of experts for any microtunneling project over $5 million.

3.6.1 GEOTECHNICAL BASELINE REPORT (GBR)

NUCA supports the position of the ASCE on the principle of risk-sharing with the realization that the owner really owns the underground where the pipe will be installed. The owner is entitled to receive the project as designed at the level of quality specified which has been constructed safely within the allowable budget and delivered on time. It was observed in the industry that changed condition claims were increasing and that a major challenge was for the parties to agree that a changed condition existed. To deal with this the concept of a geotechnical baseline report was established initially within he large tunneling industry. This concept is easily adapted to the trenchless industry as a risk management tool.

The background, justification and description of the geotechnical baseline is presented in ASCE's GBR for Underground Construction (Essex, 1997) publication. These guidelines and practices were prepared by the Technical Committee on Geotechnical Reports of the Underground Technology Research Council (UTRC), which is sponsored by the Construction Division of ASCE and the American Institute of Mining, Metallurgical, and Petroleum Engineers.

The purpose of the GBR is to establish a contractual statement of the geotechnical conditions expected to be encountered during underground construction. This contractual statement is referred as the baseline. This baseline would provide a clear indication in the contract regarding anticipated subsurface conditions for resolution of disputes. Secondary purposes of the GBR include:

- The presentation of the geotechnical and construction considerations that formed the basis of the design for the project
- The enhancement of the contractor's understanding of the key project constraints, and selected requirements in the contract plans and specifications
- The identification of important considerations and constraints that need to be addressed during bid preparation and construction
- Assistance to the contractor in evaluating the requirement for excavating and supporting the ground
- Guidance to the construction manager in administering the contract and monitoring contractor performance.

The GBR is a prescriptive and structured report that includes following section:

1. **Introduction,** (Project name, project owner, design team, purpose of report)
2. **Project description,** (Project type and location, key project features, references)
3. **Geological information source,** (Reference to GDR, other available geotechnical reports)
4. **Project geological setting,** (geologic and ground water setting, maps, figures, site exploration, testing program, environmental and topographical conditions, exposures and outcrops, geologic profiles)
5. **Previous construction experience,** (nearby relevant project, past project features, difficulties of construction projects and solutions)
6. **Ground characterization,** (Physical characteristics, baseline purposes ranges and values, ground mass permeability, rock mass properties)

Table 3.7 Ground Conditions and Suitability of Trenchless Methods.

Ground Conditions	Auger Boring	Slurry Microtunneling	Auger Microtunneling	Slurry Boring	Pipe Ramming	Thrust Boring Compaction	Impact Moling Compaction	HDD	Mini-HDD	Pipe Jacking	Utility Tunneling
Soft to very soft clays, silts & organic deposits	Y	Y	M	Y	Y	Y	N	Y	Y	M	Y
Medium to very stiff clays and silts	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Hard clays and highly weathered shales	Y	Y	Y	Y	M	M	M	Y	Y	Y	Y
Very loose to loose sands Above water table	M	Y	Y	M	Y	Y	M	Y	Y	M	Y
Medium to dense sands Below the water table	N	Y	N	N	N	N	N	Y	Y	N	N
Medium to dense sands Above the water table	N	Y	N	N	N	N	N	Y	Y	N	Y
Gravels & cobbles less than 50-100 mm diameter	Y	Y	Y	N	Y	M	M	M	M	Y	Y
Soils with significant cobbles, boulders, and obstructions larger than 100-150 mm diameter	M	M	M	N	Y	N	M	M	N	M	M
Weathered rocks, marls, chalks, and firmly cemented soils	Y	Y	Y	N	M	N	M	Y	Y	M	Y
Slightly weathered to unweathered rocks	Y	N	Y	N	M	N	N	M	M	N	Y
Yes (Y)	Generally suitable by experienced contractor with suitable equipment										
Marginal (M)	Difficulties may occur, some modifications of equipment or procedure may be required										
No (N)	Substantial problems, generally unsuitable or unintended for these conditions										

NOTE:
1) Utility tunneling in flowing cohesionless soils using compressed air ground stabilization was a regularly used method into the early 2000s before slurry microtunnel pipe jacking became the dominate tunnel method.
2) "Hard" granitic rock microtunneling has been successful for 15 years with the advent of face access MTBMs.

7. **Design recommendations,** (Ground classification systems, design methodologies and criteria, environmental considerations)
8. **Construction recommendations.** (ground behavior, sequences of construction, construction difficulties, rationale for baseline estimates, blinded ground improvement techniques, source of delay)

The primary benefits of GBRs can be summarized as follows:
- The Owner will have a more informal understanding of how the financial risks for subsurface conditions have been allocated
- The Bidder will have a closer and more objective basis upon which to price the work and assess risks
- There may be fewer disagreements among the contractors, owner, and engineer regarding the anticipated site conditions
- There will be a more clearly defined bases for the parties and/or third-party adjudicators to determine what does and what does not constitute a differing site condition
- If all bidders can base their estimates on a well-defined set of site conditions with assurance that equitable reimbursement will be made when changed conditions are encountered, the owner will receive the lowest reasonable bids with a minimum of contingency for unknowns
- Assists in the administration of the Defining Site Conditions (DSC) clause

The baseline statements are best described using quantitative terms that can be measured and verified during construction. A quality control program measuring the baselines during construction should be included in the report.

Extreme baselines should be avoided, particularly if they are not supported by the GDR test data or a geotechnical engineer's explanation based on previous construction in the area. If not avoided, then either the owner will pay considerably more for the project to the contractor who prices in the extreme baseline, or the owner will get a low bid from a contractor willing to gamble. Either way, it is or can be a costly choice by all parties involved.

3.7 SUMMARY AND RECOMMENDATIONS

For the trenchless contractor, understanding the subsurface conditions is critical for selection the appropriate equipment, means and methods. This chapter has provided an overview of what is involved in determining soil types and classifications that are appropriate for geotechnical consideration for a trenchless project. It provided a description of the major soil classification systems. It clearly stated NUCA's position on the principles of risk-sharing.

The following table provides a general guideline.

NOTE:
1) Utility tunneling in flowing cohesionless soils using compressed air ground stabilization was a regularly used method into the early 2000s before slurry microtunnel pipe jacking became the dominate tunnel method.
2) "Hard" granitic rock microtunneling has been successful for 15 years with the advent of face access MTBMs.

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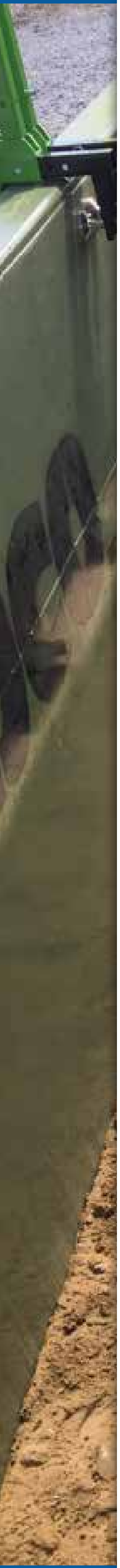
ABBREVIATIONS:

AASHTO: American Association of State Highway and Transportation Officials
CPT: Cone Penetrometer Testing
DSC: Differing Site Condition
FHWA: Federal Highway Administration
GBR: Geotechnical Baseline Report
GDR: Geotechnical Data Report
GIR: Geotechnical Interpretative Report
GPR: Ground Penetration Radar
ISRM: International Society of Rock Mechanics
OSHA: Occupational Safety and Health Administration
TGCS: Tunnelman's Ground Classification System
USCS: Unified Soil Classification System
UTRC: Underground Technology Research Council



CHAPTER 4
AUGER BORING

4.1 INTRODUCTION

Auger Boring (AB) is a trenchless technique used extensively throughout the United States, primarily for road, and railroad crossings. This method utilizes a simultaneously jacked casing though the earth while removing the spoil inside the encasement by means of a rotating flight auger. 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 jacking system located in the bore pit. The casing supports the soil around it as spoil is being removed. Generally, an AB bore is performed from an excavated bore pit to a receiving pit. AB's are not designed to be curved, either horizontally or vertically. The AB method has been around for over 80 years, having started in the 1930s in the coal mining industry.[1] Major developments continue to expand the technical envelope to bore larger diameters and longer distances while achieving higher accuracies in a wider range of ground conditions.

4.2 AB METHODS

The AB method is traditionally classified as:

1. Track method
2. Cradle method

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Photo courtesy of Barbco

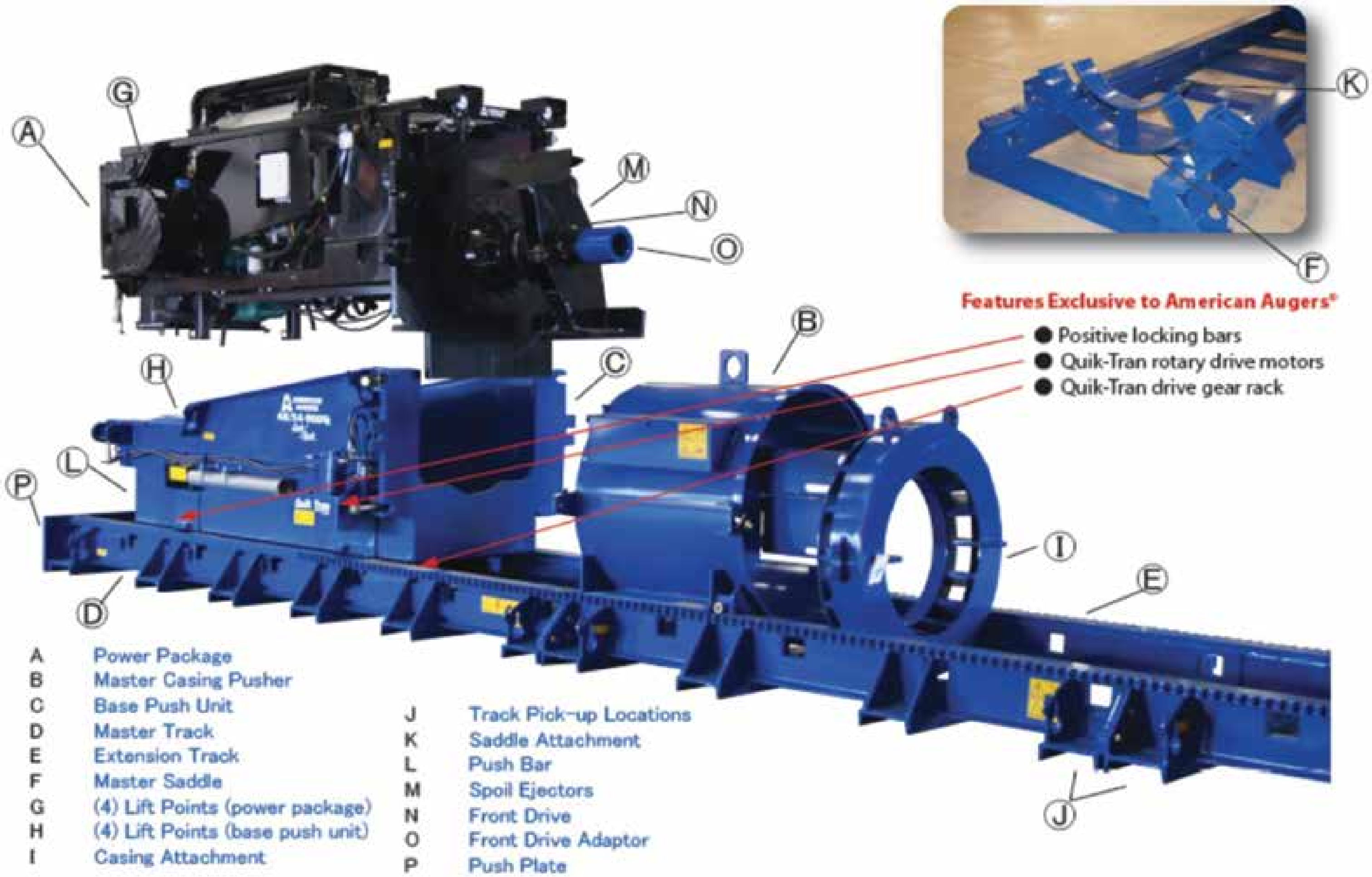

Figure 4.1 Track type auger boring machine. ***(Courtesy of American Augers)***

4.2.1 Track Type AB Method

The track type auger boring machine includes the track system, the machine, casing pipe, cutting head, and augers as the major components. The optional components consist of a bentonite lubrication system, grade control head, casing leading edge band (also known as a clearance band, or overcut band), and water level or electronic indicator. There are also other options available to use with AB machines such as, guided boring machines (GBM) and boring machine tunnel attachments. A track type auger boring machine is shown in Figure 4.1.

The two main factors that affect auger boring are the torque and thrust. Every effort is made to minimize torque and thrust. The torque is created by the jacking system, which can be pneumatic, hydraulic, or an internal combustion engine through a mechanical gearbox. The torque rotates the auger that, in turn, rotates the cutting head. The casing, with torque plates attached between machine and casing, remains stationary as it is jacked through the soil. Hydraulic thrust cylinders located at the rear of the boring machine create the jacking thrust force. One end of the cylinders is attached to the machine, while the other end is attached to lugs (push bar dogs) that lock into the track system.

If either the required torque or thrust exceeds the machine's capacity, then all forward advancement is halted. Since actual conditions to be encountered are never known or visible, every effort should be made to minimize torque and thrust, and both should be closely monitored throughout the auger boring operation. When either torque or thrust increases substantially, it is an indication of developing problems and should be investigated immediately.

For a project to be successful, soil test borings, unit weight of soil, soil classification, groundwater level determination, standard penetration test (SPT) value, unconfined compressive strength of soil, etc. should be available. Even with this information, changed conditions and/or obstacles may be encountered during the auger boring process. Therefore, all efforts should be made to ensure that unexpected conditions can be handled safely.

The basic steps in a typical track type AB project are as below.

4.2.1.1 Jobsite Preparation

The jobsite should be surveyed for overhead power lines and other obstructions, water drainage problems, job access, and working space. All utilities should be contacted, located, marked, and if necessary, exposed to positively identify and locate any potential underground obstructions to the bore. This is especially true beneath the roadway to be crossed. Underground utility lines damaged beneath the roadway by the boring equipment will most likely result in open cutting of the roadway. An open cut on a roadway carries a substantial expense, poses a health and safety risk to the public and is a significant inconvenience to the public. In most instances, an entrance pit is required on the approach side of the bore. The site should allow adequate room for a boring pit excavation and subsequent stockpiling of the excavated material (in accordance with Federal Regulations OSHA Code, Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P), unless such material is to be removed from the job site. The site must have space for unloading, loading, and stockpiling equipment, plus room to set such equipment into the boring pit. Natural water drainage should also be a consideration. Precautions should be taken to ensure that in case of a heavy rain the pit, equipment, and materials are not flooded. The possibility of building a temporary drainage system to route the water flow around and past the construction site should be investigated.

Figure 4.2 The boring pit bottom filled with crushed stone.
(Courtesy of Brent Scarbrough Company)

Figure 4.3 The boring pit.
(Courtesy of Brent Scarbrough Company)

4.2.1.2 Bore Pit Excavation and Preparation

The contractor is responsible for constructing the pits and following the requirements such as pit construction, protection, barricades, traffic control, installation, and personnel safety in accordance with Federal Regulations OSHA Code, Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P. The beginning and the end of the bore should be located far enough from existing structures to allow adequate safety for both the structure and the public. The boring pit should be located enough distance from the roadway to provide for safe sloping of the pit walls. Shoring of the jacking and receiving pits need to be considered through means such as trench boxes, solder piles and lagging, and steel sheet piles, for the safety of the personnel and surrounding structures and utilities.

With all the utilities located and marked, excavation can begin. All cuts, grades, and slopes should conform to the construction plans. The boring pit should be offset slightly to the side of the bore line on the side that spoil exits from the machine. This allows more access for spoil removal. Any utilities encountered in the pit must be supported. If ground water is expected, an adequate dewatering system must be utilized.

The boring pit bottom must be firm enough to support the boring machine tracks, boring machine, casing, and the auger. In most cases, the pit must be excavated below grade and then filled with crushed stone (Figure 4.2). Usually 2-in. (50 mm) X 8-in. (200 mm) X 16-ft. (5 m) wood planks are placed parallel to the track rail under the track for support. A concrete floor may be placed if the bore is of considerable length, size, and duration, and/or soil conditions warrant it (Figure 4.3). In all cases, the track support must be set to the proposed line and grade of the bore.

The boring machine applies thrust to the back of the boring pit. To withstand this thrust, a backing plate should be installed against the back wall of the boring pit, square with the line of thrust. For low to medium thrust pressure, steel sheet-

Figure 4.4 Cast in place concrete thrust blocks.
(Courtesy of Midwest Mole, Inc.)

Table 4.1 Jacking pit construction considerations.

Jacking Pit	Remarks
Overall space	• Large enough to accommodate the AB machine, rails, thrust block, and hydraulic jacks.
Length	• Long enough for machine assembly, desired thrust block length, length of casing, and have room for joint assembly. (Always refer to manufacturer's manual).
Width	• Wide enough to allow easy spoils removal, provide enough workspace for workers and welders.
Bottom	• Filled with crushed stone or gravel to provide a firm subgrade to support the tracks, AB machine, casing, and auger. • Concrete floor is an option to help stabilize the track in certain ground conditions.
Track support	• Wood planking should be placed down the length of the track on both sides.
Thrust block	• Installed at the back side of the jacking pit and should be designed and constructed to withstand the maximum thrust force of the AB machine. • For uniformly distributing of the jacking force to the supporting soil. • Counterbalance the thrust exerted by the AB machine. • Steel sheeting, steel plate, timber, concrete.
Sump	• Pit sump for pumping is placed on the right or left rear, depending on the pit floor slope. If pit is level grade, install two pit sumps. One front right, and one left rear (facing receiving pit).

ing, a steel plate, or wooden timbers have been found to be adequate. Figure 4.4 shows the steel plates which are used as the thrust blocks. However, on long and large diameter bores, a concrete backstop is desirable in addition to steel plate. Care must be exercised to ensure that the developed thrust pressures do not disturb any existing utilities in or around the bore pit area. Each boring pit should be constructed as if it were to be in use for a much longer period of time than expected for the completion of the bore. This allows for a successful bore in the event of unexpected problems.

In most cases, an exit pit is required at the end of the bore. The safety requirements for an exit pit are the same as for the entrance pit. Unless absolutely necessary, no personnel should be allowed in the exit pit during the boring operation. The unexpected entry of the boring head into the pit can catch the person and cause serious injury. As the casing pipe approaches the exit pit, care should be taken to prevent collapse. Table 4.1 illustrates some comments related to the jacking pit construction considerations.

4.2.1.3 Equipment Setup

Many different types of equipment may be required on or around the boring site. Excavators and/or cranes are needed to dig the boring pit and set the equipment. A boring machine and tracks appropriate for the job are also required. Augers must be placed in the casing sections. A cutting head is installed on the front of the first auger section. The cutting head type selected for a particular project should be compatible with anticipated soil conditions.

The most critical part of the bore is the setting of the machine track on line and grade. If the alignment is not accurate when the bore is started, it is not likely to improve during the boring process. The master track is placed in the pit with the push plate against the backstop. The master track is aligned with the proposed bore and the machine is set on the master track. The push bar dogs are engaged in the rearmost holes in the track. The master casing pusher and the casing adapter are then installed. Water may be needed in the boring operation in some soil conditions to help facilitate spoil removal, for use with bentonite lubricants, and for monitoring grade with grade control head.

4.2.1.4 Lubrication System

Minimizing torque and thrust are the two major concerns of auger boring. Application of a lubricant to the outer skin of the casing reduces the friction between the casing and soil, which reduces the thrust requirements. There are two basic types of lubricants. One is bentonite, which is an expensive montmorillonite, colloidal material that when mixed with water becomes an excellent lubricant and sealant. It is the best type of lubricant for sand and porous soils because of its sealing qualities. The second type of potential lubricant are various types of polymers, which in some cases work better than bentonite. They ease the problem of separation when they get in the casing. Polymers like Baroid's "EZ Mud" work better than bentonite in certain types of soils, such as clay, where the encapsulation of the clay by the polymer reduces their sticking and balling tendencies. Either method improves the thrust capabilities in almost all types of soils. The complete lubrication system consists of a mixing tank, a pumping method, and a distribution system. The lubricant is transferred to a point of application near the leading edge of the casing through a steel pipe generally 0.5 in. (12 mm) to 1.5 in. (38 mm).

4.2.1.5 Water Level

The water level (Dutch level) is a device used to measure the grade of the casing as it is being installed. It permits the monitoring of grade by using a water level sensing head attached to the top of the leading edge of the casing. The level operates in the same way as the sight tube on a boiler. Both ends of the system are vented to ambient pressure. A pit mounted control and indicator board is located at some convenient point in the pit near the operator (Figure 4.5). A hose connects the bottom of the indicator tube to a water pipe running along the top of the casing. Water is used to fill the system. The level of water in the pit indicator will then show the level of the sensing head (water block) at the end of the casing as it is pushed into the ground. It is important that the system is full so that an incorrect reading is not obtained.

As an alternative to the water level and laser systems, an on-target steering system is a designed control head can be used for monitoring and steering the AB machine. The on-target steering system's control head has two halogen lights (Figure 4.6), which are installed at the back side of the control head. Those two halogen lights project two parallel lines of lights that can be monitored from the jacking pit, and lateral position of the head can be controlled by them.

Figure 4.5 Water Level indicator board.

Figure 4.6 Grade control head examples.

4.2.1.6 Grade Control Head

The grade control head is used for making minor corrections in the grade. It can be used to make vertical corrections only. During the boring process, the actual grade can be monitored with the water level and necessary adjustments can be made with the grade control head. If a grade control head is used, then the leading end of the casing must be properly prepared. When water or polymer is injected in the casing to facilitate spoil removal, the point of injection is located behind the grade control head to prevent the water from contacting the excavation face. For this injection, a 0.5-in. (12 mm) diameter steel pipe, attached to the casing by welding strap bands to the casing, injects water through a 0.5-in. (12 mm) wide x 3-in. (75 mm) long slot approximately 30 in. (750 mm) behind the grade control head. On long bores, it becomes difficult to turn the steering rods for grade corrections. Hence, miniature gear boxes are used to bring down the effort to turn the rods. Also, the use of bearings in the hinges reduces the friction to the gear extent, which reduces the turning effort on the steering rods (for additional boring machine attachments see section 4.6).

4.2.1.7 Mechanical Line and Grade Control

The line and grade control head are used for making minor corrections both on line and grade. Controlling both line and grade is a patented technique and is done by using biaxial hinges. In the mechanical line and grade control head, the grade is monitored by using a water level while the alignment is monitored using a sonde transmitter. There are two sets of encased steering rods that are used for making the corrections for line and grade. Miniature gearboxes are used to reduce the effort to turn the rods for line and grade control on long bores. The leading edge of the casing should be properly prepared, and care should be taken to minimize torque and thrust and to keep the casing along the design alignment. To control the line, a proposed alignment is marked on the surface above the proposed bore. As the bore progresses, the offset is measured from the proposed line and the corrections made to compensate for it. The limitation of this method is that the surface above the bore must be accessible to take readings for controlling the line (for additional boring machine attachments see section 4.6).

4.2.1.8 Steering Shoes

Contractors use windows/coupons cut into the casing behind the leading edge to aide in controlling line and grade of the casing during installation. This methodology does require man entry into the casing to make adjustments. Proper PPE and other OSHA required safety items must be followed for this to be performed. These shoes are cut in the top, bottom, and sides of the casing. The size and placement are dependent upon ground conditions and the size of the casing being installed.

Figure 4.7 Steering shoes cut into lead casing.
(Courtesy of The Robbins Company)

4.2.1.9 Preparation of Casing

The casing should be of good quality and well prepared. In most cases, the lead casing is prepared in the yard prior to its transport to the jobsite. The proper sized auger is placed inside the casing and the cutting head is attached to the leading end of the auger. All casings should be measured to confirm proper length. It is recommended that all bores be done with a complete string of full-size auger sections. However, under conditions where the auger loading is light and the spoil moves easily in the casing, lead sections to full size auger can be followed with smaller sections. Smaller diameter augers are never used in the lead section of the casing. The step down of auger size should not exceed 1/3 of casing size in two steps down. (This is not a recommended practice in most ground conditions).

When the downsizing recommendation is neglected, problems normally occur. This decreases the efficiency of the boring as all the spoil is not removed from the casing where the smaller

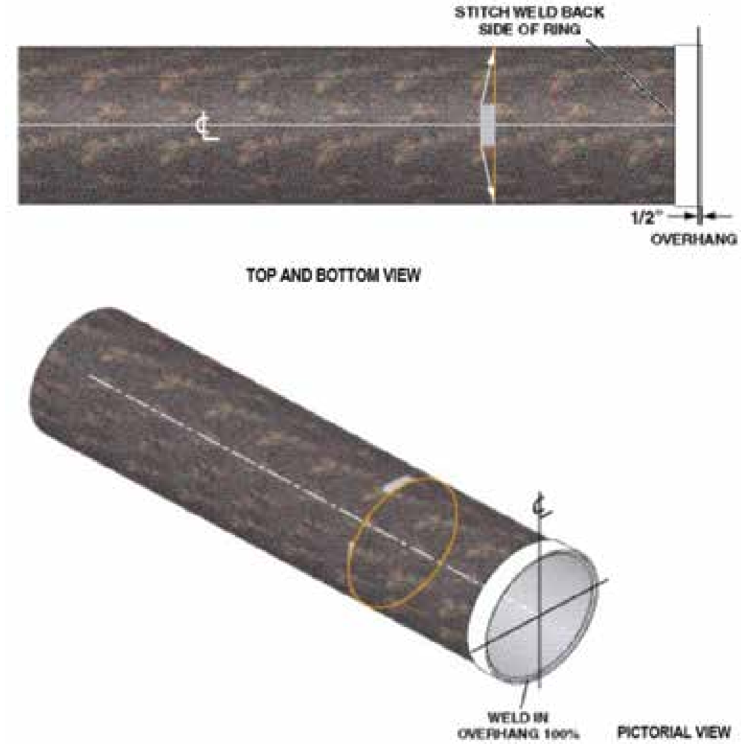

Figure 4.8 Banding the casing.

diameter auger is being used at the rate of excavation. Decreased efficiency requires more auger revolutions to remove the same volume of soil. These inefficient auger revolutions can then result it in the rotation of the auger without forward advancement.

Other factors in the use of smaller augers are bending and torque. The undersized auger creates bending that results in stresses in the auger stem. Also, the smaller auger will have more windup from the same torque loading than the full-size auger. Torque windup pulls the cutting head back toward the casing and could cause the wing cutters to contact the casing, further increasing torque, and causing even more damage.

Figure 4.7 illustrates a process referred to as banding the casing (overcut band or clearance band). The use of a partial band at or near the head end of the casing is recommended when boring in most soil conditions. The band compacts the soil and relieves pressure on the casing by decreasing the skin friction. The banding process is most effectively utilized in unstable soil conditions where wing cutters are not used. Banding in the absence of wing cutters allows the casing to be pushed forward without the borehole being over-excavated. This results in an increased soil compacting benefit, relieving the pressure on the following casing sections. Banding is also beneficial in rock or boulders as it strengthens the leading edge of the casing.

Wing cutters are devices attached to the cutting head that open and close. When the cutting head is rotated clockwise, the wing cutters open up to provide over-excavation of the borehole. The over-excavation of the borehole allows the casing to enter more easily because it minimizes the casing skin friction. **Wing cutters are used only in stable soil conditions and are never used with the cutter head inside the casing.** The wing cutters are adjustable, which allows control over the amount of over-excavation. The standard overcut is 0.75 in. (19 mm) when not using a steering head and 1 in. (25.4 mm) when using a steering head. When the cutting head is rotated counterclockwise, the wing cutters close up so that the cutting head can slide back inside the casing for auger removal purposes. The wing cutters must be set so as not to over-excavate at the bottom of the casing. If the wing cutters are not set properly this may cause the bore to drift in a downward direction.

Over-excavation of the bottom can be prevented by keeping the boring head centered. To keep the boring head centered, new or built-up augers should be used in the lead section of the casing. A worn auger in the lead section will allow the head too much freedom and the wing cutter pattern will be erratic. Some cutter head manufacturers offer a drift ring that is mounted to the back of the cutting head. This ring is far enough inside the casing to allow for shims to be placed between the ring and the casing to aid in centering the head to the casing. Shimming allows a more even over cut of the casing and reduced drifting of the line and grade.

Once the casing pipe has been placed into the track and the machine prior to commencing the boring operation, torque plates should be added on to the casing at the spring line on each side. The casing should be braced against the torque plates mounted to the dirt box to prevent the casing from rotating with the augers while they are turning inside of the casing. See equipment manufacturers operator's manual for further information on the placement of the torque plates and how to attach them.

4.2.1.10 Installation of Casing

Collaring is the first operation in beginning a bore. The objective of collaring is to start the cutting head into the ground without lifting the casing out of the saddle. This is done by rotating at low RPMs and using a slow thrust advance. When about 4 ft. (1.3 m) of casing has entered the ground, the engine is shut down, the saddle is removed, and the line and grade of the casing is checked. If the casing is not in line with the proposed bore, the casing is removed and the process repeated. The success of the bore depends to a great extent upon this process and the line and grade of the first section of the casing.

After the first section of the casing has been installed in the ground, the casing is cleaned by rotating the auger until all the spoil is removed. The machine is then shut down and the auger pin in the spoil chamber is removed. The machine is then moved to the rear of the track and is again shut down. Then the next section of the casing and auger are lowered into position. The augers at the face are aligned flight to flight, the hexagonal joint is coupled, and the auger pin is installed. Steel bar stock (scabs) set on edge are welded on the casing and aligned with the first casing by resting the scabs on top of the installed casing (Scabs may be welded to the installed casing at the bottom. It is better to have the bottom joint as smooth as possible to avoid plowing of material forward) and using minimum 4-ft. (1.2-m) straight edges along the top and the sides. Or, two 2-ft framing squares, one on each casing, can be brought together to determine if the casing is aligned. If the second casing is in line with the first casing and seriously out of line at the machine end, it means the first casing is misaligned and must be corrected or it will result in an unacceptable bore alignment.

Once the second casing is aligned with the first casing in the ground, the two are tacked together, and then welded completely. The joint is made by pushing the pipes together without a root gap (This will not meet AWS D1.1 specifications). If AWS D1.1 specification is required, then further conversation should be had between contractor and the engineer. The drive is then coupled to the auger and the casing is secured to the master pusher with torque plates. Water and lubricant lines, if being used, are added. The machine is then started, and the casing installed. The process is repeated until the bore is complete. Table 4.3 illustrates the recommendations on auger speed, rate of penetration, and other parameters that should be considered for different ground conditions.

4.2.1.11 Removal of Auger at the Completion of Bore

Once the bore is completed, the machine is shut down and the cutting head is removed. The casing is then cleaned by rotating the augers in the clockwise direction. The torque plates are removed to detach the machine from the casing and the augers are retracted until the coupling is well outside the casing. The auger section is uncoupled from the machine, as well as the other auger sections, and is removed. The machine is then coupled to the next auger and the process repeated until all the auger sections are removed. It is important to note that once the AB machine is pulled off the casing and the torque plates, the auger should not be rotated clockwise or counterclockwise as this is free boring and very dangerous for machine upset. The general rule is to dead pull the augers but at longer distances this creates issues with auger pins breaking. If possible, try to pull the augers. If the augers get held up by weld joints or cause hydraulic pressure to increase, rotate auger in counterclockwise direction for one revolution at a time and then dead pull again without rotating again. When the auger is rotated in reverse it is trying to push itself out of the casing and this method usually helps free up the auger. Rotating the auger in this way should be used only if necessary.

4.2.1.12 Installation of Carrier (Product) Pipe

After successful installation of the casing pipe, the carrier (also called product or distribution pipe) can be installed. Carrier pipe is installed by first attaching wooden skids or pre-manufactured casing spacers to the carrier pipe before assembly. The carrier pipe is then installed, one piece at a time, from either the entry or exit pit. It can be installed by pushing (by hand or with a boring/jacking machine), by pulling with a winch, or other methods. There are several methods of carrier pipe installation, each for a specific type of application and with different advantages and disadvantages as described below.

- For product pipe installation, where installation of the carrier pipe on precise line and precise grade is necessary, it is extremely important that the carrier pipe be blocked down to prevent flotation. This can be accomplished by using "differential" wood blocking banded to the carrier pipe to allow for adjustment of the grade inside the casing. Wood skids banded to the carrier pipe are typically used. It should be noted that a line and grade bore would allow spacers or regular blocking. Both methods protect the bells of the carrier pipe during installation and support the pipe off the bottom of the casing. Where glotation or "hammering" is a concern, "centering" spacers can be used to hold the pipe down tight. Skids can also be banded along the top of the carrier for the same effect. It is recommended that three or four skids, 2 ft (0.61 m) long, be evenly spaced around the carrier at each joint and in the center (for plastic pipe) of the carrier pipe (the amount of blocking per pipe joint should be verified by the pipe manufacturer and take into consideration if the annulus will be grouted and if the line will be filled with water prior to grouting). Ideally, the casing should be designed to allow approximately 1 in. (25 mm) of clearance between the top of the skids and the inside diameter of the casing. Casing spacers that center the carrier pipe inside the casing do not allow for "differential" blocking.
- After the carrier pipe is blocked inside the casing, grout or sand backfill can be used in the annulus of the pipe. Lightweight cellular grouts are an excellent product for this application. Compared with regular sand-cement grout, cellular grouts have a lower density that reduces flotation of the carrier pipe, and has superior fluidity, allowing for low installation pressure. Usually a compressive strength of 150-psi. (10,342 kPa) is adequate. If using regular sand-cement grout, care must be taken when grouting small pipes in a large annulus, because the heat of hydration of the sand-cement can be significant enough to cause damage to some plastic pipes. Grout also has some disadvantages. First, when using grout in the annular space, removal of the carrier pipe for future maintenance, if necessary, is almost impossible. Second, grout is far denser than most carrier pipe, even if the pipe is capped and filled with water. There is a very real potential to "float" the carrier pipe if grout is used (flotation calculations should be evaluated during the grout planning phase). However, lightweight, low-strength cellular grout can overcome most of these problems.
- When using sand, blocking the pipe down is important because sand and pea gravel do not have significant strength to hold the carrier pipe down to keep it from floating. This is especially true with larger diameter pipes. Carrier pipes sometimes displace sand and pea gravel even if the annular space between the carrier and casing is entirely full. This is especially true with larger diameter pipes. Smaller casings that do not allow personnel-entry are very difficult, if not impossible, to fill completely. Sand and pea gravel may damage the carrier pipe. Plastic pipe has become larger and lighter in recent years, making it more prone to floating and "hammering". Sand and pea gravel are abrasive, and the installation process involves using air to jet the material inside of the casing under high pressure. It is possible to have joints separate or actually blow holes in the pipe.
- Another method of carrier pipe installation is the use of pre-manufactured spacers or casing insulators. These spacers come in plastic, fiberglass, stainless steel, and carbon steel. They can also be coated in epoxy, rubber, and various other materials. Manufacturers provide recommendations for design and spacing of the spacers. If the carrier pipe is properly supported with spacers, sometimes there is no need for any fill inside of the annular apace between the carrier pipe and the casing. A further advantage of spacers is the ease with which the carrier pipe can be removed if future maintenance becomes necessary.
- It should be noted that in certain conditions, such as existence of high ground water, filling the annular space may be necessary, and the art of using grout becomes an issue.

4.2.1.13 Site Restoration

Once all the augers are removed, the boring machine and the track are removed from the pit, the desired utilities are installed through the casing, and the required connections are made. The boring pit and the receiving pit are then backfilled to restore the site to its prior condition. It is important that the pit foundation is properly restored to prevent any differential settlement.

4.2.2 Cradle Type AB Method

The cradle type AB method is suitable for projects that provide adequate room. The bore pit size is a function of the bore diameter and the length of the bore. This method is commonly used on petroleum pipeline projects where large rights-of-way are essential. With the cradle type AB methods, the bore pit design and construction are not as critical a factor as with the track type method because the boring machine and the complete casing auger system is held in suspension by construction equipment (pipe layers, excavators, and/or cranes) during boring operations.

The Cradle Type AB method offers the advantage that all work is performed on the ground level rather than in the pit. The bore pit is excavated several feet deeper than the invert of the casing pipe to allow space for the collection of spoil and water as the borehole is excavated. The method does not require any thrust structures; however, a jacking lug (Deadman) must be securely installed at the bore entrance embankment. This allows a winch cable with rated hook to be attached, allowing the machine to pull itself, and the casing, into the bore. Figure 4.8 illustrates typical cradle type auger boring equipment. Casing length for the entire bore is welded together on the bank at the jobsite. The complete auger and cutting head unit is placed inside the casing. The boring machine is attached to the end of the casing and the auger to the machine connection is made.

The total system is then suspended into position in the bore pit. The operator's station is often located on the machine, however the safest way to operate the cradle machine is through a wireless remote. This keeps the operator out of harm's way of the winch cable and eliminates the possibility of falling off the machine. The winch cable is attached to the Deadman and the cutting head is properly positioned at the initial point of entry for the bore. The desired line and grade are established by using appropriate survey equipment and making necessary adjustments with the construction equipment suspending the boring equipment system. Cradle type AB machines have become a lot safer with the installation of wireless remotes, making "no onboard operator required." Pipeline contractors prefer these because they are extremely fast, plus line and grade are not critical. A water level is still used with this method and does increase accuracy. Once the desired line and grade of the casing is established, the boring process is started and continued until complete. Depending on the length of the bore and the tolerance limits, it may be necessary to make one or more brief stops for line and grade checks, along with any other necessary adjustments. Some advantages of the cradle type method over the track type method include the following:

- The bore pit construction is simpler and safer since no one is required to enter the pit.
- All pipe fitting and welding can be accomplished at one time on the bank rather than in the bore pit at unpredictable intervals.
- The boring operation is continuous rather than cyclic.

The cradle type method also has some limitations when compared with the track type method. They include the following:

- This method is not appropriate for systems that require stringent line and grade tolerances. Hence, this method is not recommended for gravity flow piping systems but is more appropriate for pressure systems.
- The use of a steering head is not suited to this method.

Generally, water is injected into the casing at the leading edge to facilitate spoil removal. Lubricants may be used to coat the outer skin of the casing. It is important that special consideration be given to the foundation of the carrier pipe that is installed through the bore pit. Because the pit is excavated as much as several feet below the invert of the carrier pipe to provide a sump for the collection of spoil and water, this zone can be extremely unstable. If the foundation is not properly restored, failures can occur due to differential settlement.

Figure 4.9 Cradle auger boring machine.
(Courtesy of Precision Pipeline - Barbco)

4.3 APPLICATION OF AB IN VARIOUS GROUND CONDITIONS

AB can be used in a wide variety of soil types. However, in the case of a soil containing large boulders, this method cannot be used advantageously because the size of boulders and other obstacles this method can handle is limited to one third the nominal casing diameter. In the case of unstable soil, the relationship between the cutting head and the leading edge of the casing can be adjusted to prevent excessive spoil flow from above the bore path. Excessive spoil flow can cause a void between the casing and the borehole, which may lead to surface subsidence. Note, wing cutters should be removed when the cutting head placement is flush with the end of the casing or inside the casing. Table 4.2 illustrates the recommendations on auger speed, rate of penetration, and other parameters which should be considered for different ground conditions.

4.3.1 Wet, Running Sand

The cutting head should be run inside the casing about one to two times the diameter of the casing being drilled. For example, on a 24-in. (600 mm) bore, the cutting head should be 24 in. (600 mm) to 48 in. (1,200 mm) back from the end of the casing depending on the soil conditions. Because the cutting head is run inside the casing, the wing cutters cannot be used. An overcut band should be used. The rate of penetration should be fast while the auger is turned slowly. The pressure should be monitored to determine the rate of penetration. Long bores may require the use of bentonite and continuous installation. A steering head does not effectively work in this type of soil and should be avoided. The use of the sand auger is recommended. The sand auger is an auger section specially built for sandy situations. It has a smaller pitch than a regular auger section. A high degree of operator skill is required when boring in wet running sand. Verifying line and grade with laser reading at intervals usually should not exceed 80 ft and many contractors measure the alignment at least every 40 ft by removing the auger.[1]

4.3.2 Wet, Stable Sand

The cutting head should be run inside the end of the casing. An overcut band should be used. The rate of penetration should be controlled by the rate of spoil removal while the auger is turned at medium speed. Long bores require the use of bentonite and continuous installation. A steering head may be used in this type of soil but may not be effective. If a steering head is used, then wing cutters should also be used. When using a steering head, a strong light-weight sheet metal is used to cover the gap where the steering head hinges. The sheet metal should be welded to the front steering section only. A sand auger should be used for insurance against pockets of runny sand.

4.3.3 Dry Sand

If the sand is stable, then the cutting head should be run flush with the end of the casing. An overcut band is also advised. The rate of penetration should be fast while the auger is turned slowly. The use of bentonite is highly recommended. Long bores may require continuous installation. However, in most situations, the main problem with dry sand is the running action of the sand and subsequent void condition. To prevent this problem, remove the wing cutters and keep the head inside. The head will help hold back the sand, especially a sand head (full face with small openings for minimal spoil removal). Sand augers can be used on the first 10-ft section of auger with shorter pitch to control influx of spoil as well. The use of a steering head is not recommended

Table 4.2 Recommendations for different auger boring parameters.

Parameter	Wet Running Sand	Wet Stable Sand	Dry Sand	Dry Clay	Wet Clay	Small Gravel	Hard Pan	Large Gravel	Small Boulders	Soft Rock	Hard Rock	Road/Railroad Embankment
Auger Speed	Slow	Fast	Slow	Fast	Medium	Medium	Slow	Slow	Slow	Slow	Roller Cone or Disc Cuter Head	Cautious
Rate of Penetration	Fast	Fast	Fast	Fast	Fast	Fast	Medium	Low	Low	Low		Low
Cutting Head	Soil	Soil	Soil	Soil	Soil	Rock	Rock	Rock	Rock	Rock		Rock
Wing Cutters	No	No	No	Yes	Optional	Yes	Yes	Yes	Yes	Yes		Yes
Head Position	Inside	Inside	Inside	Flush	Flush	Outside	Outside	Outside	Outside	Outside		Outside
Bentonite	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No		Yes
Water Inside	No	No	No	Yes	Yes	Yes	Yes	No	No	No		Yes
Band	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes		Yes
Bore Continuous	Yes	Yes	Yes	Optional	Optional	Optional	Optional	Optional	Optional	Optional		Optional
Clean Casing	Pack	Pack	Pack	Clean	Clean	Clean	Clean	Clean	Clean	Clean		Clean
Pit Base	Concrete	Stone	Optional	Optional	Stone	Optional	Optional	Optional	Stone	Optional		Concrete
Backstop	Concrete	Concrete	Concrete	Steel	Steel	Steel	Steel	Steel	Steel	Concrete		Concrete

in this soil condition. The use of a sand auger is recommended. A high degree of operator skill is required. Figure 4.9 shows the settlement that can occur in sand.

4.3.4 Dry Clay

The cutting head can be flush or just ahead of the end of the casing. An overcut band is advised. The rate of penetration should be fast while the auger is also turned fast. Long bores may require bentonite. Wing cutters should be used on the cutting head to provide an overcut. The problem with clay is that it tends to ball up and stick to the turning auger. A water line should be used to add water to the inside of the casing to lubricate the clays and get the cuttings out. A steering head can be suitably used in dry clays.

4.3.5 Wet Clay

The cutting head should be run flush with the end of the casing. An overcut band is recommended. The rate of penetration should be fast while the auger is turned at medium speeds. The use of wing cutters is optional depending on the stiffness of the soil. Long bores may require bentonite. A water line should be used to add water to inside of a casing to lubricate the clays and get the cutting out. A steering head may be used in wet clays.

4.3.6 Gravel/Small Boulders

The cutting head should be just ahead of the end of the casing. An overcut band should be used. The rate of penetration should be medium while the auger is turned at medium speed. Long bores may require bentonite. Wing cutters should be used on the cutting head to provide an overcut. A steering head may be used in small gravel. If there is no binder with the small gravel, it can act more like dry sand. However, the properties of gravel may make it difficult to overcome the face pressure with the machine without potentially heaving the ground above the bore or maxing out the capabilities of the thrust of the machine.

Figure 4.10 Surface settlement in sand section.
(Courtesy of TTC Auger Boring School)

4.3.7 Hard Pan

The cutting head should be just ahead of the end of the casing. An overcut band should be used. The rate of penetration should be medium while the auger turns slowly. Long bores may require bentonite. Wing cutters should be used on the cutting head to provide an overcut. A water line is used to add water to the inside of the casing to get the cuttings out. A steering head can be used in hard pan. The problem with hard pan is that normally there are rock, stones, and boulders associated with it. When hard pan is discovered, consideration should be given to the casing size. A 30-in. (750 mm) diameter or larger in size should be used if rocks are present. This would allow worker entry if necessary.

4.3.8 Soft, Solid Rock (>4,000 psi UCS ASTM D7012)

The cutting head should be ahead of the end of the casing. A rock head equipped with wing cutters should be used. The auger speed should be slow with a steady pressure applied. An overcut band and a water lubricant line should be used. The problem with soft rock is that it is time consuming to cut the rock and this may wear out the teeth on the cutting head. Hence, it may be necessary to replace the teeth on the cutting head in middle of the bore by pulling out all the augers. The operator needs to be very careful while boring through rock because rock seams may cause the cutting head to lock, which might make the machine tip over.

4.3.9 Hard Rock (<4,000 psi UCS ASTM D7012)

Today there are better ways to handle rock conditions. To determine these rock conditions the following tests should be used for clarifying properties of the rock. These tests include the unconfined compressive strength tests (UCS) per ASTM D7012 and the Cerchar Abrasivity Index.

Roller cone heads can be fixed OD and collapsible, disc cutter heads, and hammer pilots and cluster hammers. The principle of the drill stem method is to create a pilot for a larger reaming head, fixed OD or collapsible. There is no steering control while using this method.

The first method for rock conditions is, a roller cone cutting head welded to a 10-in. or 12-in. drive casing pipe to bore the pilot. This method also uses a water swivel and thrust bearing, mounted at the boring machine end of the casing. While the boring machine turns the drive casing and applies a steady pressure on the face of the rock, the cutting head breaks the rock into fine cuttings. At the same time, water is pumped inside the casing through the water swivel. This serves two purposes. It cools the cutting head and washes the cutting off the face, while bringing it out to the installation pit around the casing being installed.

The second method is a disc cutterhead that uses pressure to break the rock. Each disc is a pressure point and they roll on the face of the rock and cause the rock to crack and chip. These disc cutters are recommended to run dry, but some water can be used to reduce the dust. There are features to help control line and grade on these units. The head and built-in thrust bearing are welded to the lead casing. With use of the main bearing, the thrust load is transferred to the casing and removed from the auger string. They have a tapered drive hex for easy auger connection. The augers are never pinned to the head so during the bore they can easily be removed to monitor line and grade. There are tunnel type machines that are used in conjunction with the Auger Boring Machine to do larger diameters, usually 48 in. up to 78 in.

The third method for rock conditions is used in conjunction with a Guided Boring Machine (GBM). First a pilot hammer is run through the bore path using line of sight locating for an accurate bore. The pilot machine is removed and a cluster hammer with a short pilot nose is installed with casing on the boring machine to follow the pilot. An air swivel mounted to the drive on the boring machine, with airway augers to install the casing and auger connections and welding the casing is as common as the conventional auger bore explained in section 4.2.1.10.

Recent improvements in cutter heads makes AB a practical option in various ground conditions. Table 4.3 summarizes the suitability of AB application in different ground conditions.

4.4 MAIN AB CHARACTERISTICS

4.4.1 Type of Pipe Installed

In AB, the auger rotates inside the casing as it is being jacked. Hence, there is a danger that it may damage any interior coating or liner that may be in the pipe. The standard casing material used with AB is steel. Presently, most of the railroad and highway specifications require the use of steel casing with AB. In AB, traditional welding pipe joints or weldless interlock pipe joints system can be used. Figure 4.10 shows the weldless interlock pipe joints system before and after locking.

Figure 4.11 The weldless interlock pipe joints system. ***(Courtesy of TTC Auger Boring School)***

Table 4.3 Summary of AB application under different ground conditions.[7]

Ground Conditions	Suitability	Remarks
Soft to very soft clays, silts, and organic deposits.	M-Yes	Steering system, if used with an AB machine, are not effective because of a lack of soil reaction forces.
Medium to very stiff clays and silts.	Yes	Soils must be displaceable for Pilot Tube Method.
Hard clays and highly weathered shales.	Yes	N<50 below per foot with Pilot Tube Method.
Very loose to running sands above water table.	M	Ground modifications may be needed to limit settlements.
Medium to dense sands below water table.	No	Dewatering is required for AB.
Medium to dense sands above water table.	Yes	Lubrication required for use with Pilot Tube Method.
Gravels and cobbles less than 2-4 in. (50-100 mm) in diameter.	M	Dependent on boring diameter; cobbles and gravel tend to bind the AB machine.
Soils with significant cobbles, boulders, and obstructions larger than 4-6 in. (100-150 mm) in diameter.	M	Dependent on boring diameter; should be limited to 33% of the excavated diameter.
Weathered to unweathered rocks and firmly cemented soils.	Yes	Approximately 25,000 psi (172 MPa) for an AB machine.
Yes: Suitable	**M: Marginal**	**No: Not recommended**

4.4.2 Pipe Size Range

The size of pipe or casing that can be installed by AB ranges from 12 in. (305 mm) to more than 72 in. (1,500 mm). For sizes less than 8 in. (200 mm), other trenchless technologies such as slurry method and compaction method are more suitable and economical especially where the line and grade are not very critical. For larger diameters where the line and grade are more critical, alternative techniques such as pipe jacking and tunnel boring machines provide greater accuracy and safety, and are more cost effective. The applicability of a certain size casing being installed by AB is very ground dependent along with the anticipated length of the crossing. It is recommended that engineers reach out during the design phase to local AB contractors around the project and discuss the ground conditions anticipated, to assist in determining the correct size casing, and to confirm the AB method is the correct method based from the contractors' experience in the area.

4.4.3 Bore Span

The AB method was initially developed to cross under a two-lane paved roadway with an average length of 40 ft (12 m) and a maximum length of 70 ft (21 m). Since then, several innovations have enhanced the equipment capability. The average bore now ranges between 175 ft (53 m) to 225 ft (68 m) with the maximum bore span being greater than 600 ft (180 m). The extremes of the limits for the length of a bore are very ground dependent and size dependent. It is recommended that an engineer should reach out to local boring contractors who have worked in the area of the project being designed to confirm the applicability of the method. They should ask the contractor what the reasonable size and length limitations are using the AB method in the anticipated ground conditions.

4.4.4 Disturbance to the Ground

With proper procedures, good equipment, and operator skill, surface subsidence is minimized. In stable ground conditions, the cutting head leads the casing by several inches and the wing cutters over-excavate by 1 in. (25.4 mm) greater than the outside diameter of the casing. Due to the stable nature of the soil, this does not result in any surface subsidence. In unstable soil, the wing cutters are not used, and the cutting head is withdrawn into the casing so that there is no over-excavation. In case of over-excavation, the void between the casing and the borehole should be pressure grouted. If the over excavation is greater than allowable tolerance, care should be taken when pressure grouting not to heave the ground above the bore. Finally, when the cutting head is placed inside the casing there is potential to heave the ground above the bore if the spoil removal is not matched to the jacking speed.

4.4.5 Area Requirements

The auger boring method requires bore pits at both entry and exit points of the bore. The size of the boring pit depends on the size and length of the casing pipe being used. Generally, the length varies from 30-40 ft (7.5-12 m) and the width is approximately 12-16 ft (3.6-4.8 m).

The working area requirement is a function of the depth of the bore pit and the method of excavation protection. The greater the slope required for the soil to be stable, the greater is the working area required. Thus, if sheet piles are used, less area is taken up by the pit. OSHA regulations must be adhered to for safety and all excavations. There must be sufficient room to allow for excavation and subsequent stockpiling of excavated material. If the material is to be removed from the site, adequate room for loading and hauling the spoil should be provided. Sufficient space should also be available for loading, unloading, and storage of materials and equipment. There must also be room to operate the equipment and to place materials and equipment in the bore pit.

4.4.6 Operator Skill Requirements

The auger boring method calls for a high level of operator skill. Because the success of an auger bore depends to great extent on the initial set up, the operator must know how to construct the bore pit for the soil conditions encountered and how to set up the equipment. A skilled operator would always know the location of the cutting head with respect to the leading edge of the casing. The operator should be able to interpret changing conditions in soil and take corrective actions. The operator should also know when to check the alignment and how to make line and grade corrections. Unknown conditions or changing ground conditions can greatly impact line and grade. Mixed face conditions can cause multiple issues with having a successful project and should be further discussed with industry experts to determine the correct methodology and tooling for the project.

4.4.7 Accuracy

The accuracy of installation will depend to a great extent on the initial setup and the operator's skill. An accuracy of +/-1% of the length of the bore is normally achieved. For projects that require a higher accuracy, an oversized casing is generally installed to provide maneuvering room for the carrier pipe inside the casing to obtain the specified tolerance. There are other options beyond simply over sizing the casing. These options include pilot tube guided bores, boring machine tunnel attachment, hydraulic controlled steering head, and manually controlled steering heads.

4.4.8 Cutterheads

Selection of a cutterhead has a direct relationship with subsurface ground conditions. The most used type of cutterhead is all-purpose head. These heads can be used in a wide variety of subsurface and soil conditions. An all-purpose head is not meant for granular soil without binder, or for solid rock greater than 4,000 psi UCS. The plastic limit for the soil should also be considered to prevent a flowing ground condition once boring commences. The proper selection of the cutterhead for the specific ground condition is a main factor in creating the balance

Figure 4.12 Various cutterhead configurations. ***(Courtesy of American Augers, Barbco, The Robbins Co.)***

Flat Face Rock Head
(Courtesy Barbco)

Modified Dirt Head
(Courtesy of Barbco)

Christmas Tree Rock Head
(Courtesy of Barbco)

T-Bar Dirt Cutting Head

T-Bar Rock Cutting Head

Roller Cone Head

Wing Cutters

Single Disc Cutter

Two Row Carbide Cutter

Roller Cone Head

Standard Disc Cutterhead,
6.5” Discs, SBU

Mixed Ground Cuttterhead,
6.5” Discs, SBU

Soft Ground Cutterhead, Tooling
in place of discs, SBU

Rock Cutterhead 9.5” Discs, SBU

Hard Rock Cutterhead, 11.5” Discs

between penetration rate and spoil removal rate. Reaching this balance is essential in a successful AB project.

For AB projects boring through dense and hard soil, or highly soft and squeezing cohesive soils, overcuts are needed to reduce the friction between casing and surrounding soils. Overcuts are also needed to make the casing movement easier during the jacking process. The overcuts can be provided by two means: adjustable wing cutters or welded collars (i.e., overcut band or clearance band). The standard overcut without using a steering head in soft soils is 0.75 in. (19 mm) per side (totaling 38 mm). With use of a steering head in hard rock, the standard overcut is 1 in. (25.4 mm) per side (totaling 2 in. or 50.8 mm).

Wing cutters are extra devices that attach to the cutterhead. Depending on the cutterhead's rotation direction (clockwise or counterclockwise), wing cutters can be open or closed. During clockwise rotation of the cutterhead, wing cutters will open and will provide active borehole overcuts. Furthermore, collars (i.e., overcut band or clearance band) can be welded onto the outside of the lead casing. During jacking of the casing, these collars will provide overcut capabilities to reduce the friction and ease the movement of the casing.

To aid in providing the required overcut, wing cutters need to be attached to a cutterhead, which is positioned outside of the casing. Over-excavation and its related problems, such as downward and lateral direction drift, can be reduced or prevented by following the standard amount of overcuts and keeping the cutterhead centered. This centering can further be accomplished by using drift rings that are added to the back side of the cutter head and extend into the casing. Figure 4.12 demonstrates different types of cutterheads.

Figure 4.13 BMTA (Bores Head) for large diameter bores.
(Courtesy of Leo Barbera)

Courtesy of Barbco

4.5 CAPABILITIES AND LIMITATIONS OF AB

The AB is an adaptable system and can be considered a most economical method compared to encasing of the carrier pipes in concrete under roadways or railways. The major advantages of auger boring are that the casing is installed as the borehole excavation takes place. Hence, there is no uncased borehole, thereby reducing the probability of a cave-in that could result in surface subsidence. Also, this method can be used in a wide variety of soil types, making it very versatile.

The AB method requires different sized cutting heads and auger sizes for each casing diameter, which calls for a substantial equipment investment. This method also calls for a substantial investment in bore pit construction and the initial setup. This method is not well suited for soils containing large boulders, because the size of boulders and other obstacles this method can handle is limited to one-third the nominal casing diameter.

The AB method is also not practical in a cohesion-less soil condition below groundwater, though AB is very good with groundwater level up to 1/3 of casing invert (except in running sand). To use AB in granular soil layers interbedded with cohesive soil, the cutter head need to be pulled back about three times the casing diameter to form a soil plug in the leading section.

For cutting soft rock with UCS up to 4,000 psi. (28 MPa), Christmas tree cutter heads with tungsten carbide teeth must be used.[4] For excavating harder rocks with UCS up to 25,000-psi. (170 MPa), cutter heads with disc cutters need to be used.

Initially, AB was used with maximum lengths of 70 ft (21 m), but with improvements and developments such as hydrostatic drives, lubrication processes, and stronger hydraulic thrust, AB capacity is significantly enhanced. The average excavation length using AB method is between 175-225 ft (53-68 m) with the maximum bore span of 600 ft (180 m). AB can achieve a larger excavated diameter with a large diameter Boring Machine Tunnel Attachment (BMTA) (Also known as Bores Head - Figure 4.12).

4.6 SMALL BORING UNITS (SBU)

The Small Boring Unit (SBU) was introduced in 1996 by The Robbins Company, who pioneered the technology for an excavation project in Pennsylvania of a 42 in. diameter tunnel in hard rock. An SBU consists of a small diameter cutterhead and thrust bearing assembly, which connects to the lead casing. The main purpose of using an SBU in AB projects is to overcome hard rock ground conditions. With use of this attachment, AB machines can excavate through medium to hard rock, as well as mixed ground conditions under railroads, highways, and streams. The overall process is very similar to conventional auger boring. The SBU can efficiently cut rock with UCS greater than 4,000-psi. (28 MPa), and has been used to cut rock up to a UCS of 25,000-

Figure 4.14 Kerf Cutting Pattern in Rock Face.
(Courtesy of The Robbins Company)

SBU-M

SBU-RH

SBU-A

Figure 4.15 Typical SBU units. (Courtesy of The Robbins Company)

psi. (172 MPa).[4] The range of the casing installation while using SBUs is between 24-87 in. (609-2,210 mm) in diameter.[5] However, while SBUs are the efficient solution in rock conditions, SBUs are not recommended for use in soil, sand, or conditions below the water table. There are four different types of SBUs that have been introduced to the market:

1. Small Boring Unit-Auger, (SBU-A)
2. Small Boring Unit, Motorized (SBU-M)
3. Small Boring Unit, Remote Control (SBU-RC)
4. Small Boring Unit - Rockhead, (SBU-RH)

The SBU-A and SBU-M are based on AB-powered systems, and as they are the most common SBU products, they will be explained in this manual. The original auger-driven Small Boring Unit (SBU-A) attachment is the first SBU to be used with all standard AB machines, and to install casing within the range of 24-87 in. (609-2,210 mm) in diameter at maximum drive lengths of approximately 600 ft (183 m). A full face auger is connected to the SBU-A through a standard hexagonal shaft and is used to remove the spoils behind the SBU-A. The shield of the SBU-A is normally welded to the lead steel casing. By jacking the casing forward, the thrust forces are transferred through the casing to the SBU cutterhead via the main bearing.

With this attachment, there will be no thrust load transmitted through the auger string while excavating hard rock. The thrust force and rotating motion of the cutterhead turns the disc cutters against the rock face, causing the face to fracture in a kerf pattern (see figure 4.14). The fractured rock chips are transferred behind the cutterhead with abrasion-resistant bore scrapers that feed into the full-face auger and then to the jacking pit via the auger string.

SBU Attachments with bore diameters of 30 in. (762 mm) and larger are fitted with manually adjustable stabilizer pads. The stabilizers have two purposes: 1) The stabilizers hold the cutterhead steady in the bore to prevent movement and oscillation, which in turn reduces disc cutter wear. 2) The stabilizers can be adjusted to assist in minor steering corrections. The boring process must be stopped frequently, and augers are removed to let the personnel enter the casing. Workers can manually adjust the stabilizer pads and/or Steering Shoes as noted in section 4.2.1.8 to maintain the designed bore alignment.

Table 4.4 SBU-A vs SBU-M.

	SBU-A	SBU-RC / SBU-M
Casing diameter	24 to 87 in. (610 to 2,210mm)	36 to 87 in. (914 to 2,210 mm)
Drive length	600 ft. (183 m) up to 54"	>600 ft. (>183 m)
Accuracy	Acceptable	Very Good
Auger	Full Face	Smaller Size
Steering System	Manually adjustable stabilizer pads	Laser-based steering, articulating front shield
AB powered system	AB machine	Hydraulic/Electric motor using VFD
Personnel entry	By removing the augers	No need to remove augers

Table 4.5 Suitability of SBU method under different ground conditions.[7]

Ground Conditions	Suitability	Remarks
Weathered to un-weathered rocks	Yes	Up to 25,000-psi. (172 MPa) UCS rock
Soft to hard cohesive soils	No	Not a suitable technology for full-face soil applications
Medium to dense sands and silts above groundwater	No	Not a suitable technology for full-face soil applications
Very loose to loose sands and silts above groundwater	No	Not a suitable technology for full-face soil applications
Sands and silts below groundwater	No	Not a suitable technology for full-face soil applications
Cobble and gravelly ground	M	Required to be in cemented matrix, free of groundwater
Mixed face of soil and rock	M	Transition from soil to rock and then back should be limited to slightly cemented soil with limited cohesion
Yes: suitable	**M: Marginal**	**No: Not recommended**

The Motorized Small Boring Units (SBU-M) and Remote Control Small Boring Units are designed to install steel casing within the range of 36 to 87 in. (914-2,210 mm) diameter. When using the SBU-M or SBU-RC, small size invert augers or vacuum suction tubes are used instead of full-face augers. This will allow for the use of constant laser-based steering using a hydraulic articulating forward shield body. An electric drive motor and gear reducer are used to drive the cutterhead through a torque limiting device to protect the drivetrain. In larger diameters, the SBU-M and SBU-RC can achieve longer distances when compared to the SBU-A, because they do not require torque to spin a full-face auger. The small diameter auger or vacuum system is used to transfer the spoils from behind the cutterhead to the jacking pit. Figure 4.15 presents the different types of SBU. Table 4.4 illustrates the comparison of SBU-A and SBU-M.

Subsurface ground conditions play an important role in SBU performance. If the ground condition is very hard with UCS of >25,000 psi. (172 MPa), or very soft with UCS of <3,000 psi. (21

MPa), then the SBU's progress may be very slow. In the case of very soft ground, the conditions can cause a clog of the cutterhead. To combat this tendency the rotation and speed of advance need to be slowed down specifically when groundwater is present. In very hard rock, higher thrust loading is required, which can again make the process slower. Table 4.5 lists the suitability of SBUs under various subsurface ground conditions.

4.7 SAFETY ISSUES

One of the main issues regarding AB safety is the AB machine overturning and related onsite problems. There are several safety features that existing machines already have, and additional safety features have been added to newer AB machines to make the process safer, they are as follows:

- **Tilt sensor kill switch**
  - The main purpose of a tilt sensor kill switch is to prevent the overturning of the AB machine. The kill switch works by sensing the upcoming rapid torque and switching off the machine while also pulling back the clutch. This is especially useful if the cutter head encounters obstructions like rock, boulders, and other buried items. When these obstructions are encountered, the torque that is applied by machine can cause the AB machine to overturn, which can create significant damage including loss of life of the personnel.
- **E-Stop kill switches**
  - The main purpose of this switch is to allow the workers the ability to kill all functions of the AB machine in case of emergency. See the equipment operator's manual to find locations of the E-stops.
- **Remote control**
  - This option aims to provide the operator the ability to stand in a safer location away from the AB. unit. The AB machine remote control has functions to remotely start-up and shutdown of the machine. The remote can also control thrust, dogs, head rotation, throttle control, and e-stop.
- **Operator platform relocation**
  - To provide increased safety for the machine operator in the event of overturn, the location of the operator's station is located to the back of the machine on some smaller AB machines. Some manufacturers have the operator platform located on the opposite side of the spoil ejection chute and some have the operator platform located on the spoil ejection side of the machine. There are various reasons for this related to safe operation of the machine.
- **Track and machine base widening**
  - To reduce the risk of the AB machine overturning, some manufacturers have increased the width of the track and machine base. This change enhances machine stability by lowering the center of gravity of the AB unit.[3]
- **Positive lock system**
  - Replacement of the hook rollers with the positive lock system increases the safety of the AB operation and crew. The positive lock system locks the machine into the track, but does so automatically, rather than manually via hook rollers. This eliminates the possibility that personnel might forget to set the hook rollers.
  - Operator-tethered kill switch
  - The operator-tethered kill switch is an additional feature to help the emergency shutdown switch. A lanyard is connected to the kill switch and then attached to the operator. If the operator gets off the AB machine without disengaging the auger drive of the machine, the lanyard will activate the kill switch and the AB machine will be shut down or go into low idle and disengage all operating functions of the machine.[3]
- **Spoil ejection chute's door flap**
  - A spoil ejection chute door flap absorbs the energy and deflects the direction of the spoils downward that might otherwise rapidly exit from the paddles in the area that personnel is working to remove the spoils from the ejection chute.
- **Hydrostatic drive machine**
  - Hydrostatic drive machines allow for the hydraulic pumps to include a pressure relief valve that will stall the machine at a set pressure to reduce the risk of machine overturn if an obstruction is encountered with the cutting head during the boring operation.

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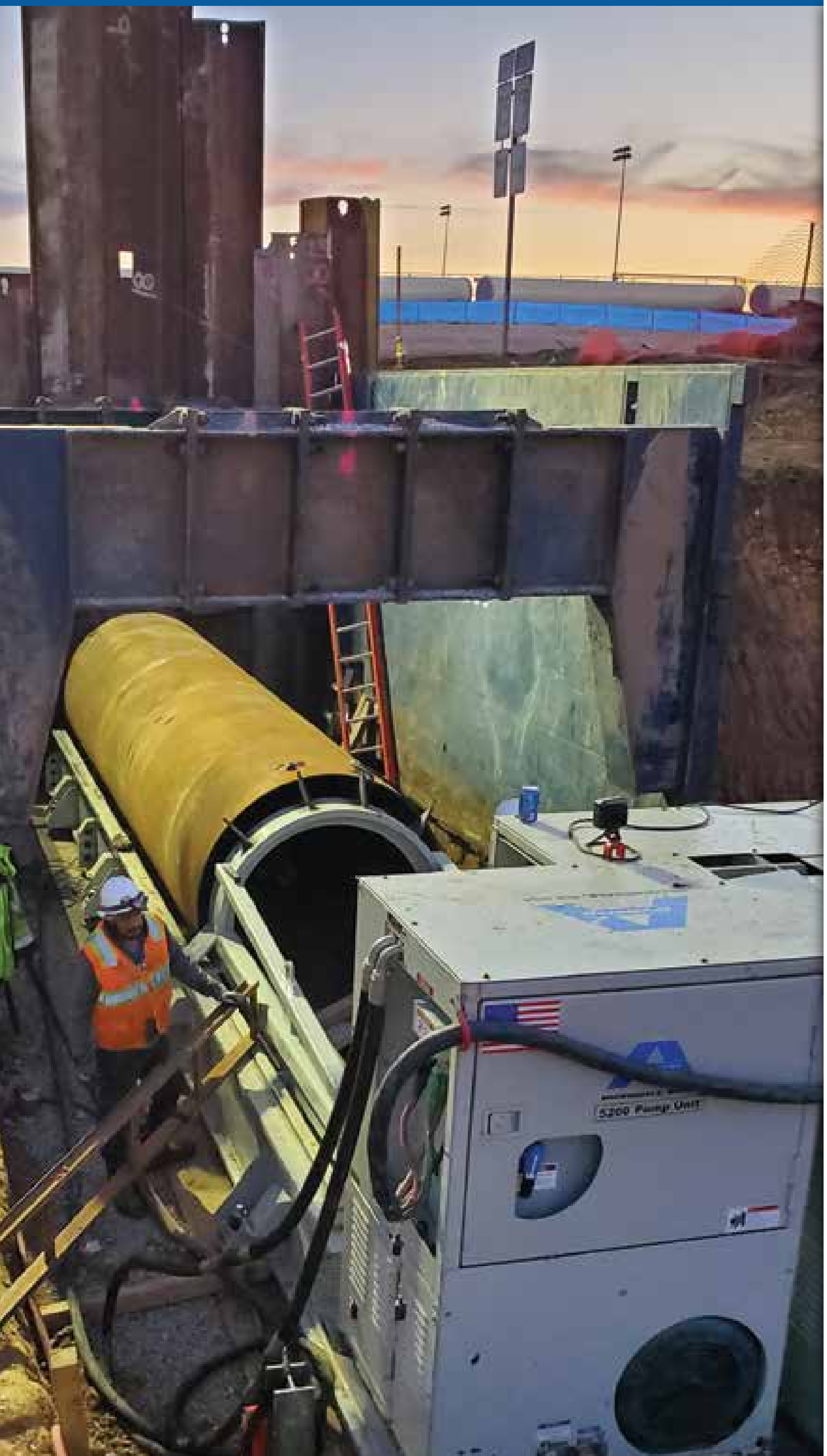

CHAPTER 5
Pipe Jacking & Utility Tunneling

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Photo courtesy of Akkerman

5.1 INTRODUCTION

Pipe jacking (PJ) and utility tunneling (UT) are trenchless construction methods that require workers inside the jacking pipe or tunnel. The PJ or UT operation is generally started from an entry pit. The excavation can be done manually or mechanically. The excavation method varies from the very basic process of workers digging the face with pick and shovel to the use of highly sophisticated tunnel boring machines (TBM). Since the method requires personnel working inside the pipe or tunnel, the method is limited to personnel entry-size diameters. Hence, the minimum diameter recommended for this method is 42-in. (1,075-mm) inside diameter. Even though it is theoretically possible for a person to enter a 36-in. (900-mm) diameter tunnel, it is very difficult for the person to work safely and efficiently.

Regardless of the method, the excavation is generally accomplished inside an articulated shield which is designed to provide a safe working environment for the people working inside and to support the ground to remain open for the pipe to be jacked in place or the tunnel lining to be constructed. The shield is guidable to some extent with individually controlled hydraulic jacks.

5.2 INTRODUCTION TO PIPE JACKING

Pipe Jacking (PJ) can be used as a technical term to describe an installation method, and it can also be used to describe a fundamental process used in other trenchless methods such as auger boring, microtunneling, etc. The basic process of pipe jacking is a method of person entry size underground pipe installation by jacking and adding pipe sections at the jacking pit. Pipe jacking was introduced into the United States of America at the end of the 19th century (Peckworth 1959).

PJ is a process which utilizes a tunneling operation but relies on using thrust boring and hydraulic jacks to push the pipes. Numerous innovations have occurred with the PJ method to achieve longer drive lengths with greater line and grade accuracy in a wider range of soil conditions. Innovations also include the introduction of new pipe materials and improving ground face-stabilizing shields. The jacking force is transferred through the pipe-to-pipe contact to the excavating face. In pipe jacking, workers need to be in the pipe for both face excavation and spoil removal.

5.2.1 PIPE JACKING METHOD

During the PJ operation, while the face is being excavated hydraulic jacks provide power to jack the pipes forward. The spoil is conveyed through the pipe to the jacking pit and removed. After each pipe segment has been installed the next pipe segment will be fitted in position for jacking. The typical components of a pipe jacking operation are illustrated in Figure 5.1.

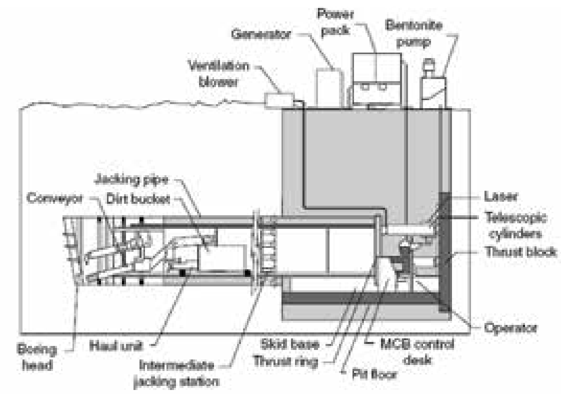


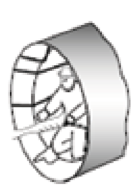
Figure 5.1 Typical components of a pipe jacking operation. (Iseley and Gokhale, 1997)

Subsurface instability evaluation, based on the principles and practices presented in Chapter 3 of this manual, will provide information to select the proper excavation method. If excavation face collapse is a possibility, soil stabilization solutions such as dewatering and grouting must be considered. As an alternative, closed-face earth pressure balance or slurry microtunneling methods may be suitable. Figure 5.2 illustrates the different types of excavation processes including hand mining, mechanical excavation inside a shield, and tunnel boring machines (TBMs).

Designing and constructing the proper jacking pit is critical for successful PJ projects, especially when it involves large diameter pipes requiring large jacking force.

5.2.2 PIPE LUBRICATION SYSTEMS AND INTERMEDIATE JACKING STATION (IJS)

Pipe lubrication systems and utilization of intermediate jacking station (IJS) have proven to significantly improve the productivity and capability of PJ. The pipe lubrication system is for applying bentonite or polymer slurry to the pipe's external sur-

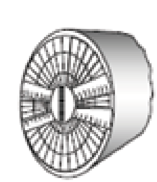


Hand shield: An open-face shield for manual excavation

Backacter: An open-face shield with a mechanical backacter

Cutter boom: An open-face shield with a cutter boom or road header.

Tunnel boring machine (TMB): A shield with a rotating cutting head



Earth pressure balance machine (EPBM): A full-face tunnel boring machine with a balanced screw auger to control the face pressure.

Figure 5.2 Tunnel boring machines and excavation techniques. (Iseley and Gokhale, 1997)

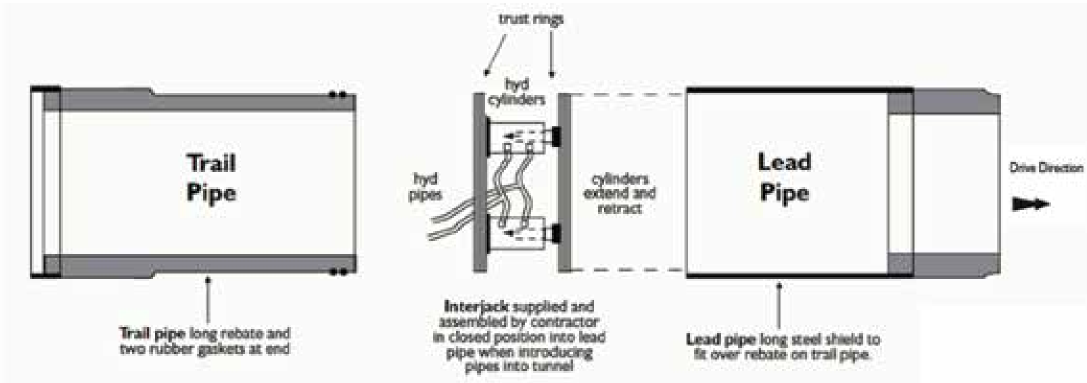


Figure 5.3 Intermediate jacking station. (Abraham and Gokhale, 2002)

Table 5.1 General pipe jacking procedure.

General Pipe jacking procedure	1. Ground excavation and driving shaft preparation.
	2. Setting up the jacking frame and the hydraulic jacks.
	3. Installation of laser guidance system in the driving shaft.
	4. Setting up the boring machine inside the driving shaft.
	5. Connecting the jacking push plate (thrust ring) to shield or TBM.
	6. Advancing the shield or TBM through the prepared opening in the forward shaft support structure.
	7. Beginning the excavation and spoil removal process and continue the advancement process until the shield or the TBM is installed.
	8. Retracting the jacks and push plate for providing a room for the pipe segment.
	9. Placing the first pipe segment on the jacking tracks.
	10. Connecting the push plate to the pipe and the pipe to the shield or TBM.
	11. Initiating forward advancement, excavation, and spoil removal. soil conveyance systems include: (a) wheeled carts or skips, (b) belt or chained conveyors, (c) slurry system, (d) auger system, and (e) vacuum extraction system
	12. Repeating pipe jacking cycles until the complete line is installed.
	13. Removing the shield or the TBM from the reception shaft.
	14. Removing the jacking equipment, IJS, and the tracks from the jacking pit.
	15. Restoring the site as required.

face and filling the annular space between excavation and the pipe, with the objectives of friction reduction and stabilizing the excavation. Such practices can commonly reduce the jacking force by 20 to 30 percent (Terzaghi, 1950). The technique of bentonite application varies from manual application before the pipe is inserted in the ground, to pumping bentonite from inside the pipe through small diameter nipples that are prefabricated in the pipe for this purpose. Recently, polymers have been successfully used to reduce the friction between the soil and jacking pipe. The advantage of using polymers is that only a small quantity of polymer is sufficient for the whole operation.

For projects with pipe diameters greater than 36 in., IJSs might be considered to redistribute the required jacking force on the pipes. IJSs include a steel cylinder fitted between two pipe segments with the hydraulic jacks placed around the internal steel cylinder periphery. After the main jacks reach about 80 percent of the design load, then the jacking force on the pipe behind the IJS is kept constant, and the IJS jacks push the forward section of the pipeline. There is no limit to the number of IJS that can be installed in a line. Interjacks (hydraulic ram system within a steel cylinder) are commonly used on long drives where jacking forces exceed the maximum capacity of the pipes or main jacks. Interjacks are installed during a drive and reduce the forces on the main jacks by pushing the pipes in front of the interjacks first, leaving the main jacks to push only the rear section of pipes. The IJS typically contain four to six jacks evenly spaced around the circumference. Figure 5.3 shows the IJS.

The IJS is assembled on the surface and lowered into the path of the pipe through the main jacking shaft. Many pipe jacking contractors install the shell of the IJS at regular intervals as a safety factor. Then, if they are needed, the IJS can be placed in the shell providing the additional power. If not needed, the investment is considered as insurance. Table 5.1 illustrates the general pipe jacking procedure. Table 5.2 illustrates Pipe Jacking (PJ) method.

Table 5.2 Pipe Jacking (PJ) method.

Pipe Jacking Method	Remarks
Face	• **Shield** o Conventional o Compressed air o Auger o Pressure chamber • **Spoil Disposal (haul length, soil type, and shield type)** o Skips on rails or wheels o Conveyors o Auger flights in tubes o Slurry pumping (Pressure Chamber)
Line	• **Pipes and casings are the main components.** • **Lining approaches** o Single pass (driven pipe is the permanent lining) o Double pass (temporary casing is first installed and then jacked out by the permanent pipes); rarely used. Is suitable when the finished pipe is to serve as a telecommunication multi-duct conduit. o Casing system (permanent pipe is laid through the outer duct and annular space is filled), for crossing work, under railways, highways and existing pipelines and canals. • **Pipes** o Concrete o Clay o Steel o Ductile iron o Glass-reinforced plastic (GRP) • **Load distribution ring** o Introduced at the joint faces for transferring the jacking load evenly between the pipes.
Jacking Pit	• **Set of rams (usually hydraulic)** • **Means of uniformly transferring the loads onto the end of the pipe.** • **Reaction elements (usually a block), commonly failed.** • **Anchor or tie.** • **Spoil removal equipment.** • **Monitoring tools** • **Pit size is depending on section size and the individual length of the pipe elements.** • **Circular shafts are preferred to the temporary pits since they can be converted to the permanent access points.**

A cutting edge or shield is installed to the pipe jack's leading end. Shield functions can be mentioned as, soil temporary support, cutting edge of hard faced, safe working place, mounting for cutting and face-stabilization equipment, monitoring location, and adjustment of directional attitude. Different types of the shields are illustrated in Table 5.3.

The drive length depends on the jacking capacity and the pipe's applied safe load. Jacking force can be affected by the weight of the pipe and, significantly, by the friction between the pipe and the soil. Also, if the mechanical excavation is used, pressure (jacking force) will be required for the excavation to occur.

5.2.3. MAIN FEATURES AND APPLICATION RANGE

Pipe Jacking (PJ) is differentiated from Auger Boring (AB) in the sense that pipe jacking requires workers inside the pipe. Although it is possible to install pipes up to 60 in. (150 cm) using the AB method, it should be classified as HAB if the excavation is done by a cutting head and the spoil is removed by augers or by any means that does not require workers inside the pipe.

Figure 5.6 illustrates the typical components of a pipe jacking operation. The process involves a simple, cyclic procedure of utilizing the thrust power of hydraulic jacks to force the pipe forward. In unstable conditions, the face is excavated simultaneously with the jacking operation to minimize the amount of

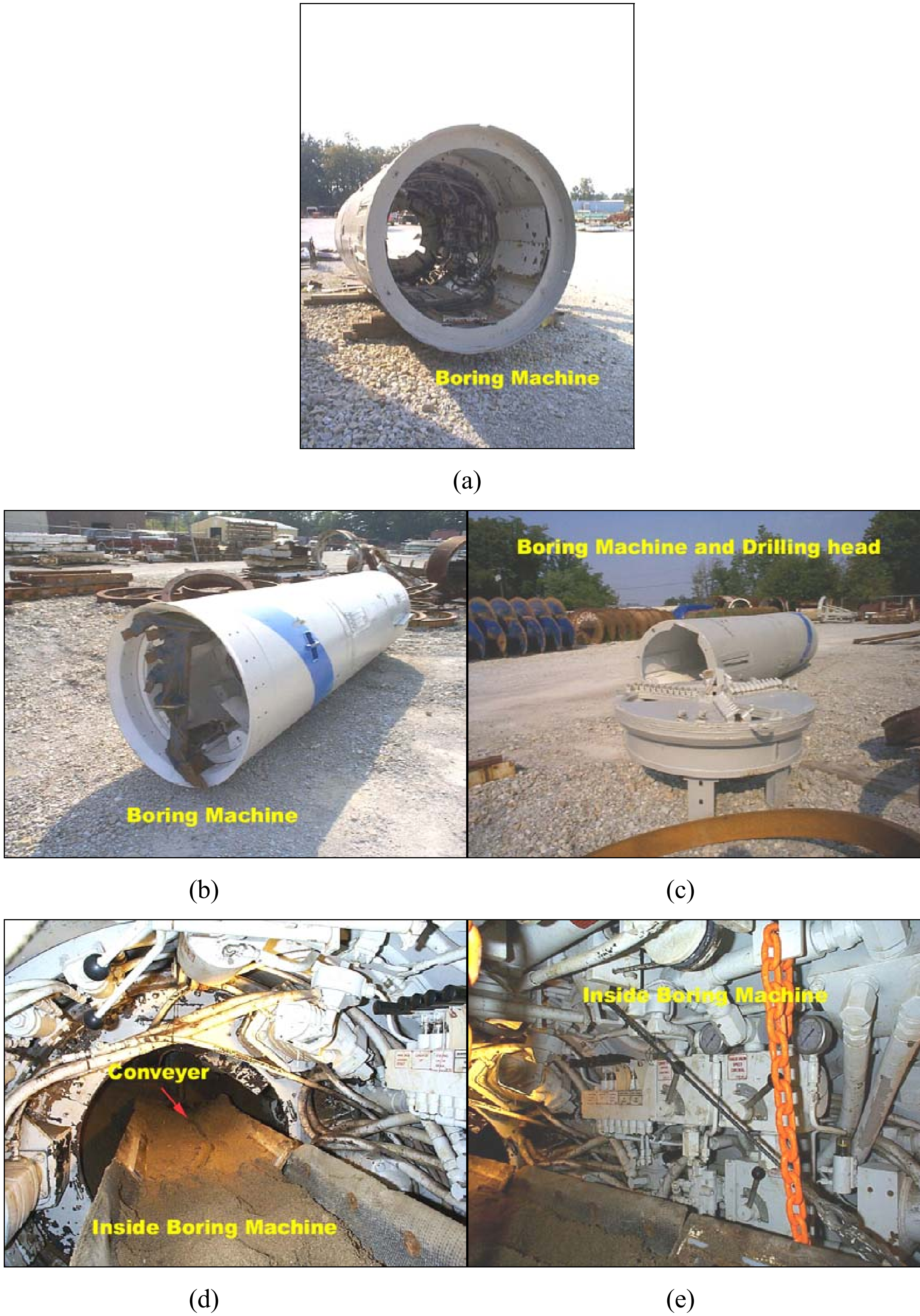


Figure 5.4 Pipe jacking tunnel boring machine. (Abraham and Gokhale, 2002)

Table 5.3 Shield types.

Shield types	Remarks
Conventional	• Allows direct access to the face or behind the face cutters. • Cheap, simple, and flexible. • Short drives, stable ground conditions. • Manual, mechanical, semi-mechanical, and rotary.
Compressed air	• Used in tunneling to apply a counterbalancing pressure for holding back the soil and ground water. • Avoiding the need of the operator to work in compressed air is possible through locking of only the face (glass panel or CCTV). • Skilled and experienced operator.
Auger	• Fully mechanical shields. • Remotely operated. • Full face rotary cutting head connected and driven by auger flights. • Work with different cutter heads • Small diameters, including microtunneling.
Pressure chamber	• Full tunnel boring machine (TBM) with a pressure chamber installed behind the front cutting head or disc. • Chamber is filled with water, slurry, or soil. • No access to the face and all operations are remotely.

over-excavation and the risk of face collapse. In stable ground conditions, excavation may precede the jacking process if necessary. Spoil is removed through the inside of the pipe to the jacking pit. After a section of pipe has been installed, the rams are retracted and another joint is placed into position so that the thrust operation can be started again.

The first step in any pipe jacking project is site selection and equipment selection as per the site requirement. The site must provide space for the storage and handling of pipes, hoisting equipment for the pipe, spoil storage, and handling facility, etc. If adequate space is available, a larger jacking pit is preferred so that longer pipe segments can be jacked, thereby reducing the total project duration. The jacking pit size is a function of the pipe diameter, length of pipe segment, shield dimensions, jack size, thrust wall design, pressure rings, and guide rail system. The space available at the site governs the selection of all the above components.

The jacking pit should be shored and braced unless it is very shallow and in stable ground conditions. It can be shored with timber, steel piling, or shaft liner plates. Due to the jacking forces required to push large diameter pipes through the ground, the design and construction of the jacking pit are critical.

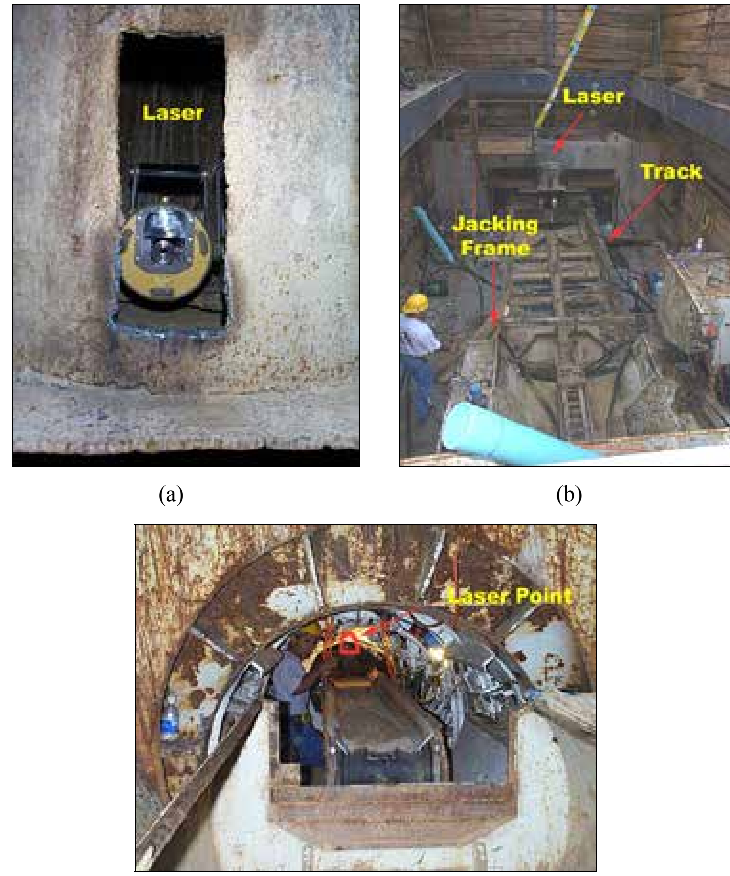


Figure 5.5 Laser guidance system for pipe jacking. (Abraham and Gokhale, 2002)

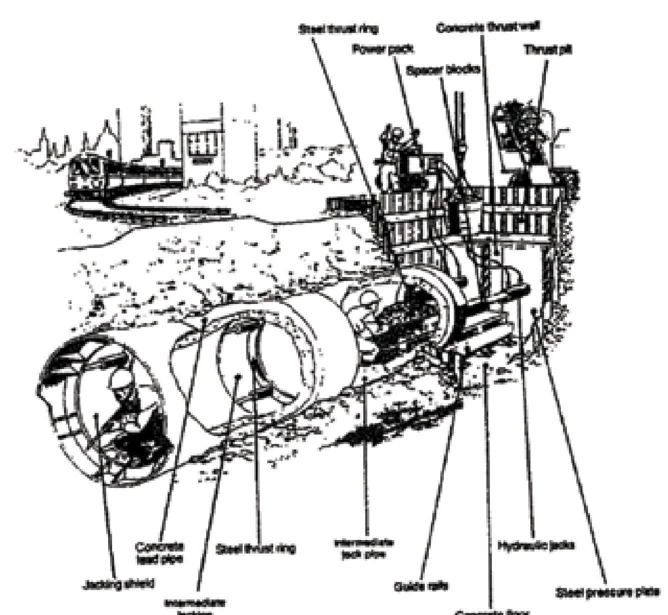


Figure 5.6 Typical components of a pipe jacking operation with shield. ***(Courtesy of U.K. Pipe Jacking Association)***

Table 5.4 Typical jacking force per linear foot of reinforced concrete pipe.

Pipe Outside Diameter (in.)	Jacking Force Per Linear Foot of Pipe (ton)	
	Sandy Soil with No Excavation in the Face	Hard Soil with Excavation in the Face
18	1.0	0.40
24	1.4	0.52
30	1.7	0.64
36	2.0	0.76
42	2.3	0.88
48	2.7	1.0
54	3.0	1.1
60	3.3	1.2
66	3.6	1.4
72	3.9	1.5
78	4.3	1.6
84	4.6	1.7
90	4.9	1.8
96	5.2	1.9
102	5.5	2.0
108	6.2	2.3

The pit embankment supports must also be properly designed and constructed.

The shaft floor must be designed to support all construction loads, including: the weight of the pipe, the thrust reaction structure/block, and the jacking loads which will be continuously exerted as the operation is being conducted. Preparation of the floor of the pit (i.e., soil, stone, or concrete slab) will be determined by the length, size, and/or duration of the job. The final alignment and grade will depend to a great extent on the initial setup. Therefore, it is advisable to set up a concrete slab foundation for large jobs that are likely to take a long time. The pit should have space for personnel to walk on both sides of the pipe. It is important that the pit should be dry, and dewatering provisions should be considered in the contract.

Once the jacking pit is ready, the guide rails are placed in the pit and aligned with the proposed bore. It is important that the guiding frame be aligned and supported throughout the operation since success of a pipe jacking operation depends to a large extent on the setup. Even though the shield is equipped with steering jacks, the jacks are for corrective action only and not for the establishment of line and grade. Proper alignment reduces the probability of developing deflections that result in high pressure point loadings and reduces the amount of thrust required.

Anticipated jacking thrust should be calculated based on the job conditions. This will include the penetration resistance, friction resistance between the pipe and the earth, and friction resistance force due to dead weight of the pipe. Table 5.4 presents the approximate thrust force per linear foot required to jack reinforced concrete pipe. The values should only be used as a guide, since actual jacking forces are dependent on the project specific conditions, such as operator skills, soil conditions, quality of the pipe, and the contractor's experience. It should also be noted that the jacking forces can be reduced with use of bentonite or the other suitable lubricants, as discussed above. The required thrust is applied using two, four, or six jacks, with the jacking thrust being balanced around the pipe centerline. The standard practice is to use two jacks, one on either side of the pipe centerline. Because the total jacking capacity of the available jacks often exceeds the compressive strength of the pipe, it is important that the total jacking force not exceed the pipe strength.

The thrust block, which is typically made of concrete, transfers the jacking force from the jacks into the soil. The thrust block should be designed to resist these forces. It is important that the thrust block be perpendicular to the jacking axis. If it is not, it may lead to uneven load distribution on the pipe that can result in problems in obtaining line and grade.

The excavation process generally takes place inside a shield. The excavation can be done manually or mechanically. Manual excavation is done by workers utilizing either pneumatic tools or simply picks and shovels. Mechanical excavation can be done by using a full-face cutting head like those used in the auger horizontal earth boring method or by a hydraulic backhoe mounted inside the shield. In either technique, the operator is located near the face so that he can readily see what is going on and take necessary action to encounter any situation that might develop.

The spoil is commonly removed by small carts. These carts are either battery-powered or powered by a winch cable. The spoil can also be removed by using small diameter augers or a conveyer belt system. Packing material is placed between the pipe joints to provide cushioning and flexibility. The most common and only industry-wide packing material used is 0.5 to 0.75 in. (12 to 19 mm) plywood. The main features and application range are illustrated in Table 5.5.

Table 5.5 Pipe Jacking main features and application range.

Pipe Jacking main features and application range.

Diameter

- Limited to person-entry pipe sizes.
- Minimum 42-in. outside diameter or 36-in. inside diameter.
- The largest is approximately 12 ft in diameter.
- If frictional forces can be overcome, theoretically there is no limit to the pipe size.
- Most common sizes ranging from 48 to 72 in.

Drive length

- Determined by available jacking thrust and pipe compressive strength.
- Minimize jacking thrust by:
    - providing an adequate overcut,
    - applying adequate lubrication,
    - maintaining accurate line and grade control,
    - using high-quality pipe products,
    - using IJSs.
- The longest drive length of pipe jacking project in the United States is approximately 3,500 ft.
- The most common drive lengths range from 500 ft to more than 1,000 ft.

Pipe type

- Must be capable of transmitting the required jacking forces.
- Steel pipe, reinforced concrete (RCP), FRPM, and PCP.
- A cushioning material (particleboard) should be used between the pipe segments to assist in distributing the jacking loads evenly.

Working space

- Need adequate space for storage, pipe handling, spoil, and the shaft.
- The jacking pit size is dependent on:
    - pipe diameter,
    - pipe segment length,
    - jacking shield dimensions,
    - jacking system dimensions,
    - thrust wall design,
    - pressure rings,
    - guide rail system.

Soil condition

- Most favorable: Cohesive soils.
- Unstable soil conditions require:
    - Dewatering.
    - closed-face machines.
    - earth pressure balance machines.

Productivity

- 33 to 60 ft. per 8-h shift with a four- or five-person crew
- Factors that can affect productivity:
    - Groundwater presence.
    - Unanticipated obstructions (boulders or other utilities).
    - Changed conditions (encountering wet silty sand after selecting equipment for stable sandy clay).

Accuracy

- PJ is highly accurate.
- With laser application, can expect accuracy of within an inch.
- A reasonable anticipated tolerance is ± 3 in. for alignment and ± 2 in. for grade.

Table 5.6 Applicability of pipe jacking method for different soil conditions.

Type of soil	Applicability
Soft to very soft clays, silt, and organic deposits.	Marginal
Medium to very stiff clays and silts.	Yes
Hard clays and highly weathered shales.	Yes
Very loose to loose sands (above water table).	Marginal
Medium to dense sands (below the water table).	No
Medium to dense sands (above the water table).	Yes
Gravels and cobbles less than 2 to 4 in diameter.	Yes
Soils with significant cobbles, boulders, and obstructions larger than 4 to 6 in diameter.	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils.	Marginal
Significantly weathered to un-weathered rocks.	No

5.3. INTRODUCTION TO UTILITY TUNNELING

Utility tunnels are differentiated from the major tunneling industry by virtue of the tunnel's typical sizes and use. Utility tunnels are used primarily as conduits for utilities rather than as passages for pedestrian and/or vehicular traffic. While methods of excavation for PJ and utility tunneling are similar, the difference is in the lining. With PJ, pipe is the lining, while in utility tunneling, either tunnel liner plates or rib and lagging become the lining. The lining for utility tunnels is considered to be temporary structures providing support until the utility is installed and the annular void between the utility and tunnel lining is filled with an adequate filler material.

As with PJ, excavation for utility tunneling takes place inside a specially designed tunneling shield. The excavation can be either manual or mechanical. Manual excavation is done by craftsmen utilizing either pneumatic tools or picks and shovels. Mechanical excavation can be done either by using a full-face cutting head like those used with auger horizontal earth boring or by a hydraulic mounted inside the shield. In either technique, the operator is located near the face so that he/she can readily see what is going on and can take necessary action to encounter any situation that might develop. In cohesive soil conditions, and in some instances where the shield cannot be removed due to not having a relieving pit, steel liner plates can be installed without the use of a shield. Experienced and properly trained tunnel operators are required when a shield is not used.

The spoil is commonly removed by small carts. These carts are either battery-powered or powered by a winch cable. The spoil can also be removed by using small diameter augers or a conveyer belt system. This method is differentiated from the pipe jacking method by the lining. In the pipe jacking method, pipe is the lining while in the utility tunneling either liner plates or rib and lagging from the lining. With liner plates and rib and lagging, the tunneling shield thrusts against the lining. However, the lining is not thrust into place but is constructed in place in the tail section of the shield. Hence, skin friction between the liner and the soil is not a factor. Construction of the liner in the tail section of the shield provides protection for the workers assembling the lining system.

Tunnel Liner Plates are prefabricated modular units utilized to construct a temporary circular lining for the purpose of encasing a utility. The liner plates are typically made of steel or precast concrete. However, steel liner plates are predominantly used for utility tunnels, while precast concrete liner plates have become cost effective and are used for larger tunnels. The steel tunnel liner plate is a prefabricated steel plate that has been cold-formed so that all edges are flanged to provide an integrated tunnel system. The plates are relatively thin, and curved longitudinally to provide the desired tunnel diameter when the liner plates are bolted together. The flanges of the liner plate project inward, and they are pre-punched with oversized bolt holes to facilitate the placing and fastening operation. The backs of the plates are deformed with corrugations to provide a higher degree of stiffening. Plates can be pre-punched to provide grout holes economically.

The width of a steel liner plate is the dimension parallel to the tunnel axis and perpendicular to the plate curvature. The length of a plate is the dimension from end to end as measured along the periphery of the tunnel. The radius of curvature is measured from the tunnel centerline to the back of the plate and corresponds with the outside dimension of the tunnel. The principle behind the design of the steel liner plates is to develop a unit that can be handled comfortably by one person in congested areas, like those commonly encountered at the heading of a tunnel or in the tail section of a shield.

The typical operation procedures for utility tunneling involves excavation being conducted at the tunnel face either manually or mechanically with the spoil being removed by carts, conveyers, augers, etc. or a combination of these systems. The tunneling

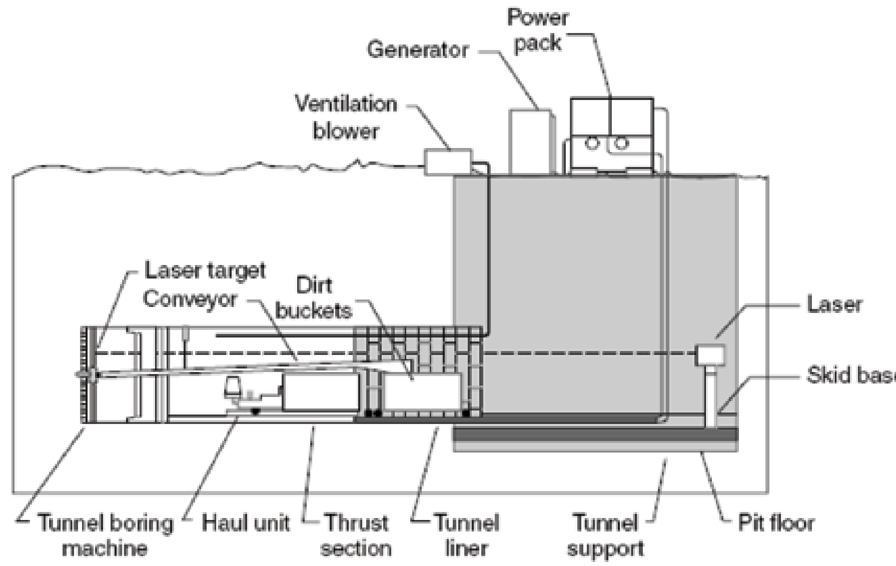


Figure 5.7 Typical components of utility tunneling system techniques (Iseley and Gokhale, 1997).

shield is hydraulically jacked forward as excavation is conducted. The amount of excavation that can occur before jacking is a function of the standup time of the ground conditions. The jacks of the shield thrust against the previously installed liner plates. After the shield has been pushed forward far enough for one or more courses to be placed, the jacking operation ceases, and the jacks are retracted so that the liner plates can be installed in the tail section of the shield. This operation is repeated until the whole length of the tunnel has been installed.

Steel ribs with wooden lagging are commonly installed as lining system for utility tunnels because of their simplicity, higher forward advancement rate, and relatively low cost. The most commonly used lagging materials are wooden boards that vary in length up to 48 in. (1,219 mm). However, steel liner plates, standard steel channels, corrugated metal sheets, etc. are also used as lagging material. The wooden lagging creates a bridging action transmitting the ground loads to the steel ribs. They must also be designed to resist the axial compressive force created as the shield is propelled forward. The minimum thickness used for wooden lagging is 3 in. (75 mm) while the most common size is 3 in. (75 mm) by 6 in. (150 mm). However, the size of wooden lagging depends on engineering design principles. The steel ribs are typically standard H-beams that have been cold-formed to the curvature requirements of the tunnel. The minimum size H-beams used are normally 4 in. (100 mm) with a minimum weight of 10 lb. per foot (15 kilogram per meter).

The typical construction procedure by a rib and lagging system is the excavation of the tunnel face at the leading edge to the shield and jacking forward of the shield utilizing the previously erected rib as the thrust reaction structure. The steel rib is erected in sections to form a complete ring perpendicular to the tunnel centerline axis and at a distance equal to the width of one course. The wooden lagging is then placed so that it forms a tight enclosure in the tail section of the shield. As the shield is pushed forward and the rib and lagging system is exposed to the tunnel excavation, the ribs are immediately expanded outward and upward to produce continuous contact with the supporting ground.

The process of underground construction and lining between two shafts is known as utility tunneling. Figure 5.7 shows the typical sections of a utility tunneling operation. Typical utility tunneling procedure is illustrated in Table 5.7.

Based on the method of excavation adopted at the front of the shield, pipe jacking and utility tunneling can be classified into the following categories:

a) Hand Mining:

Hand mining is accomplished by craftsmen utilizing either pneumatic equipment or pick and shovels. Excavation is generally accomplished inside a shield. The shield can be articulated or fixed. It is a very slow method but it has its advantages. This method is particularly effective in mixed face condition or soil with boulders. In the case of obstacles like big boulders or large tree stumps, workers can use jack hammers or power saws to cut through the obstacle and prevent over-excavation. Compressed air is often used to minimize the inflow of ground water and to support the face in case of unstable soils. In an articulated shield, line and grade corrections are accomplished by activating the hydraulic cylinders. In a fix shield, minor line and grade changes are accomplished by differential excavation in the direction of the desired change of direction. Hand carts are most used to remove the spoil from the face to the entry pit. In particularly long tunnels, a rail system with battery-operated carts is used to reduce the travel time and increase the efficiency of the operation. The major advantage of this method is that the initial cost in equipment is minimal. However, this method is

Table 5.7 Typical utility tunneling procedure

Utility Tunneling procedure

- **Pits construction.**
- **Field setup.**
- **Soil excavation.**
  - Hand mining.
    - Picks, shovels, or pneumatic hand-held tools.
    - Unstable soil conditions: protective shield is normally required.
    - Articulated shield: line and grade corrections accomplished by activating the hydraulic cylinders.
    - Fixed shield: minor line and grade changes are accomplished by differential excavation in the desired direction.
    - High underground water table: compressed air can be applied to prevent or minimize ground water inflow.
    - Slow and short drive process.
    - Requires minimum workspace; able to install linings as small as 30-in. diameter.
  - Open-faced mechanical excavation.
    - Special shields equipped with powered excavation devices.
    - Soil cutting devices; rotary cutter beams, a modified hydraulic backhoe, a rotary boom cutter, or any combination.
    - Unstable soil conditions or high underground water table; compressed air is normally used (with proper system design, the tunneling operator is not required to work inside the pressurized zone).
    - On-line adjustment of cutter head, manual handling of unexpected obstructions.
  - Closed-face TBMs.
    - Hydraulically or electrically driven rotary cutter heads or disc cutters.
    - Much improved face stability during soil excavation
    - Suitable for non-cohesive soils below water table.
    - High equipment cost, limited face access, and it can only be used to install circular tunnels.
    - Incorporating with a pressure chamber that provides a balance between the soil face pressure with the external water head and the mixed soil pressure inside the chamber.
  - Road Header Method.
  - New Austrian Tunneling Method (NATM).
    - Tunneling in rocks.
    - Allow the ground surrounding the tunnel to deform just enough to mobilize its shear strength.
    - Requires that the soil be strong enough to be self-supporting once the required deformation occurs.
    - Not recommended for shallow covers because soil is unable to arch properly.
- **Soil removal.**
  - (i) wheeled carts or skips, (ii) belt and chain conveyors, (iii) positive displacement pumping device, (iv) slurry system, (v) auger system, and (vi) vacuum extraction system.
  - The selection of appropriate soil removal system is dependent on; (i) the available space inside the tunnel, (ii) the manner of soil excavation, (iii) the mechanism of the face pressure balance, and (iv) the total tunnel length.
- **Segmental liner installation.**
  - Tunneling shield; the retracted jacking cylinders are extended and contact against the front profile of the new lining.
  - Tunneling through the rock formation; there might be no lining needed.
  - Prefabricated liner plate and rib and lagging systems. See Table 5.8.
  - Liner plates are typically made of steel or precast reinforced concrete.
    - The steel liner plates (widely used for utility tunnel applications) have flanged edges that allow the overlapping and bolting together of successive liner plates to form an integrated lining.
    - Concrete plates are bolted together through precast holes.
- **Direction steering.**
  - Theodolite (low cost, skilled operator, a light source, cannot be used for continuous monitoring).
  - Laser systems (immediate adjustment of any direction variation, sensitive to temperature, might become dispersed over a long distance of dusty air).
  - Combination of laser and theodolite can produce more satisfactory results.
  - Gyroscope may be used for curved tunneling
- **Tunnel (shield) advancing.**
  - Jacking cylinders at rear portion of tunneling shield.
  - After the shield has been advanced a certain distance, jacking cylinders are retracted to leave room at the rear of the shield for the in-situ installation of new segmental liners.

limited to relatively short drives that do not justify the investment in tunneling equipment to conditions that demand hand tunneling.

b) Open Face Shield:

In the open face shield method, a small backhoe or other excavation equipment is mounted inside the shield. This equipment is used to excavate the face of the tunnel. This method is faster than the hand mining method because the excavation per unit time is much greater. Often special equipment is designed that can excavate and convey the material to a conveyer belt on the rear of the machine. The conveyer then transfers the spoil onto a muck cart or some other system to convey the muck out to the entry pit. Compressed air is often used to minimize inflow of water and to support the face in case of unstable soil. Usually, the backhoe dumps the excavated soil behind it into a battery-operated cart that is used to remove the spoil from the face to the entry pit. The technique of differential excavation can be used to accomplish minor corrections in line and grade. The advantage of this method is that in case of major obstacles, the option of hand mining is always available. Hence the possibility of not being able to complete the tunnel is minimized.

c) Tunnel Boring Machine:

Tunnel boring machines are full face machines with a rotating cutter head used to excavate the soil. It usually has an articulated shield with hydraulic rams to make corrections in line and grade. The cutter head can be driven by electric or hydraulic motors. It rotates and excavates the soil that comes inside through small openings in the cutting head. From there, the spoil is transferred to the rear of the shield through conveyers that dump it into muck carts or convey it out of the tunnel or the pipe being installed. Alternatively, an auger system may also be used to convey the spoil to the entry pit. The line and grade is monitored by using a laser beam. The operator sits behind the cutting head, monitors the whole operation, and makes the necessary adjustments. Tunnel boring machines are very fast and efficient since they permit excavation and muck removal at the same time. Tunnel boring machines have been found to be cost effective especially in case of long drives. The major disadvantage of tunnel boring machines is the high initial cost, which sometimes does not justify their use for short tunnels. Tunnel boring machines can only be used to install circular tunnels.

d) Road Header Method:

A road header is either wheel- or track-mounted and consists of a boom with a spherical ball with teeth attached at its end. When the ball is rotated, the teeth cut into the soil or rock and thus excavation proceeds. A road header is a very flexible piece of equipment and can excavate very quickly. It is especially useful in rocky conditions and mixed face conditions. It is conveniently used inside tunnels because it permits considerable flexibility in maneuvering. In case of larger tunnels, more than one boom may be used to increase the speed of excavation. This method is convenient particularly for non-circular tunnels. Any type of mucking method can be conveniently used with this method. For larger size utility tunnels, the machine is mounted on a jumbo. The flexibility permitted by the use of the road header method allows the tunnel to be excavated in a single pass even for large tunnels.

e) Sequential Excavation Method (SEM):

The basic principle of SEM is to allow the ground surrounding the tunnel to deform enough to mobilize the shear strength of the soil or rock. Frequently, this limited deformation is achieved using wire reinforced shotcrete and steel ribs to form a flexible tunnel lining. SEM is quite adaptable to varying soil and rock conditions. However, the soil must be strong enough to support itself once the required deformation takes place. This method is not recommended for shallow covers since the soil is not able to arch properly. Soil settlement is an integral part of this method just as ground movement is an essential part of the tunnel support process.

5.3.1. MAIN FEATURES AND APPLICATION RANGE

In utility tunneling, the segmental liners in-situ form the lining, and a tunneling shield is applied. The tunneling shield is the only part that is jacked forward along the whole tunnel length. The jacking force requirement is fairly small and ideally there is no limit for the length of a single-pass tunneling process. This method is applicable for different soil conditions. Soil stability is fairly controlled due to personnel's accessibility to the front face during tunneling. Furthermore, this method has high accuracy in steering and the possibility of curved tunnel alignment. The original tunnel lining can apply as temporary support to allow final carrier pipelines to be transported and installed inside. The annular space between the liner and the pipe must be grouted. Utility Tunnelling's main features and application ranges are illustrated in Table 5.8. Table 5.9 shows the applicability of utility tunneling method for different soil conditions.

5.4. MAIN PJ AND UT CHARACTERISTICS

1. Type of Pipe or Liner Installed:

PJ is used for the installation of reinforced concrete pipe, steel pipe, fiber glass pipe, etc. Quality considerations should be given to pipe jacking projects. This includes squareness, roundness, smoothness, etc.

Liner plates are traditionally constructed of cold-formed steel plates or precast concrete. Precast concrete plates are used primarily on larger tunnels, and steel liner plates are commonly utilized for the smaller diameter utility tunnels. The possibility of utilizing plastic liner plates is also being investigated. The rib and lagging systems are constructed of steel H-beam cold formed to the desired curvature to function as the ribs. Wooden boards, steel liner plates, standard steel channels, or corrugated metal is used for lagging. Wooden lagging is the predominant material for utility tunnels.

Table 5.8 Utility Tunneling main features and application range

Diameter Range
• The tunnel liner materials are normally steel ribs with wooden lagging or steel plates. • There is no restriction on the final carrier pipeline material. • The minimum tunnel diameter recommended is 42 in. • For longer drives, the recommended diameter is 48 in. • Currently, the upper limit for utility tunneling is considered to be 132 in.
Productivity and Special Concerns
• Remote control of many elements of the tunneling process. • The installation of segmental liners always requires manual operation at the front face of the tunnel. • Slow and labor-intensive process. • The actual tunnel advance rate depends on: o soil conditions encountered, o soil excavation method, o soil removal method, o liner materials, o field coordination, o skill level of the tunneling personnel.

Table 5.9 Applicability of utility tunneling method for different soil conditions

Type of soil	Applicability
Soft to very soft clays, silt, and organic deposits.	Yes
Medium to very stiff clays and silts.	Yes
Hard clays and highly weathered shales.	Yes
Very loose to loose sands (above water table).	Yes
Medium to dense sands (below the water table).	No
Medium to dense sands (above the water table).	Yes
Gravels and cobbles less than 2 to 4 in diameter.	Yes
Soils with significant cobbles, boulders, and obstructions larger than 4 to 6 in diameter.	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils.	Yes
Significantly weathered to un-weathered rocks.	Yes

2. Size Range:

Since PJ requires people working inside the jacking pipe, this method is limited to personnel entry-size pipes and tunnels. This has been accomplished for pipes as small as 30 in. (750 mm). Even though it is theoretically possible for a person to enter a 36-in. (900-mm) diameter pipe, it is practically very difficult for the person to work in it. Hence the minimum tunnel diameter recommended by this method is 42 in. (1,060 mm) inside diameter. Pipe jacking has been accomplished for sizes as large as 132 in. (3,350 mm), and there is theoretically no upper limit for utility tunnels.

3. Jacking Span or Tunnel Span:

Theoretically there is no limit on the span that can be accomplished with these methods since the liner system is not pushed through the ground in utility tunneling and there is no limit on the number of intermediate jacking stations that can be installed in pipe jacking. Spans as large as 1,600 ft (485 m) have been accomplished without the use of intermediate jacking stations. However, jacking span is dependent on the soil and project conditions.

4. Disturbance to the Ground:

In stable ground conditions, with proper procedures and good skill, surface subsidence is minimal. In unstable ground conditions, geotechnical soil stabilization techniques are used to stabilize the soil. These stabilization techniques may include groundwater lowering with well points, or the use of cementitious chemical grouts. It is very important that the subsurface conditions be properly evaluated during the planning and design stage of the project. The risk of ground disturbance is also minimized with these methods because the tunnelers are always physically located at the face during excavation. Hence, they can immediately detect changed ground conditions and take necessary safety precautions if unstable conditions develop.

5. Area Requirements:

In pipe jacking, the jacking pit size is a function of the diameter of pipe, length of pipe segment shield dimensions, jack size, thrust wall design, pressure rings, and guide rail system. The space available at the jobsite will govern the selection of all the above components. If sufficient space is available, a big jacking pit is constructed so that longer pipe segments can be jacked, thus reducing the total time for the operation. When adequate space is available, jacking pits vary from 25 to 30 ft. (7.5 to 9 m) in length or diameter and for congested area, jacking pit sizes range from 10 to 25 ft. (3 to 7.6 m) in length or diameter. In congested conditions, reaction block is often constructed by excavating into the embankment opposite the jacking direction.

Utility tunneling requires a tunnel shaft primarily to serve as an access to the tunnel. It should be of sufficient size to accommodate spoil removal equipment and air handling equipment. The size of the access pits commonly ranges from 9 to 25 ft. (2.7 to 7.6 m) in diameter.

Utility tunneling requires a relatively smaller surface area compared to the pipe jacking method because of the compactness of the liner systems. Liner plates, ribs, and lagging materials require very little handling space compared to the equivalent pipe size. However, sufficient area should be available to accommodate the spoil removal system, air handling equipment, hoisting equipment, material storage, etc.

6. Operative Skill Requirements:

Although the pipe jacking and utility tunneling operations appear very simple, these methods are specialized operations and require a significant amount of skill and experience. The line and grade tolerances are almost always tight, and the contractor generally does not get a second chance to conduct the operation, as corrective actions are very expensive and time-consuming. The operator must be experienced enough to handle all situations that develop in the field.

7. Accuracy:

These methods are capable of installations to a very high degree of accuracy. Since lasers are used for controlling the line and grade, installations to an accuracy of within an inch are common. However, costs increase relative to the level of accuracy desired. So, designers should specify tolerances compatible with the required gradient.

8. Recommended Ground Conditions:

Stable granular and cohesive soils are the most favorable soils for these methods, whereas wet, unstable sand and swampy grounds are the most unfavorable soil conditions. Nonetheless, pipe jacking and utility tunneling can be executed on almost all types of soil conditions with necessary precautions. In the case of unstable soils, compressed air or slurry shield may have to be used to counterbalance the ground pressure. Soil containing large boulders is difficult because it requires frequent work stoppage until the obstacles are removed.

5.4.1 Major Advantages

Pipe jacking and utility tunneling has several primary advantages: These methods can be conducted in almost all types of soils. Further, a high degree of accuracy can be obtained with a minimum amount of skilled labor. Since the operator is located at the excavation face, he can see what is taking place and take immediate action for changing subsurface conditions. And the face can be readily inspected personally or by using video camera. When unforeseen obstacles are encountered, they can be identified and removed. Many options are available for handling the soil conditions.

Finally, only a small jacking force sufficient to drive only the shield has to be developed with utility tunneling. And large sections of prefabricated pipe do not have to be handled or stored.

5.4.2 Major Limitations

From a limitation perspective, pipe jacking and utility tunneling are specialized operations that require a lot of coordination. While these operations can be conducted on a radius, it is recommended that all direction changes be made at the shafts. The pipe and liners used for the operation should be strong enough to resist the jacking forces. Hence not all types of pipes and liner systems can be used for this operation. Finally, the liner systems are classified as temporary structures. Therefore, a carrier pipe or utility must be inserted through the tunnel liner and the void between the carrier pipe and the tunnel liner filled to provide adequate support.

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CHAPTER 6
MICROTUNNELING METHODS

6.1 INTRODUCTION

The microtunneling method (MT) is a remotely controlled pipe jacking process that provides continuous earth pressure and hydrostatic support at the excavation face. It is typically defined as a remotely controlled, guided, pipe-jacking operation that provides continuous support to the excavation face by applying mechanical or fluid pressure to balance groundwater and earth pressures with limited man entry. A laser is typically used to establish the desired line and grade on microtunnels shorter than 1,200 ft (365 m). For tunnels with curves or which are longer than 1,200 ft (365 m), an enhanced guidance system is used. The microtunnel method can install pipe with a high degree of accuracy, typically within a tolerance of 1 in. (25.4 mm) with respect to both the horizontal and vertical alignments. Gravity sanitary and storm sewer lines are the most common type of underground infrastructure system installed by microtunneling. However, MT can be used to install other underground utility systems as well.

Research and development efforts date back as early as the mid-1960's in the United States as shown in Figure

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Photo courtesy of Akkerman

Figure 6.1 MTBM R&D Efforts by D.H. Akkerman – Circa 1966.

6.1. Microtunneling was developed in Japan in the mid-1970s and in Germany in the early 1980s. The first use of microtunneling technology in the United States was in 1984. It involved using a machine with an outside diameter (OD) of 72 in. (1,800 mm) to install 615 ft (190 m) of pipe under Interstate 95 in Miami, Florida. The milestone for acceptance of microtunneling in the United States occurred in Houston, Texas, in 1987 as part of the River Oaks Project and the City's subsequent wastewater program. The project involved more than 3.8 mi. (6.1 km) of sewer system installed with microtunneling. The pipe diameters installed were 10, 18, and 21 in. (250, 450, and 525 mm).[1]

Microtunneling was designed and developed to work in soft, wet, unstable soils, but, as the industry matured, machines now work in almost all geotechnical conditions, from the softest and wettest sands and silts to rock, including conditions with gravel/cobbles and boulders. The spoil removal system for microtunneling is typically a slurry transportation system, however, some small encased screw auger conveyor systems are used.

6.2 MICROTUNNEL PROCESS

Figure 6.2 shows the microtunneling and jacking process. Depending on the geotechnical conditions being excavated; soft soil or soil with cobbles, boulders, rock, scraper cutterheads, cutting wheels or rock cutter heads equipped with roller cutter might be applied (see Figure 6.2)

Microtunneling requires jacking and reception shafts at the opposite ends of each drive. The microtunneling process is a cyclic pipe jacking operation. A microtunnel boring machine (MTBM) is pushed into the earth by hydraulic jacks mounted and aligned in the jacking shaft. The jacks are then retracted, and the slurry lines, power and control cables are disconnected. A product pipe or casing is lowered into the shaft and inserted between the jacking frame and the MTBM or previously jacked pipe. Slurry lines, power, and control cables are reinstalled in the new pipe or casing and reconnected. The pipe and MTBM are then advanced by the hydraulic jacks and this process is repeated until the MTBM reaches the reception shaft. Upon completion of the tunnel, the MTBM and trailing equipment are retrieved, and all equipment and support lines are removed from the inside the pipe.

Most microtunneling operations include the following systems: A hydraulic jacking system to advance the MTBM and pipe or casing string, a closed loop slurry system to transport the excavated spoils, a slurry cleaning system or plant to remove the spoil from the slurry water, a lubrication system to

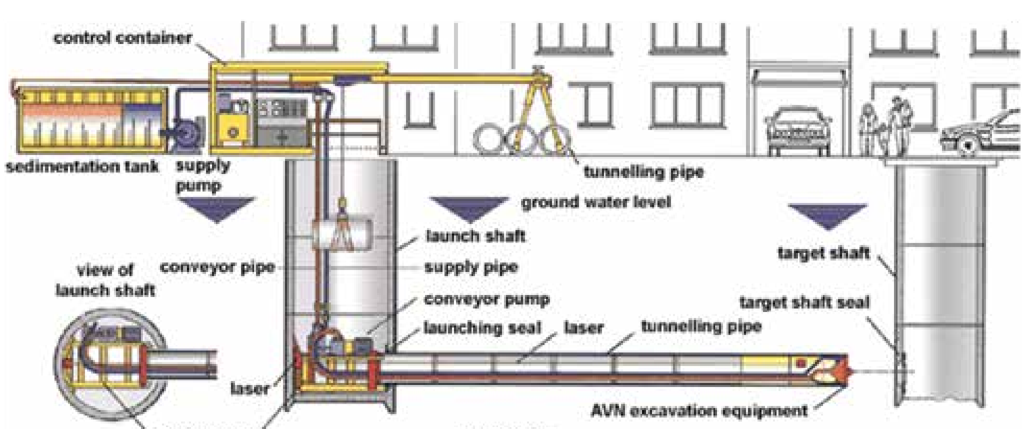


Figure 6.2 Microtunneling Jacking Process

Cutter head for soft ground.

Cutter head for mixed ground.

Cutter head for hard rock.

Figure 6.3 Microtunneling Cutter Heads.

lubricate the exterior of the pipe string during installation, a guidance system to provide line and grade control, an electrical supply and distribution system to power equipment, a crane to hoist pipe sections into the jacking shaft, and various trucks and loaders to transport material and spoil.

MTBMs have a rotating cutting head to excavate the ground material, a crushing cone to crush larger particles into smaller sizes for transport through the slurry lines, a hydraulic or electric motor to turn the cutting head, a pressurized slurry mixing chamber behind the cutter head, an articulated steering unit with steering jacks located the machine, various control valves, pressure gauges, flow meters, and a data acquisition system and gauges. Additionally, the MTBM has cameras to relay information to the operator and a target system for guidance control.

MTBMs are capable of independently counter-balancing earth and hydrostatic pressures. Earth pressure is counter-balanced by careful control of advance rates and excavation rates of spoil materials; the machine uses mechanical and slurry to maintain earth pressure. Groundwater pressure is counter-balanced by using pressurized slurry in the mixing chamber of the MTBM. Readings on various gauges are used by the operator to maintain earth and hydrostatic pressures.

The MTBM cutter head must be a slightly larger diameter than the machine and the machine must have an external diameter greater than jacking pipe diameter. The over cut developed between the cutter head, machine and pipe is filled with a lubricant as the tunnel progresses to help minimize jacking forces. The size of the of the over cut can and does vary depending on the geotechnical conditions, size, and length of tunnel and whether the tunnel is straight or has curves. Typical overcut ranges from 0.5 in. (12 mm) to 1.0 in. (25 mm) on the radius.

6.3 MICROTUNNELING GUIDANCE

From a guidance perspective, a laser is typically used to establish the desired line and grade on straight microtunneling alignments shorter than 1200 ft (365 m). For tunnels longer than 1200 ft (365 m) or tunnels that include curves, enhanced guidance systems are recommended. The MTBM operator actively makes steering corrections based on feedback received from the guidance system to maintain line and grade within the required tolerances.

6.3.1 Tunnel Laser-to-Target Guidance Systems

A tunnel laser is mounted in a position inside the jacking shaft so that it will not be affected by movement induced by the jacking frame. The tunnel laser must be calibrated and installed to the desired line and grade of the tunnel installation and aligned with the MTBM guidance system target. Modern MTBMs use active targets that provide instantaneous feedback to the operator such as horizontal deviation, vertical deviation, inclination, yaw, pitch, roll, and projected cutterhead position based on current MTBM steer. MTBM heads equipped with passive targets provide visual feedback to the operator through a camera system that is integrated into the control system. A passive target will typically use a mechanical grid to help the operator determine required steering corrections. A tunnel laser-to-target guidance system is shown in Figure 6.4 below.

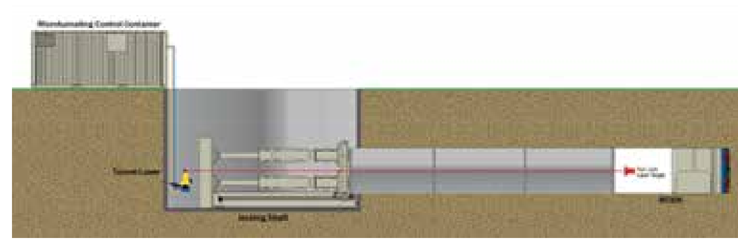


Figure 6.4 Tunnel Laser Guidance System.

Tunnel Laser Guidance System Advantages

Pipe lasers have been used in the trenchless industry for many years and can be used for a variety of other purposes in the construction industry. Modern tunnel lasers are highly functional and provide reliable operation in controlled environments.

Tunnel Laser Guidance System Disadvantages

Tunnel distances should be limited to approximately 1,200 ft (365 m) when using a tunnel laser system to hold line and grade tolerances. The laser beam is susceptible to refraction inside the tunnel string due to temperature gradients, moisture, or dust. Proper ventilation and control survey checks are required to ensure the microtunnel operator is not following a laser beam that has refracted due to these environmental issues.

Laser-Target Guidance System Enhancements

Water levels can be used simultaneously with laser-based or other systems such as gyro compass navigation as a back-up for grade control during microtunneling. The water level is used only to calculate vertical machine position relative to a hydrostatic sensor located in the jacking shaft.

6.3.2 Gyro-Compass Navigation System

Gyro navigation guidance systems are designed for curved or extended length tunnel alignments and use a self-leveling, north-seeking gyrocompass to detect direction of the MTBM. The gyro system requires feedback from the MTBM to calculate the current position. An electronic water level system is integrated to determine vertical deviations.

Gyrocompass Guidance System Advantages

Modern gyro navigation systems can be installed in relatively small diameter MTBM heads and are best suited for diameters ranging from 30 in. (800 mm) to 48 in. (1,200 mm) nominal ID pipe diameters. Since line-of-sight is not required from the launch shaft to the MTBM, gyro navigation is suitable for curved tunnel alignments.

Gyro navigation can generally be integrated into an existing microtunneling system with the support of the equipment manufacturer as part of the control package, or as a stand-alone third-party guidance system package.

Gyrocompass Guidance System Disadvantages

Gyro navigation uses feedback from the MTBM system such as steering corrections from the operator to calculate current position. These calculations are based on assumptions that the MTBM will move exactly along the alignment to which the operator has steered. In some ground types, the MTBM may not be as reactive to steering inputs causing a “drift” to occur. Due to the influence of MTBM drift, calibration of the guidance system must be checked by survey every 65 ft (20 m) to 130 ft (40 m).

Gyrocompass operation generally consists of a rotating disc on an axis to determine true north. Depending on the gyro technology used in the MTBM, a period of time may be required for the gyro to accurately stabilize prior to advancing once powered-up.

6.3.3 Azimuth-Laser Total Guidance System

Azimuth-Laser Total Guidance Systems is an azimuth-based navigation system with self-leveling total station connected throughout the alignment without the need for a manual or continuous survey. A primary shaft total station is mounted in the jacking shaft and calibrated to surveyed azimuth prisms affixed to the jacking shaft. A laser projected from the shaft total station to the MTBM target for the initial 300 ft (90 m) or until a line-of-sight is not possible. See Figure 6.5 for initial set-up of the guidance system.

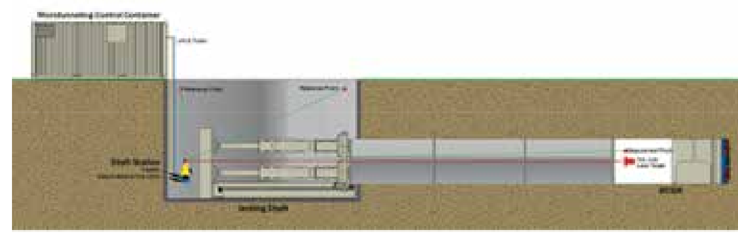


Figure 6.5 Azimuth-Laser Total Guidance System Launch.

The tunnel station that is affixed behind the MTBM then projects a laser to the project Designed Tunnel Alignment (DTA) to minimize effects from atmospheric conditions. Additional tunnel stations are then added to the pipe string for the series of total stations to continually monitor each other position. Shaft total stations are required when the line-of-sight is not present during curves between distances of 1,000 – 3,500 ft depending on the size of the unit’s prism and tunnel atmosphere. Figure 6.6 shows various set-ups between straight and curved alignments.

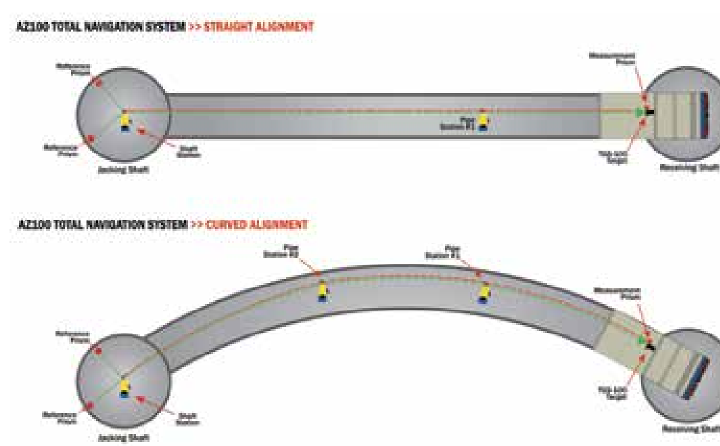


Figure 6.6 Azimuth-Laser Total Guidance System Configurations.

Azimuth-Laser Guidance System Advantages

Azimuth-Laser total guidance system perform series of self-survey checks throughout the tunnel operation. To eliminate potential errors due to atmospheric conditions, some manufacturer’s systems perform multiple back-checks from the MTBM.

The Azimuth-Laser guidance systems can be used without a base shaft station; however, without this station manual check will be required every 65 ft (20 m) to 130 ft (40 m).

Azimuth-Laser Guidance System Disadvantages

Azimuth-Laser total guidance systems require a line-of-sight between the MTBM target and the total stations. A Designed Tunnel Alignment (DTA) with multiple or tight radius curves will require multiple tunnel stations along the alignment, increasing the cost of the system.

The minimum nominal internal diameter for an Azimuth-Laser Total Guidance system is 48 in. (1,200 mm) to properly mount the theodolite. A mounted shaft station is shown in Figure 6.7 below.

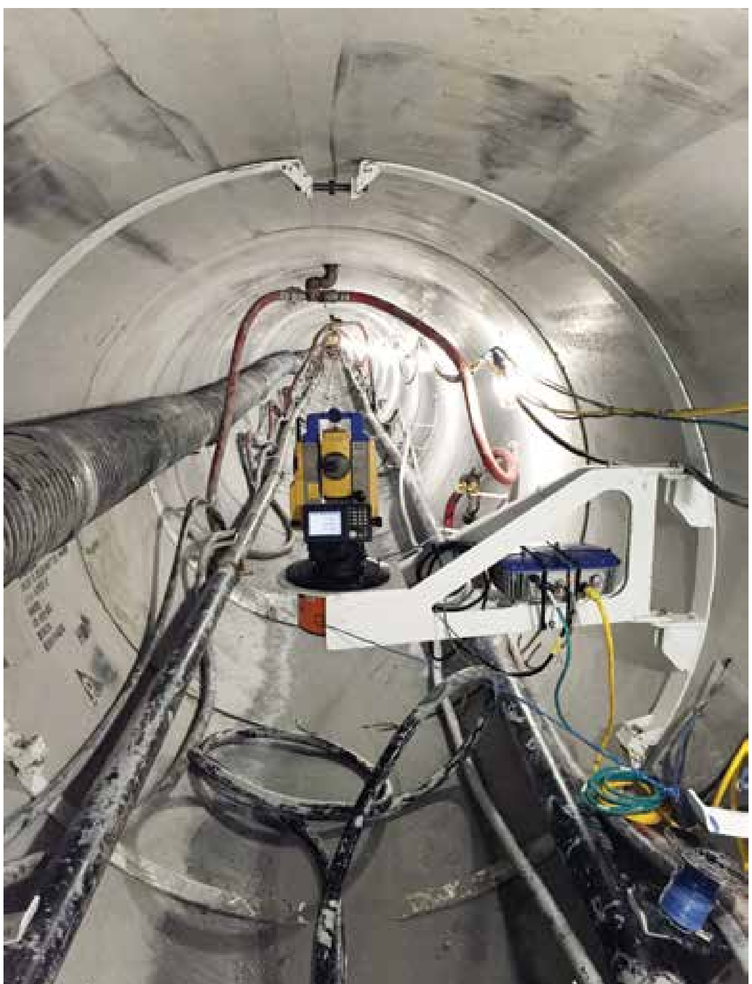


Figure 6.7 Azimuth-Laser Total Guidance System – Tunnel Station.

6.4 SLURRY MICROTUNNELING METHOD

The slurry microtunneling method is a remotely controlled pipejacking process that accurately installs a wide variety of product pipe while maintaining positive earth and hydrostatic pressure. The MTBM mechanically excavates the soil with a rotating cutterhead while being simultaneously advanced by a high-capacity thrust frame located in the launch shaft. As shown in Figure 6.8, excavated material at the cutterhead is mixed with face slurry. Larger debris is further broken down to manageable size for the slurry circuit by the crushing cone assembly. The slurry is then returned through screened intake ports and back to a slurry separation plant. Critical slurry flow velocities are required to suspend cuttings within the closed-loop slurry circuit. Typically, this flow velocity is around 10 ft/s, however, it is highly dependent on the viscosity of the slurry, which may require additional treatment depending on geotechnical conditions encountered during the tunneling process.

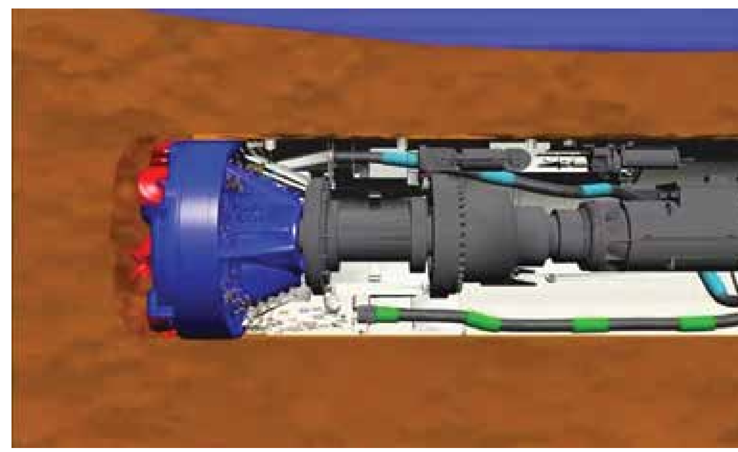


Figure 6.8 MTBM Slurry Flow Simulation.

6.4.1 Slurry Balance – Hydrostatic & Earth Pressure

Slurry balance is achieved by monitoring pressure transducers for both the inlet slurry as well as the slurry return chamber. Static ground water pressure is monitored by pressure transducers during periods of shutdown. MTBM systems monitor mechanical face pressure with active pressure sensing on the steering system. Gauges showing the mechanical resistance to the face of the steering cylinders gives an estimation of earth pressure as well as the location of face pressure during mixed face conditions.

The MTBM shown in Figure 6.9 illustrates dual and variable flow modes to regulate the drilling slurry to the cutting and slurry chambers.

This variability is to control slurry pressures and flow interactions for cohesive and non-cohesive ground conditions. For ground conditions that are predominantly non-cohesive, the MTBM operator may elect to provide the majority of the slurry flow to the slurry chamber to create a hydrostatic balance to the face while minimizing the potential for over excavation. For cohesive ground conditions, the MTBM operator may elect

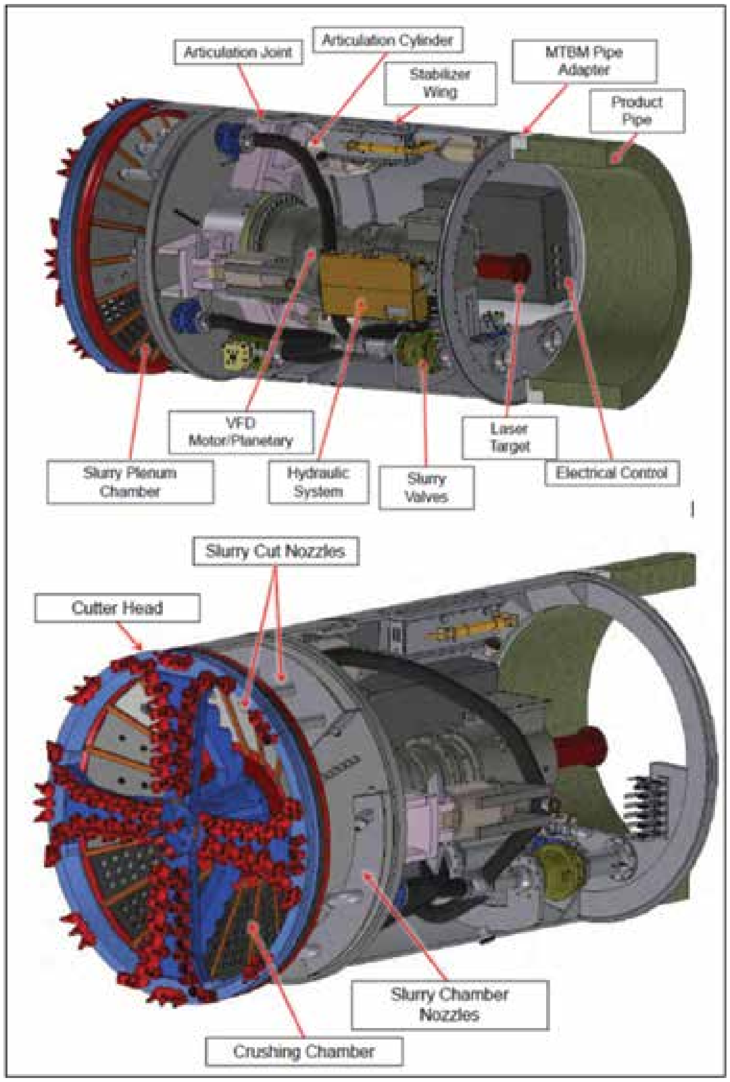


Figure 6.9 Akkerman Slurry Microtunneling.

to provide much of the slurry flow to the cut nozzles to assist with cutterhead performance and material transfer through the crushing chamber. Additional systems such as high-pressure jetting (HPJ) can also be activated in cohesive ground conditions to assist with the mechanical excavation of the cutterhead as well as to inject additives to aid the microtunnelling process. An example of the performance of high-pressure jetting nozzles can be seen in Figure 6.10.

6.4.2 Slurry Circuit – Closed Loop

Slurry microtunneling systems operate on a closed loop slurry circuit and are remotely controlled by the MTBM operator through a series of slurry pumps to maintain pressure and velocity. As shown in Figure 6.10, a basic slurry circuit contains a slurry feed pump to deliver drilling fluid to the MTBM's cutterhead, and a crushing chamber or plenum chamber as determined by the operator. Once mixed with

Figure 6.10 Akkerman SL60C with High-Pressure Jetting Nozzles.

the excavated soil, slurry return pumps transport the excavated spoil and slurry from the cutter head through the pipe to the slurry cleaning plant.

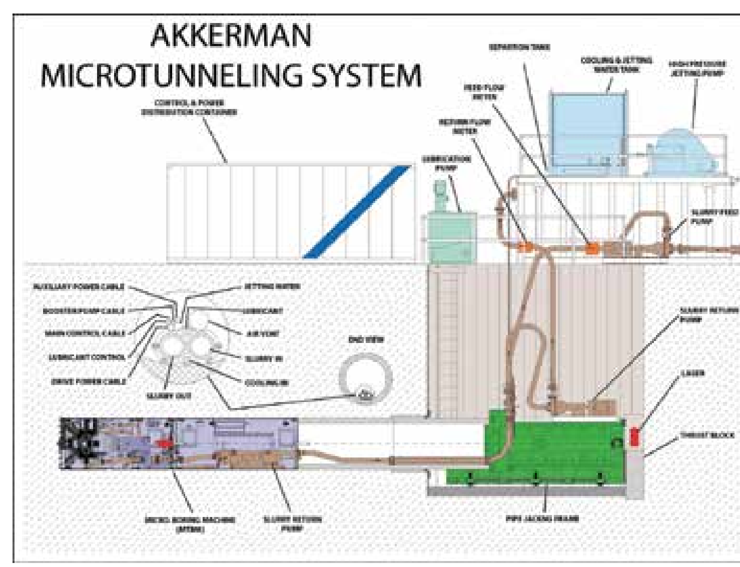


Figure 6.11 Slurry microtunneling with slurry spoil removal.

Cutter heads are bi-rotational and used to correct -MTBM roll and to provide the operator with more flexibility to help excavate, clear or pass through obstructions and/or excavate difficult ground conditions – typically ground conditions containing gravel/cobbles/boulders. The specific design of a cutter head is a function of anticipated ground condition and contractor/manufacturer preference.

Each machine also has a built-in crusher that increases the machine ability to excavate more diverse ground conditions. Machines can digest and crush cobbles, boulders and rocks up to approximately one third of the external diameter of the cutting head. The cobbles and boulders are typically crushed to smaller pieces, about the size of gravel, so the excavated material can be conveyed through the discharged line and slurry pumps without clogging.

When working in cohesive soils, such as clays with high plasticity, a high-pressure flushing system can be used to help prevent clogging of the slurry and crushing chamber. Clogging of the slurry and crushing chamber can cause a significant reduction in the effectiveness and excavation speed.

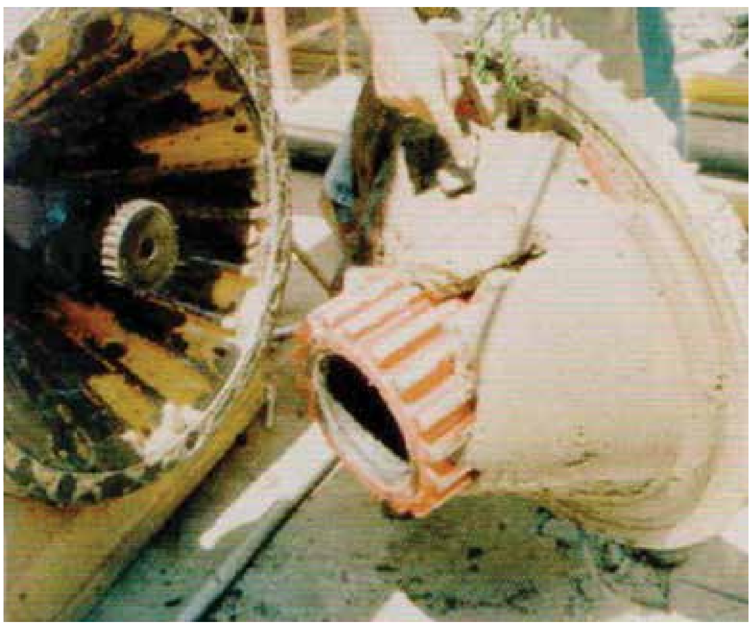


Figure 6.12 Sticking of the cutting head and crushing chamber by clay clumps.

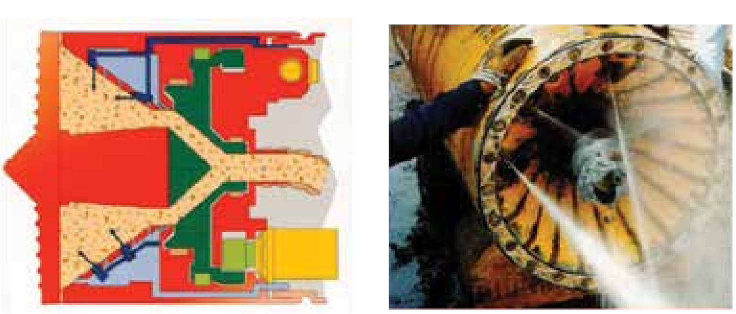


Figure 6.13 High pressure flushing system in the crushing chamber.

MTBMs are considered to have a closed face. This provides complete excavation face support (both earth and hydrostatic) while shut down. The excavated material (spoil) is cut at the face by the cutter head and passes through small parts located at the rear of the crushing chamber and into the slurry mixing chamber. This chamber provides two primary functions; (1) mixing of the spoil from the excavation face with water to form a slurry to transport the spoil from the cutter head and (2) controlling surrounding hydrostatic head. Once the spoil and water have mixed to form a slurry it is transported to the solid separation system via variable speed pumps.

The design of the slurry circulation/transportation system is a critical component of the microtunneling process. This system must be designed to provide adequate volume of flow (GPM) and velocity of flow (ft/sec). Adequate GPM ensures there is sufficient capacity to remove the volume of solids from the excavation face. Adequate velocity of flow ensures that the speed of transportation from the slurry mixing chamber is high enough to prevent the solids from settling out in the return slurry line.

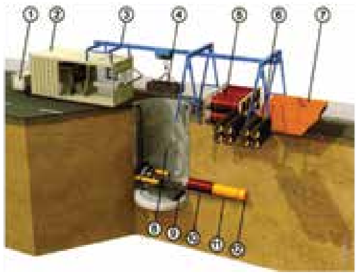
The slurry transport lines may be 3 in. (76 mm) for microtunneling systems that are less than 30 in. (762 mm) O.D., 4 in. (102 mm) for systems between 30 in. (762 mm) and 72 in. (1,829 mm) O.D., and 6 in. for systems larger than 72 in. (1,829 mm) O.D. However, the actual design of the slurry transportation system for a specific ground condition may require a change to the size of the slurry lines. For example, should the logistics of the project dictate that the solids separation plant be located a relatively far distance from the jacking shaft, or the microtunnel length is relatively long, then the design may require that the lines be upsized to reduce the amount of resistance to flow, similar to using a larger gauge wire to reduce voltage loss. In addition, soft, flowing soils may require the use of a smaller slurry line to reduce the risk of over excavation.

It is important that the water flowing into the slurry mixing chamber is as clean of excavated material as possible. However, some geotechnical conditions require the use of bentonite or other additives to increase slurry density, typically needed when the excavated ground rock or otherwise coarse in nature, with little to no fines (clay or silt).

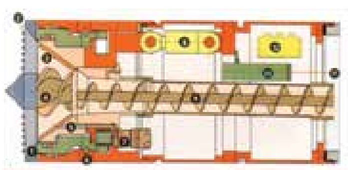
The separation system selected for a particular project is a function of the anticipated geotechnical conditions and the anticipated production rate (feet or meters installed/day) for the specific size of pipe to be installed.

The ability of solids to separate from the slurry and the rate of settlement is a function of the geotechnical conditions encountered. Broadly speaking, soils can be classified as either coarse or fine. Coarse soil can be separated from the slurry much easier than fine grained soils. Often, the solids separation system for predominantly coarse soils will consist of the slurry tank and various stages of vibrating screens and hydrocyclones. However, for predominantly fine soils, the solids separation system contains additional hydrocyclones and centrifuge(s).

It is critical that the contractors obtain adequate information on the anticipated geotechnical conditions so that a compatible solid separation system can be determined and designed.



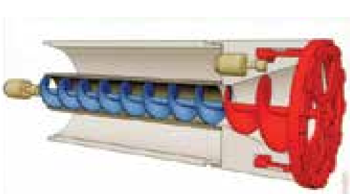
1. Power generator
2. Container with steering desk and hydraulic power plant
3. Crane way
4. Spoil transport bucket
5. Jacking pipe storage
6. Spiral conveyor guide pipes (steel) with inserted auger rods
7. Spoil container
8. Starting shaft
9. Jacking station
10. Jacking pipe
11. Microtunneling machine
12. Cutting head



1. Cutting head
2. Cutting tools
3. Crushing chamber
4. Crusher spiral
5. Water nozzle
6. Main bearing
7. Rotation drive
8. Steering cylinder
9. Spiral conveyer
10. Target board
11. Laser beam
12. Hydraulic manifold

Figure 6.14 Microtunneling with auger spoil removal. ***(Source: Visaplan GmbH)***

Driving the cutting head with the spiral conveyor



Direct driving of the cutting head and separately driven spiral conveyor

Figure 6.15 Cutting head driving options. ***(Source: Visaplan GmbH)***

6.4.2 Microtunneling with Auger Spoil Removal

Microtunneling with auger spoil removal is a process of jacking the pipe in soil and excavating the soil at the same time by using a cutter head and removing the spoil with an auger toward the jacking shaft. There are two different cutting head drive options; a cutter head that is driven by the auger and a cutting head driving separately from the auger. In the first option, the auger and cutting head are fixed together and are driven by using the drive assembly which is installed on the jacking frame in the jacking shaft. In the second option, the cutting head and auger are not connected to each other. The cutting head is driven with a motor that is installed in the microtunneling machine and only the auger is driven as described for the first option. Figure 6.7 illustrates the microtunneling with auger spoil removal. Cutting head drive options are shown in Figure 6.14.

The spoil material is excavated by the cutterhead and spoil is compacted and forced into the auger. The most common size of this auger is 4 in. (102 mm) diameter. The seal for the excavation face is provided by the spoil being loaded and compacted in the auger system.

This method is only suitable for cohesive soil in stable conditions. When high hydrostatic head or porous geotechnical conditions are encountered, there is a possibility that the compacted soil that forms the ground control seal may blow out leaving behind an open conduit that will allow water and running soils to flow. This uncontrollable loss of soil and water can result in significant settlement.

The auger MT system utilizes a laser beam as line and grade reference. Because the excavated material is removed mechanically rather than hydraulically, the spoil does not need to be mixed with water to a pumpable consistency; however, water or bentonite/polymer slurry can be added at the MTBM face to facilitate the spoil removal process and to keep the cutterhead clean. This is essential when sticky clay is encountered.

At the jacking shaft the spoil is collected in a container (i.e., skip) located beneath the MT jacking frame. The skip is designed to be large enough to hold the spoil from one pipe segment. The spoil can be dumped into container and hauled off the jobsite or stored at the jobsite.

The MTBM for the auger system has a completely articulated steering head similar in design to the slurry MT system. The drive and reception shaft must be designed and constructed to accommodate the specific equipment to be utilized, the size of pipe to be jacked, and the type of anticipated soil condition.

6.5 MICROTUNNELING CHARACTERISTICS

Table 6.1 illustrates the main microtunneling characteristics including type of pipe installed, pipe size range, bore span, ground movement, area requirements, operative skill requirements, accuracy, and recommended ground conditions.

6.6 JACKING FORCE DESCRIPTION

The force needed to push the MTBM and the pipe string forward is called the jacking force (JF). The jacking force has two major components (Figure 6.16.): (1) the MTBM cutter-face pressure, and (2) the machine and pipe frictional force resistance.

Table 6.1 Microtunneling Characteristics.

Type of Pipe Installed
• Microtunneling pipe should have a smooth exterior wall, with a uniform outside diameter and flush joints. • Sufficient strength to withstand the compressive installation load and the in-place, long-term service load. • Available pipe types for MT: • o ductile iron pipe (DIP), • o fiberglass-reinforced polymer mortar • o polymer concrete (PC) • o reinforced concrete (RCP) • o steel • o vitrified clay (VCP)
Pipe Size Range
• Microtunneling has been used to install pipe from 10 in. (250 mm) to as large as 120 in. (3,000 mm). The most common range for MT is between 24 and 84 in. (610 and 2,100 mm).
Bore Span
• Drive length up to 3,300 ft. (1,000 m) • Intermediate jacking stations ca be used on pipe with a minimum of 36 in. (900 mm) I.D. and larger.
Ground Movement
• When the microtunneling equipment is operated properly with adequate cover over the pipe, it is possible to install the pipe without heaving the ground or creating subsidence. • Typically, a minimum cover of two pipe diameters or 6 ft (2 m), whichever is deeper, is needed to allow for sufficient installation without heave or settlement. HOWEVER, specific projects can be performed with less cover without heave or settlement.
Area Requirements
• The required area for a microtunneling operator will be larger for a slurry system than auger system. • All systems need enough room for the shaft, operator's control container, lifting equipment, pipe storage, and space for loading and hauling spoil. • The slurry system will require additional room for the solid separation system. • Linear arrangement or a cluster arrangement can be applied for equipment arrangement based on the actual jobsite conditions.
Operation Skill Requirements
• High degree of skill and training required. • Must be able to interpret the readings of the various gauges correctly • Must be able to handle the various situations that develop in the field.
Accuracy
• High degree of precision mainly used for the installation of sewer lines. • The laser or guidance system for controlling the alignment permits installation to an accuracy of 1 in. (25 mm).
Recommended Ground Conditions
• Wide variety of soils from soft soils to hard clays. • Solid rock with unconfined compression greater than 30,000 psi • The equipment is capable of handling cobblers/boulders up to 30% of the machine's base outside diameter. • Working with high hydrostatic heads greater than 100 ft or 45 psi. • Auger Microtunneling is ONLY suitable for cohesive soils.

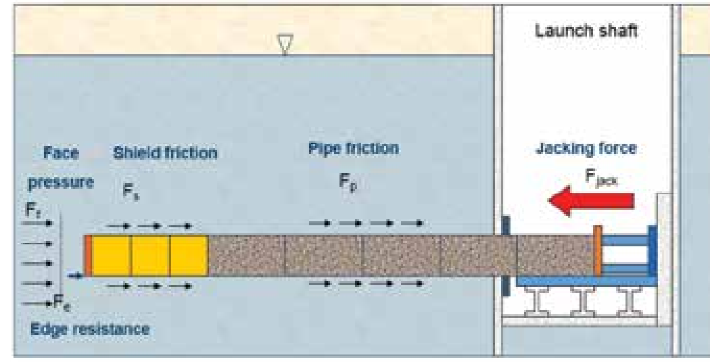


Figure 6.16 Jacking force prediction.

The theoretical anticipated jacking force can be calculated utilizing various equations; however, caution must be applied because of the numerous factors that can and will impact the actual jacking force encountered. The factors include:

- Type of soil.
- Cross-sectional area of the cutterhead.
- Type and size of pipe.
- Coefficient of friction between the soil and the outside wall surface of the pipe.
- Depth of soil over the pipe.
- Depth of water over the pipe.
- Amount of overcut.
- Amount and type of lubrication applied to the annular space between the O.D. of the pipe and the actual soil excavated.
- Accuracy of the tunnel.

Figure 6.17 Intermediate sealed jacking station.

Although, the theoretical jacking force can be calculated, it is just as important to consider historical jacking force encountered in similar conditions by contractors and/or manufacturers. This information or supplemental theoretical calculations can be used when determining: the load conveying capacity of the pipe; main jacking frame capacity; number, capacity, and spacing of intermediate jacking stations; and drive shaft and thrust wall design. Figure 6.17 shows an intermediate jacking station.

In addition to adequate jacking force being available to permit excavation of the soil at the face and installation of the pipe, the rate at which this jacking force is applied is a critical component in the operation of an MT operation. The rate at which the jacking force is applied allows the operator to ensure that the actual pressure applied to the excavation face stays within acceptable limits. These acceptable limits relate to the active and passive pressure of the soil encountered. The active soil pressure is equivalent to the pressure that the soil places on the face of the MTBM when no forward movement of the machine is occurring. This force is a measure of the tendency of the soil to run into the face of the machine. The minimum rate at which the JF is applied should be enough to ensure that the JF is greater than the active bearing capacity of the soil.

The passive capacity of the soil is the maximum pressure that can be applied to the excavation face by the MTBM without resulting in the movement of the excavation face in the direction of the MTBM. If this passive pressure is exceeded by the MTBM, the soil stress of the excavation face is increased. An over-stressed soil condition can manifest itself at the surface of the ground in the form of a heaving.

Methods of calculating jacking forces are provided in the following references:

- Bennett (1998)
- Bennett and Cording (2000)
- Pipe Jacking Association (1995)
- Thomson (1993)
- Stein (2005)
- Najafi (2013)

6.7 MAJOR ADVANTAGES AND DISADVANTAGES

The microtunneling method can install pipes to extremely accurate line and grade tolerance. It has the capability of performing in very difficult ground conditions without expensive dewatering systems and/or compressed air. Lines can be installed at greater depth without a drastic effect on the overall project cost. Safety is enhanced because workers are not required to enter trenches or tunnels. The finished product (carrier) pipe can be jacked directly without the need of a separate casing pipe.

Table 6.2 General advantages and disadvantages of different microtunneling methods.

Microtunneling Method	Advantages	Disadvantages
Microtunneling with auger spoil removal	• Excavated spoil is usually transportable and disposable without the use of a separation plant. • Relatively short set-up and takedown times.	• Reduced performance as size increases. • Applicable only in consolidated soils. • Limited application in rock. • Short tunnel drives. • Cannot perform curved tunneling.
Microtunneling with slurry spoil removal	• Application in soil and rock, with or without groundwater. • Long tunnel drives. • Curved drives. • Face access on larger machines. • Extremely accurate. • Earth Pressure Counter Balance. • Hydrostatic Counter Balance.	• Larger lay down area at jacking shaft.

Curved microtunneling adds many benefits, such as:

- Avoidance of conflicts with utilities (above and underground.
- Avoidance of known buried objects.
- Reduced disruption of traffic.
- Greater flexibility in shaft locations.
- Reduction in the number of shafts.
- Reduced overall project cost.
- High accuracy.

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2. *An Introduction to Pipe Jacking and Microtunnelling. Pipe Jacking Association. ISBN 978-1-5272-0341-9. 2017.*
3. *Mohammad Najafi. Trenchless Technology: Planning, Equipment, and Methods. McGraw-Hill Companies, Inc. 2013.*

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Photo courtesy of Akkerman

CHAPTER 7
Guided Boring Methods

7.1 INTRODUCTION

This Guided Boring Method chapter will review the technologies that have been specifically developed to install infrastructure on-line and grade via the use of a pilot hole (a small hole drilled as a guide for the drilling of a larger hole).

As with many technologies found in this manual, manufacturers each have their own unique designs and approach to the industry. This chapter will serve as a detailed overview, explaining the common elements of the Guided Boring Method.

Since the 1980s the Guided Boring Method (GBM) has proven to be a very successful way to steer line and grade critical, lateral casing installations that otherwise were performed via "poke and hope" methods. Unlike Horizontal Directional Drilling (HDD), Guided Boring is intended for straight line casing installations. The systems found in this chapter are all steered from the bore pit as these are non-man entry systems.

GBM were first used in Germany for the installation of sewer laterals. Its first appearance in the United States was 10 years later for installation of laterals and small-diameter sewer mains. Advances in the Pilot Tube Method (PTM) technology has allowed the installation of pipes up to 72-in. (1.8 m) OD at distances commonly up to 400 ft (152 m) in various ground conditions. The

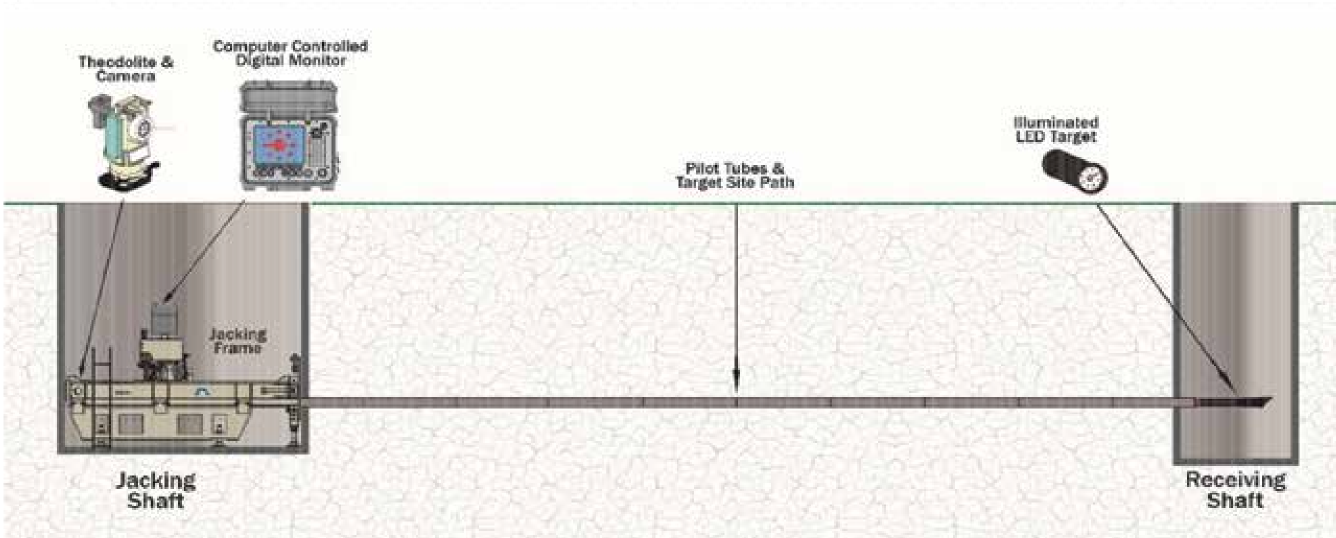

Figure 7.2.4 - On-line and on-grade installation of the pilot tube across the full length of the bore.

Guided Boring Method (GBM) is a hybrid technique that combines the characteristics of other trenchless techniques to accurately install a pipe to line and grade. The GBM is also known as the Pilot Tube Method (PTM) and has previously been referred to as Pilot Tube Microtunneling.

Choosing to include the use of GBM gives the contractor the ability to install a pilot tube across the entire length of the drive online and grade prior to installing any casing pipe. Since the pilot tubes are installed accurately along the alignment, any potential challenging subsurface conditions are generally known and can be mitigated prior to the installation of the steel casing or product pipe. GBM saves the contractor and project owner in casing costs. Typically, on an unguided auger bore, contractors and engineers will select oversized casing to create a larger annular space. The larger annular space allows them to correctly position the interior carrier pipe to achieve the required alignment. When an accurate line and grade is obtained with the GAB, a smaller sized casing can be installed. The net result of the smaller casing includes the following: a lower cost of raw materials; lower cost of spoil removal from the site; and less annular space between the host and carrier to fill with grout or another fill.

7.2 OVERVIEW OF GBM

GBM can be performed from 8-ft (2.4 m) diameter shafts up to standard auger boring pit size of 12 ft (3.6 m) wide x 40 ft (12 m) long excavated bore pits. In either case, the operation begins with the online and on-grade installation of the pilot tube across the full length of the bore. To accomplish this a few different tools are used. First is the Theodolite Guidance System. The Guidance System includes an LED illuminated target, digital theodolite, camera, and monitor. Also used is a Guided Boring Machine, which are unique in that they are designed to have hollow drive spindles. This provides clear path of vision for the camera through the rotary box of the Guided Boring Machine, pilot tube and to the steering tip which houses the Guidance System LED Target. All these things combined create a system that makes it easy for the operator to monitor exactly where the bore head is positioned in real time.

Figure 7.2.1 - Theodolite Guidance System.

Figure 7.2.2 - Vertical Shaft operation.

Figure 7.2.3 - Excavated bore pit operation.

The method tends to be used for pipes that can withstand high jacking loads. Therefore, this technology can be used to install a variety of pipe jacking materials such as concrete, clay, fiberglass, polymer concrete and steel. GBM can be used in a variety of soil and solid rock conditions. Flowing sands, large cobbles and boulders can cause some challenges during operation (see Table 7.1). As with any type of underground boring, a good soils investigation is crucial to accurately selecting tooling.

Table 7.2.1 - Applicability of GBM for different soil conditions.

Type of Soil	Applicability
Soft to very soft clays, silt, and organic deposits.	Yes
Medium to very stiff clays and silts.	Yes
Hard clays and highly weathered shales (blow counts less than 50).	Yes
Very loose to lose sands (above water table).	Yes (w/ lubricant)
Medium to dense sands (below the water table).	Yes (to 10 ft. (3 m) head) - Marginal (over 10 ft. (3 m))
Medium to dense sands (above the water table).	Yes
Gravels and cobbles less than 2 to 4 in. (51 to 102 mm) diameter.	Yes
Soils with significant cobbles, boulders, and obstructions larger than 4 to 6 in. (102 to 152 mm) diameter.	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils.	Yes
Solid rock.	Yes

7.3 PERFORMING THE GUIDED BORING METHOD

As with most equipment, similarities in design among GB Manufacturers are evident. GBM can be described in the following phases of operation: set up; drilling a pilot hole; installing a casing; and lastly, installing the product pipe. However, there are also unique developments provided by Manufacturers that in many cases combine the phases of installing a casing and installing the product pipe.

<u>The Guided Boring Process</u>:

7.3.1 Setting Up

The Guided Boring Method begins with setting up the Guided Boring Machine in a vertical shaft or on the track rail of an Auger Boring Machine (ABM) and aligning it to the centerline of the planned bore.

Once the GBM is properly aligned, pilot tube installation can proceed. Extreme care should be taken to ensure that the guided boring machine and its guidance system are properly aligned to the planned bore path. Even the slightest error can be greatly exaggerated over the distance of the bore. Figures 7.3.1.1 to 7.3.1.7 relate to the setup of a Guided Boring Machine.

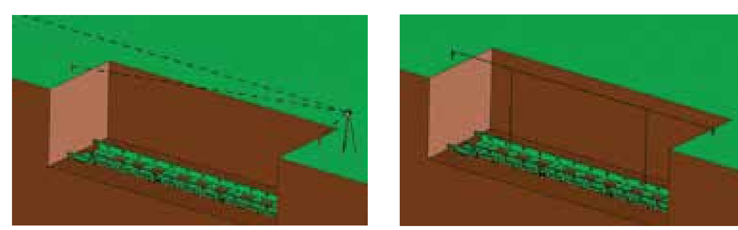

Figure 7.3.1.1 - Illustrates the use of a Transit to set pins for a stringline to be used to mark the proposed bore path. This process is the same for excavated pits or vertical shafts (excavated pit shown).

Once the stringline is set, aline the front and rear of a Guided Boring Machine or the track rail of an ABM by dropping plumbobs down from the stringline. The string must match the centerline of the machinery. Adjust the machinery as needed and lock into place. It is important to note that when an ABM is used for a guided bore the Guided Boring Machine is later placed on the track rail infront of the ABM and made to match the track alignment.

Next, adjustments are made to set the machinery to the desired grade. When an ABM is used precautions are taken in the pit excavation to rough in the floor of the pit to the grade of the bore. On longer bores it is recommended to pour a concrete floor to avoid machine settling. When GBM is performed from virtical shafts the Guided Boring Machine is anchored within

Figure 7.3.1.2 - Machinery centerline is matched to the plum bob.

Figure 7.3.1.3 - Verify grade.

the shaft at the proposed grade. In either case, once the machinery is in place it is verified and locked down.

After the machinery is set, the Theodolite and Camera are assembled behind the Guided Boring Machine.

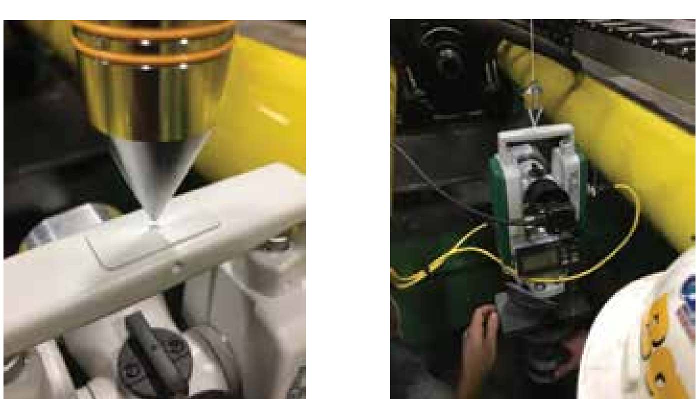

Figure 7.3.1.4 - Theodolite is matched to the plum bob.

To complete set-up, a Target housing with the chosen Steering Tip is fitted with an LED Target and coupled to the Guided Boring Machine's drive spindle. Fine adjustmets are made to the Camera and the camera's view is displayed on the Monitor.

Figure 7.3.1.5 - Guidance System setup is completed.

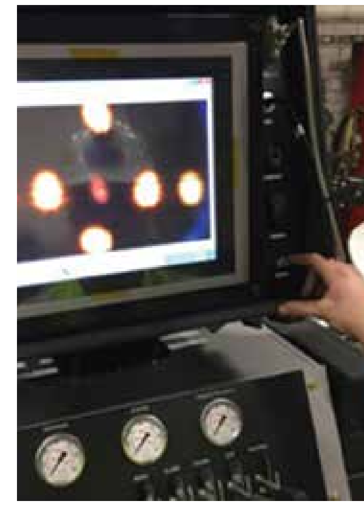

Figure 7.3.1.6 - Display of LED Target on Monitor.

Figure 7.3.1.7 - Stringlines are left in place during operation.

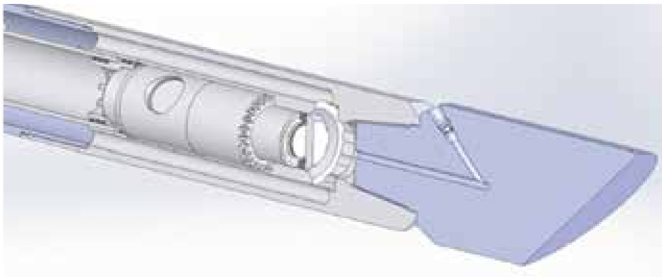

Figure 7.3.2.1 - Steering Head fitted with Target.

Figure 7.3.2.2 - Line and grade feedback to operator.

by means of a simple yet effective Slant Face Steering Head. As the slant face of a tool is pushed forward the tool is forced to the direction of the slant. Guidance System Targets lock into the Steering Head so that their 12:00 position is in correct orientation of the Slant Faced Steering Tool. As the Pilot boring operation is in process, line and grade feedback is provided to the operator in real time by the Theodolite Guidance System and displays an image of an LED target onto a viewing monitor. The Operator makes steering corrections in attempt to keep the center “bullseye” in the monitors crosshairs. This process is repeated until the steering head reaches the planned destination.

The pilot tube steering head has a slightly larger diameter than the pilot tube which creates a small annular gap as it is pushed through the ground. On systems that have dual-walled pilot tubes, fluid or air can be pumped between the dual walls of the pilot tube to a variety of steering heads used during the advancement of the pilot tube On displaceable soil bores, the drilling fluid is pumped into the annular gap to reduce skin friction. Additionally, this gap allows for chip evacuation when using air in conjunction with a hammer tool.

Pilot tube installation is complete when the steering head reaches the reception shaft (Figure 7.3.2.3 and 7.3.2.4). The pilot tubes have now established a straight-line path that was set to the desired line and grade of the bore.

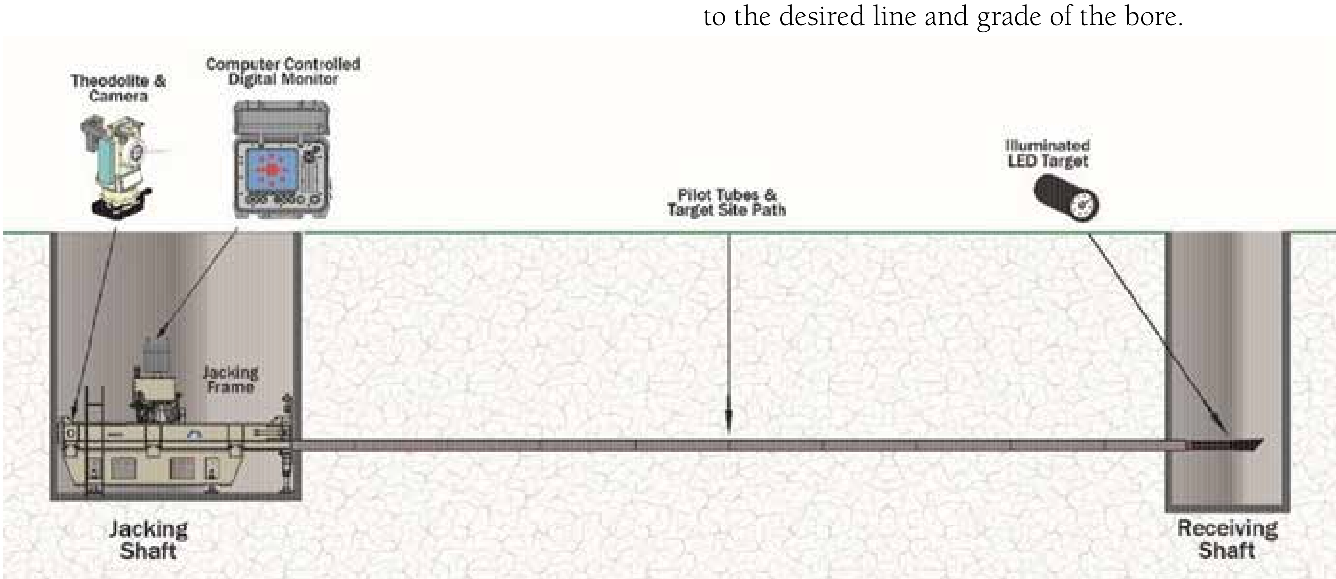

Figure 7.3.2.3 - Shaft to shaft pilot installation.

7.3.2 Boring a Pilot Hole

The pilot boring is started by pushing the Steering Head into the ground. Once the Steering Head and first Pilot Tube are in the ground, the pushing operation is stopped, and the jacking frame is retracted to its original position. Another Pilot Tube is connected to the end of the Pilot Tube already installed in the ground then it is pushed into the ground. The bore is steered

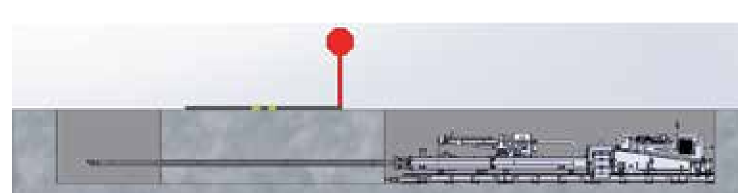

Figure 7.3.2.4 - Excavated pit to excavated pit pilot installation.

7.3.2.1 Guided Boring Machine

The pilot tubes are pushed through the ground by means of a Guided Boring Machine. These machines, produced by many manufacturers, deliver forward and reverse thrust forces as well as clockwise and counterclockwise rotational force. Guided Boring Machines are unique in that they are designed to have hollow drive spindles. This provides clear path of vision through the center of the dive spindle and through the Pilot Tubes. Some machines are specifically designed to be deployed from confined spaces like shafts. These are typically smaller in size and utilize shorter Pilot Tubes. Others are made to be used in excavated pits and can be accommodating to longer Pilot tubes.

Figure 7.3.2.2.1 - Pilot tubes with threaded and hex connections.

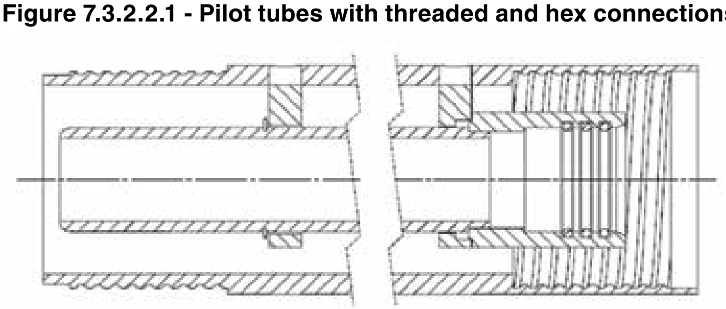

Figure 7.3.2.2.2 - Cross section view of Pilot Tube. Construction of a tube within a tube held in place by centralizers.

7.3.2.2 Pilot Tubes

Pilot tubes are hollow steel tubes that are fastened to each other with threaded or hex connections (see Figure 7.3.2.2.1 and 7.3.2.2.2). The pilot tube can be single wall or double wall. The double wall tube can be used to pump lubricant or compressed air to the steering head through the annular area between the inner and outer tubes. Lubrication reduces friction between the pilot tube string and the soil, thus helping achieve greater drive lengths.

7.3.2.3 Steering Heads

The slant faced steering head used in the GBM is common with Horizontal Directional Drilling. The slant face is an important feature of the steering head because it helps to correct alignment changes. If any changes in line and slope are ob-

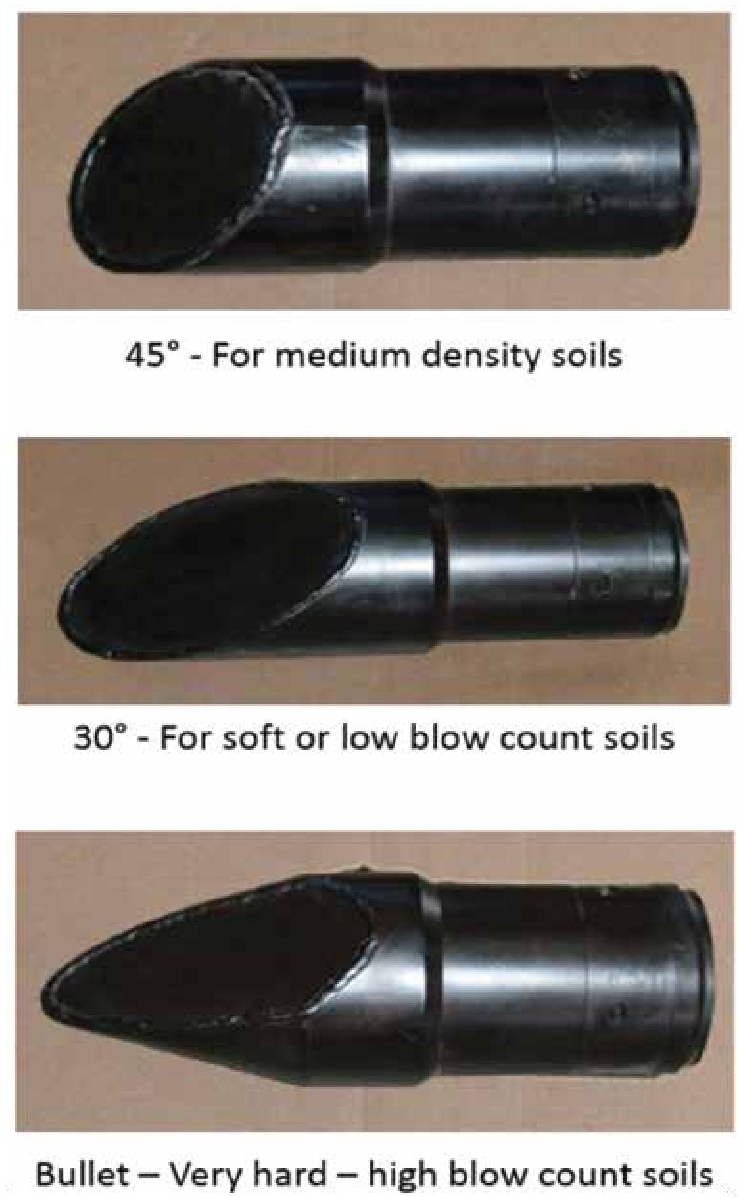

Figure 7.3.2.3.1 - Soft to hard displaceable soil Steering Heads.

Figure 7.3.2.3.2 - Soft rock Steering Head.

Figure 7.3.2.3.3 - Soft to medium rock Steering Head.

Figure 7.3.2.3.4 - Hard rock Air Hammer Steering Head.

served, stop thrusting. Then turn the steering head to the necessary position and forward thrust the pilot stem back to the alignment position. It is important to note that when steering a pilot hole, it is better to make corrections when the pilot first veers off course as opposed to attempting to make larger corrections. Larger corrections can "cat eye" the view to the target and therefore reduce the line of sight to the target.

Steering heads are made for all types of displaceable soils and rocky ground conditions.

7.3.3 Post Pilot Hole Operational Methods, Shaft to Shaft

Once the pilot tubes are successfully installed across the full length of the bore, various upsizing components can be selected based on the ground conditions, desired installation method and the product pipe to be installed on the project.

There are a growing number of operational methods used by contractors within GBM. Bores performed from shafts require consideration to the confined space and therefore primarily utilize one of following three operational methods of use: The Three-Step Method, the Modified Three-Step Method, and the Two-Step Method.

7.3.3.1 Three-Step Method

The three-step method is the most traditional form of the pilot tube method. The method includes boring a pilot hole, upsizing the bore with a temporary casing and auger, and replacing the casing with a product pipe. In the first step, a pilot tube is bored as explained in section 7.3.2. The purpose of the second step is to expand the pilot hole to a slightly larger diameter than the product pipe. To achieve this, an upsizing tool, such as a reaming head or cutting head, is attached to the previously installed pilot tube by use of a special adapter (see Fig. 7.3.3.1.1 and 7.3.3.1.2). The rear end of the reaming head or a cutter head is then attached to the jacking frame. The upsizing tool is advanced, along with the pilot string, by the jacking frame and the spoils are transported back to the jacking shaft by the internal auger.

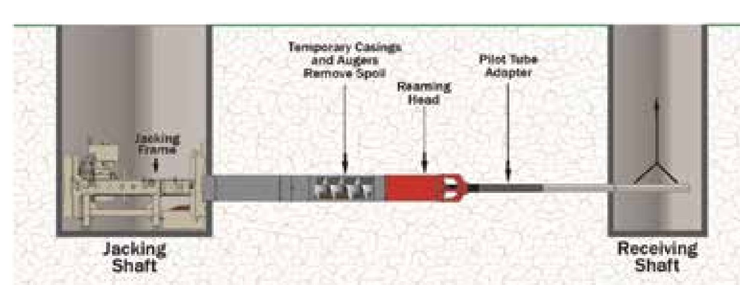

Figure 7.3.3.1.1 - Installation of temporary augers and casings.

After the upsizing tool is completely in the ground, the jacking frame is retracted to its original position. A new temporary casing with an auger inside (see Fig. 7.3.3.1.3) is lowered into the jacking

Figure 7.3.3.1.2 - Reaming Head or Cutting Head (also known as a Knife Reamer). Indicators point out lubrication ports.

Figure 7.3.3.1.3 - Temporary Casings with an auger inside.

shaft and connected to the upsizing tool already in the ground. The auger inside the new casing is connected to the auger in the upsizing tool and then the casing is joined in a similar manner. Repeat the process for connecting the casing and auger to the jacking frame. The temporary casing and auger are advanced by a combination of thrust and torque from the jacking frame. Once the casing is completely in the ground, the jacking frame is retracted, and a new casing and auger is installed as previously described. Repeat the process until the casings have replaced all the pilot tubes.

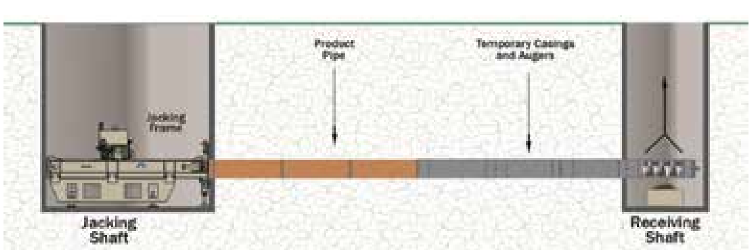

Figure 7.3.3.1.4 - Installation of product pipe.

The purpose of the third step is to replace the previously installed casings and augers with the product pipe. The front end of the first pipe section is connected to the rear end of the last casing using a special adapter (see Fig. 7.3.3.1.4). As each new pipe section is jacked into the ground a section of casing and auger is removed from the receiving shaft. The process repeats until the product pipe reaches the receiving shaft. The bore is complete.

7.3.3.2 Modified Three-Step Method

The modified three-step method uses a powered cutter head (PCH) or a powered reaming head (PRH) as a second upsizing tool in the third step. The first two steps of this method are the same as those of the three-step method. The third step of this method is similar to the three-step method except that when replacing the auger casing, the method uses PCH or PRH on the front end of the product pipe to further increase the diameter of the borehole for larger diameter product pipe.

After the second step is completed, the PCH or PRH is lowered into the jacking shaft and mounted on the jacking frame. The powered head and its internal auger are then connected to the rear end of the last casing and auger that is already in the ground (see Fig. 7.3.3.2.1). The powered head is then connected to the hydraulic system of the jacking frame. As the powered head is advanced into the ground by the jacking frame, the augers in the previously installed

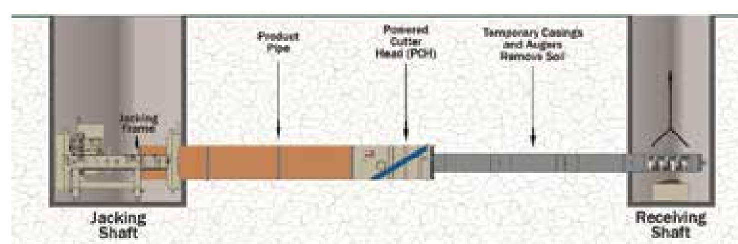

Figure 7.3.3.2.1: Installation of product pipe with Powered Cutter Head (PCH).

Figure 7.3.3.2.2 - Powered Cutter Heads (PCH).

temporary casings are rotated in the opposite direction (CCW) and the newly excavated soil is transported to the receiving shaft.

Once the powered head is completely in the ground, the jacking frame is retracted, and a new section of product pipe is lowered into the jacking shaft. The front end of the product pipe is connected to the rear of the powered head using a special adapter. The hydraulic hoses that feed the powered head are threaded through the new product pipe and then the rear of the new pipe is connected to the jacking frame using a pipe thrust adapter. The jacking frame thrusts the product pipe and the powered head forward advancing the casings and augers into the receiving shaft where they are removed.

The process is repeated until the powered head reaches the receiving shaft where it is removed along with the hydraulic lines and any lubrication hoses. The bore is complete.

7.3.3.3 Two-Step Method

The two-step method utilizes a ***Special Cutter Head*** that attaches to the pilot tube string installed during step one to simultaneously increase the bore diameter to the final product pipe size and funnel the excavated spoils to a casing and auger that is housed within the product pipe for transport back to the jacking shaft (see Fig. 7.3.3.3.1).

The first step of the two-step method is the same as the first step of the three-step method with the key difference being steps two and three are combined into one. The second step begins with lowering the cutter head into the jacking shaft and connecting it to the rear of the last pilot tube that was installed during step one. The internal auger of the two-step head is then connected to the auger drive of the jacking frame. When both ends are connected, the jacking frame thrusts the cutter head into the ground while simultaneously rotating the auger which is also attached to the cutter head. This process also advances the pilot tubes into the receiving shaft where they can be removed.

Once the cutter head is completely in the ground, the jacking frame is disconnected from the cutter head and retracted. A new section of product pipe with a separate casing and auger inside is lowered into the jacking shaft and connected to the rear of the cutter head already in the ground and to the auger drive of the jacking frame. As the pipe is being thrust forward with the jacking frame, the excavated spoils from the internal casing and auger are being deposited into a muck box that is usually contained under the jacking frame. When the pipe is completely in the ground the jacking frame is retracted. Depending on the capacity of the muck box, it may need to be emptied after each pipe section is installed. A new section of product pipe with casing and auger inside is lowered into the jacking frame and connected front and rear as described previously.

This cycle is repeated until the cutter head reaches the receiving shaft and is removed. The casings and augers within the product pipe can be removed from either the jacking shaft or the receiving shaft and the bore is complete. With the development of powered upsizing tools, this has become a seldom used method.

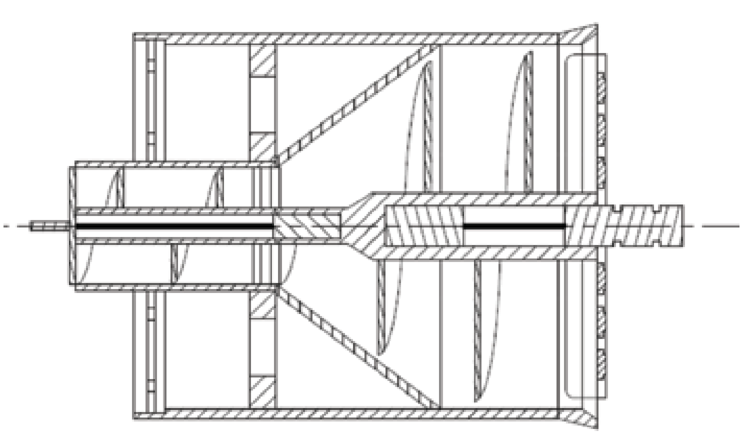

Figure 7.3.3.3.1 - Special Cutter Head.

7.3.4 Post Pilot Hole Operational Method, Excavated Pit to Pit

One of the most common applications for GBM in North America is its use in conjunction with an Auger Boring Machine (ABM). This has revolutionized the jack and bore industry by providing exact line and

Figure 7.3.4.1.1 - Soft Soil Knife Reamer.

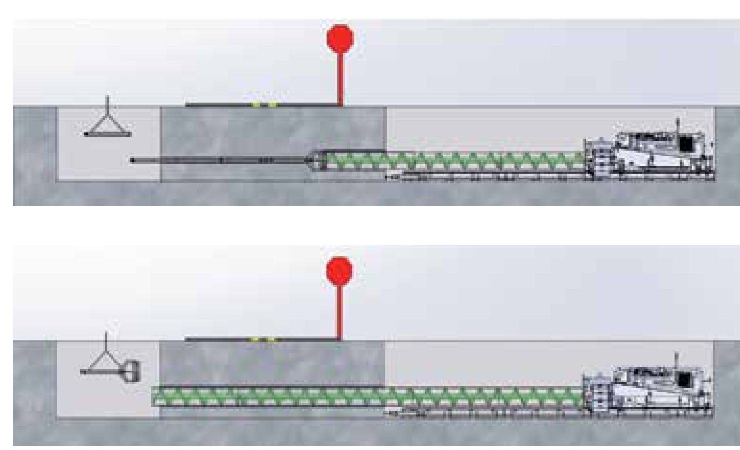

Figure 7.3.4.1.2 - Upsizing pilot hole to final casing in soft, displaceable soil using an Upsizing Tool known as a Knife Reamer.

grade accuracy. To maintain the accuracy of the pilot hole it is often recommended that bores calling for casing larger than 36 in. be installed in two steps or phases. Casing installations of 36 in. or smaller are typically installed is a single step once the pilot hole is established.

7.3.4.1 36-in. Casing and Smaller

Upon completed boring of a pilot hole in soft soils, the Guided Boring Machine is removed from the ABM rail and a connection is made between the last installed Pilot Tube and the steel casing with an Upsizing Tool known as a Knife Reamer.

Figure 7.3.4.1.3 – Rock Cutting Head with pilot receiver.

Figure 7.3.4.1.4 – Pilot Tube Swivel.

As the ABM advances, the steel casing simultaneously advances the pilot tubes to the reception shaft or pit. During the process, the spoils are moved back to the ABM via rotating auger, and then removed. This sequence is repeated until the steel casing has reached its destination and all the Pilot Tubes and Upsizing Tools are uncoupled and removed from the reception shaft or excavated pit. Next, the Auger as continuously turned clockwise until all the spoils have been conveyed back to the ABM. Once the casing is clean, the auger and ABM is removed from the starting pit. The bore is complete.

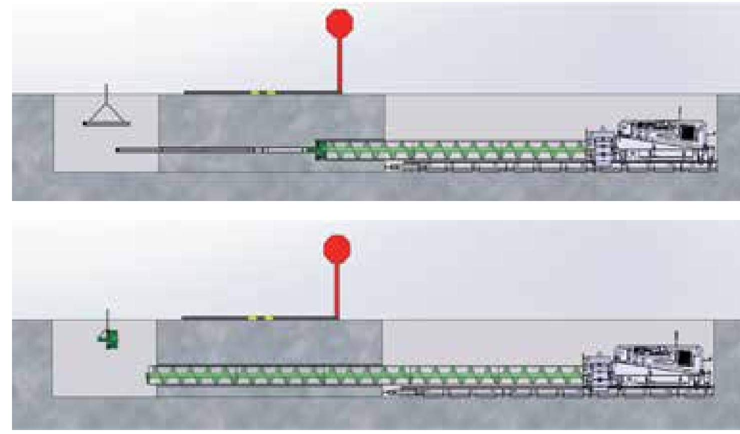

Figure 7.3.4.1.5 - Upsizing pilot hole to final casing using an Upsizing Tool known as a rock Cutting Head with pilot receiver.

Hard soils, including solid rock, must be cut or excavated as they cannot be displaced. To do this a connection is made between the last installed Pilot Tube and any conventional Auger Boring Cutting Head equipped with a Pilot Tube Receiver. The Cutting head is selected to best suit the ground conditions.

To keep the Pilot Tubes from turning as the Cutter Head is turned a Pilot Tube Swivel is used between the cutting head and the last Pilot Tube. In this case, the auger serves two purposes. First, the auger serves as a drive shaft to the cutter head. Second, the auger conveys spoils from the casing when it is turned clockwise. As the ABM advances, auger simultaneously advances the pilot tubes to the

Figure 7.3.4.2.1 - Soft soil step reaming head known as a Transition Reamer.

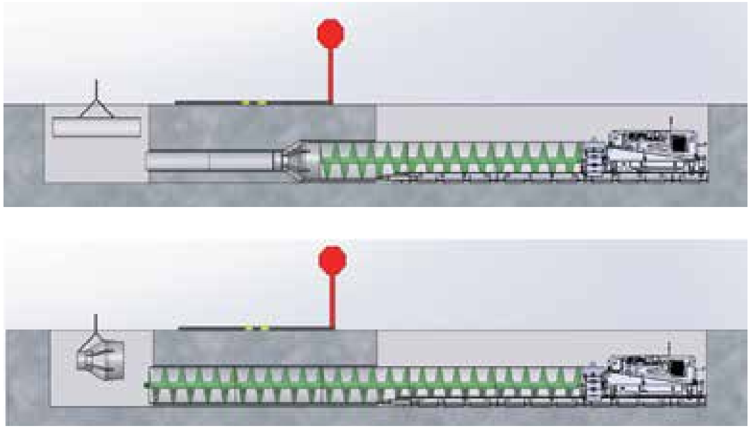

Figure 7.3.4.2.2 - Step Reaming process in soft, displaceable soil utilizing a Transition Reamer.

reception shaft or pit. This sequence is repeated until the steel casing has reached its destination and all the Pilot Tubes, Swivel and Upsizing Tools are uncoupled and removed from the reception shaft or excavated pit. The bore is complete.

Figure 7.3.4.2.3 - Casing Thrust Swivel connected to conventional cutter rick head with pilot receiver.

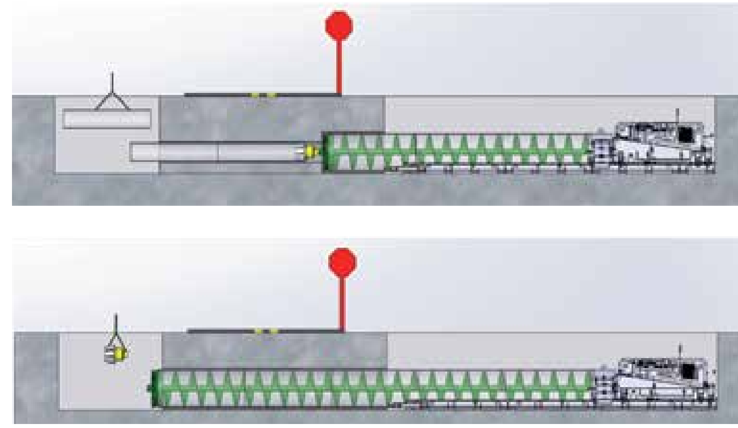

Figure 7.3.4.2.4 - Step Reaming process in hard soil or rock utilizing a Casing Thrust Swivel connected to conventional rock cutter head with pilot receiver.

7.3.4.2 Installing Casing Larger than 36 in.

Projects that call for Casings larger than 36 in. often require upsizing the pilot hole in two steps. This process is commonly referred to as Step Reaming. One of the reasons for this is because, in softer soils, the weight of the Casing and the Upsizing Tool can become heavy enough to drag down the Pilot Tubes, causing a loss of grade during the upsizing process. To avoid loss of grade, a temporary casing is installed then pushed out via the use of Step Reaming Heads and Tooling.

Two step upsizing in firm, hard or solid rock conditions require a Casing Thrust Swivel. This tool allows the temporary casing to be thrusted on by the Auger without concern for the temporary casing turning or spinning. After the temporary casing is installed and cleaned out a Casing Thrust Swivel fitted with the appropriate size centralizer is inserted into the end of the temporary casing installed in the first pass.

Figure 7.4.1 - Pilot tube guided pipe ram.

As the ABM advances, the final casing the Auger string simultaneously advances the temporary casings. During the process, the spoils are moved back to the ABM via rotating Auger, and then removed. This sequence is repeated until the final steel casing has reached the exit pit and all the temporary casing is removed. Next, the Auger is continuously turned clockwise until all the spoils have been conveyed back to the ABM. Once the casing is clean, the auger and ABM is removed from the starting pit. The bore is complete.

7.4 GUIDED PIPE RAMMING

Due to the versatility, small surface footprint, and accuracy of GBM, contractors, engineers and manufacturers have worked together to allow new techniques to be adopted in the trenchless industry. One such hybrid technique is Guided Pipe Ramming.

Guided pipe ramming was first conducted in 2006 on a rescue project in Farmington, Utah, to install a 60-in. (1,500-mm) casing between railroad tracks and has since become a standard service offering for several contractors. Guided pipe ramming uses the pilot tube guided auger boring method of accurately installing pilot tubes on the centerline of the bore to establish line and grade. The pilot tubes are followed by a pneumatic pipe hammer attached to the rear of the product pipe. The hammer advances the product pipe which is attached to the previously installed pilot tube by a special adapter such as a Weld-On Reaming Head (WORH) (see Fig. 7.4.1).

It is important to note that this method most often uses an ABM to clean out the casing. This can easily become an unsafe act known as “free boring.” Extreme caution must be used to avoid death or injury and ensure the ABM does not flip or rollover during this process.

7.5 SIMILAR TECHNOLOGY

Favorable	Unfavorable
Medium dense to dense sand	Loose Sand
Soft to stiff clay	Very soft clay
Mixed soils	Cobbles/boulders
Rock	High groundwater

Vacuum Micro-Tunneling (VMT). Utilization of Laser and Camera Guidance System

Vacuum micro tunneling is a laser guided, remote control, and non-man entry boring system that does not incorporate boring a pilot hole. The VMT has been designed to install pipes ranging in size from 10 in. (0.25 m) to 18 in. (0.46 m) with drive length up to 350 ft. (107 m) [1]. In the VMT system, the drill head contains a steering mechanism and camera which sends the target image to the operator console. A cutter bit rotates to cut through soil. The displaced soil is removed by vacuum through the drill head and drill casing and deposited into a vacuum tank. Figure 7.5.1 shows the system along with jacking set up. Applicability of VMT for different ground conditions are listed below.

<u>Advantages of the VTM method:</u>

- Small launch pit dimension requirements.

Figure 7.5.1 – VMT components along with jacking set up.

- Relatively low construction costs.
- Laser guidance system is accurate.
- Accuracy of the laser is approximately ±0.39-in. (±10 mm) over 300 ft (91 m).
- This technology can be used to install a variety of pipe materials such as polyvinyl chloride (PVC), ductile iron (DI), reinforced concrete (RCP), vitrified clay (VCP), high density polyethylene (HDPE), and steel.
- Pipe installation options of direct jack or pullback.
- Vacuum excavation results in substantially lower jacking force requirement.

<u>The VMT method has the following limitations:</u>

- Not well suited for soft clays or loose sands.
- Initial 13 in. (330 mm) pilot bore can be reamed to a maximum of 20 in. (508 mm).
- Vacuum removal of soils limits maximum size of cobbles.

Figure 7.6.1 Hydraulic Steering Head.

7.6 One-Pass Alternative — Hydraulically Steerable Guided Boring Systems

About steerable auger boring systems

In the early 1980s, another guided boring method was being developed to reduce the number of steps and equipment required for guided auger boring. Since those early prototype machines, manufacturers have perfected the technology to deliver a dependable, easy-to-operate, and quick-to-deploy GBM.

Today, GBMs like the Hydraulic Steering System include a steerable head welded to the front of the lead casing and connected using hydraulic lines to a control station where adjustments are made. A vertical alignment sensor on the control station provides continuous monitoring of grade. Also, twin line projection LED lights enclosed in the steering head give crews the ability to check and maintain line throughout the bore. Any adjustments vertically or horizontally can be made from the operator station using hydraulic actuated flaps that open and close to keep the head on the intended path without pulling augers.

Steerable auger boring systems do not require any specific makes or models of auger boring machines or specific technical training for operators, making them a more universal and economic GBM. Many contractors who currently use steerable auger boring systems will use them on every bore greater than 100 ft (30.5 m) long to help improve bore accuracy and labor efficiency on the job.

7.6.1 Setting Up and Operating a Steerable Auger Boring System

With steerable auger boring systems, there are no special entry or exit pit requirements outside of an average auger boring project. The appropriate steerable head is determined by the size of the steel casing being installed and the ground conditions. Steerable heads range in size from 16 to 72 in. (40.6 – 182.9 cm) in diameter, and there are cutters designed to work in dirt, clay, sand, and cobble, as well units built for cutting and steering through solid rock. The same control station can be used across all sizes and types of steerable auger boring systems.

After the auger boring machine is placed in the enter pit, the crew will set the control station next to it. A steerable head matching the size of the casing being installed is then welded to the first casing, and the first auger section is bolted to the steerable head's cutter. Four hydraulic lines are then connected from each steering flap on the steerable head to the control stations. These lines are connected to the top of the steerable head, and a cover protects the connections. A water line is run from the control station's recirculation tank to the steerable head for real-time accurate grade reading. An optional electrical pitch indicator can also be used in conjunction with the water level indicator.

When working in sand, bentonite can be pumped from the control station to the steerable head to help keep material from entering the steerable head's flaps. When working in clay, crews can use a detergent to prevent material from sticking to the cutter and augers. Both of these additives will also help reduce friction as the pipe goes through the ground.

Anytime throughout the bore, grade can be check using the water level indicator on the control station. Line is checked through the casing opening using two LED lights mounted inside the steerable head. Course corrections are made from the control station by hydraulicly extending or retracting the steerable head's flaps. To steer up, the operator simply needs to open the flap on the bottom. He or she can steer down by extending the top flap, steer to the right by extending the left side flap and steer left by opening the right flap. There is no need to pull augers to make course corrections. All adjustments can be made in real-time while installing the finished casing.

Once the bore is completed, the steerable head is removed on the exit pit side, and a cleanout bit is attached to the end of the auger string to remove any debris from the inside of the casing.

The minimal amount of work required on the bore's exit side also helps ensure worker safety throughout the boring process. The whole process is completed in one pass, which helps with labor costs and time on the job.

7.6.2 Sizing and Soil Conditions

A steerable auger boring system is designed to work with auger boring machines ranging from 16 to 60 in. (40-6 – 152.4 cm) in diameter. Control stations are equipped with a 50-gallon (189.3 l) water tank and a 6,500 psi (44,815.9 kPa) hydraulic pump.

Steerable head sizes available

- 16-in. (40.6 cm)
- 24-in. (60 cm)
- 30-in. (76.2 cm)
- 36-in. (91.4 cm)
- 42-in. (106.7 cm)
- 48-in. (121.9 cm)
- 60-in. (152.4 cm)
- 72-in. (182.9 cm)

Approved soil conditions

- Dirt
- Clay
- Sand
- Varying conditions mixed with rock or cobble

Note: There is a variety of cutters available with these systems to match ground conditions.

Distances

Steerable auger boring systems can effectively and efficiently steer at the same distances the auger boring machine is designed for. On average, that means steerable auger boring systems can be used on bores up to 600 ft (182.9 m).

7.6.3 Solid Rock Steering Option

In addition to steerable auger boring systems for softer soil conditions, steerable rock systems for working in solid rock formations starting at 3,000 psi (20,684.3 kPa) and up to 25,000 psi (172,368.9 kPa) and higher in some cases are now being manufactured. These steerable heads have a more robust build quality and include cutter wheels at the unit's front. They can be controlled through the same control station as the soft ground head.

Steerable head sizes available

- 36-in. (91.4 cm)
- 42-in. (106.7 cm)
- 48-in. (121.9 cm)
- 60-in. (152.4 cm)

7.6.4 Advantages

The main advantages of using steerable auger boring systems over other types of GBM are its overall efficiency and worker safety. Since this steerable solution is pulling the final casing is directly behind it, instead of making multiple passes, project timelines are shorter. The amount of equipment needed for projects is less than other GBMs. Also, almost all of the work is performed from the bore's entry side, limiting time spent on the exit side where the auger boring operator doesn't have a direct visual of his or her surroundings.

In addition, steerable auger boring systems require minimal additional training to operate.

7.6.5 Limitations

As with most auger boring machines, steerable auger boring systems do not operate as efficiently at depth below the water table. Also, these units are designed to work with steel casing, so if another type of pipe is being installed, GBMs like a pilot tube system may be a better option.

CHAPTER 8
Small Horizontal Directional Drilling Methods

8.1 INTRODUCTION OF THE HDD METHOD

Horizontal Directional Drilling (HDD) is a technique used to pull a pipe or other facility through a drilled underground tunnel. HDD began in the mid-1940s and was used to lay large-diameter, long-distance oil and sewage pipelines. HDD developed rapidly in the United States after the 1980s. HDD methods involve steerable boring systems for both small and large diameter lines. In most cases, it is a two-stage process. Figure 8.1 illustrates the two-stage process in the HDD method. The first stage involves the drilling of a pilot hole approximately 7 to 10 in. (180 to 250 mm) in diameter from one side of the obstacle to the other along the design centerline of the proposed pipeline. Generally, a drill pipe with diameter of 5 in. (130 mm) is used. The second stage involves enlarging the pilot hole to the desired diameter to accommodate the pipeline. The pilot hole is drilled with a specially built rig with an inclined carriage, typically adjusted to between 5 to 20 degrees, that pushes the drill rods into the ground. However, the optimum angle of entry of the pilot drill pipe, or pilot string as it is often called, is 12 degrees. The pilot hole continues at a

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Photo courtesy of Ditch Witch

12-degree angle until it passes through a sagbend to level out horizontally under the obstacle at a desired depth. Once the hole has passed the obstacle, it rises through another sagbend to exit on the far side with an angle of approximately 8 to 12 degrees. The location and configuration of a drilled path are defined by:[1]

- Penetration angles
- Design radius of curvature
- Points of curvature and tangency
- Desired vertical depth of cover

As the pilot hole is being drilled, bentonite drilling mud is pumped down the center of the drill rods. Bentonite is used in drilling fluids to lubricate and cool the cutting tools, electronics, to remove cuttings, and to help prevent inadvertent returns. The drill head consists of either a jetting head or a drill bit. In both cases, the soil is cut without rotating the pilot string. In the case of a jetting head, small diameter high pressure jets of bentonite slurry cut the soil and facilitate spoil removal by washing the cuttings to the surface where they settle out in a reception pit. In the case of a drill bit, the bit is driven mechanically to cut the soil. The bentonite also functions as a coolant and facilitates spoil removal. The drill head consists of a slant nose bit that when positioned and thrust without rotating, direction will change in the direction of the slant. The direction is then communicated to an electronic receiver above ground via an electronic sonde or beacon located in the drill head. It reads as a clock, i.e., if positioned at 12 o'clock and thrust forward without rotating, the movement of the drill string will go up. Once the desired depth and position is where the operator would like, the drill string is rotated and thrust forward.

The progress of the pilot hole is monitored by a specially designed electronic tracking system. One part of the system is in the drill housing and records the exact position, inclination, and orientation of the drill head. This information is transmitted by radio signals to the other part of the system located at the ground surface where a receiver is used to display this data. The actual position is then compared to the required position on the design path, and deviations, if any, are corrected by moving the slant nose bit and steering the drill head to the desired location.

As the pilot hole progresses, soil cuttings mixed with bentonite increase the stress on the drill pipe. Although rarely used, sometimes a 12-in. (300-mm) washover pipe, following approximately 250 to 300 ft (75 to 90 m) behind the drill head, is rotated over the pilot drill stem to relieve this stress and to provide rigidity to the pilot string. The pilot string and the drill pipe are then thrust alternately along the desired path until they exit at the other end. It should be noted that the washover pipe does not in any way ream the hole or help in the installation of the product line. The washover pipe is used to relieve friction and support the hole and the drill pipe. It is extracted when the product line is installed unless the product line can be installed inside the washover pipe.

The pilot string is then withdrawn through the drill pipe, leaving the drill pipe in place to act as a drawing the subsequent operation. Reaming devices are then attached to the drill pipe and pulled back through the pilot hole, enlarging it to the desired diameter suitable to accept the designed product pipe to be installed. Recommended practice is to select a reamer that is the smaller of 1.5 to 2 times the outside diameter or 12 in. larger than the diameter of the product pipe to allow for an annular void for the return of drilling fluids and cuttings. It will also reduce frictional pullback forces and allow for the bend radius of the pipe. For diameters less than 20 in. (500 mm), the pipe can be attached directly behind the reamer using a swivel device so that the total assembly can be pulled back in one pass. However, even for diameters less than 20 in. (500 mm), many contractors choose to pre-ream the borehole. Pre-reaming widens the pilot hole to a

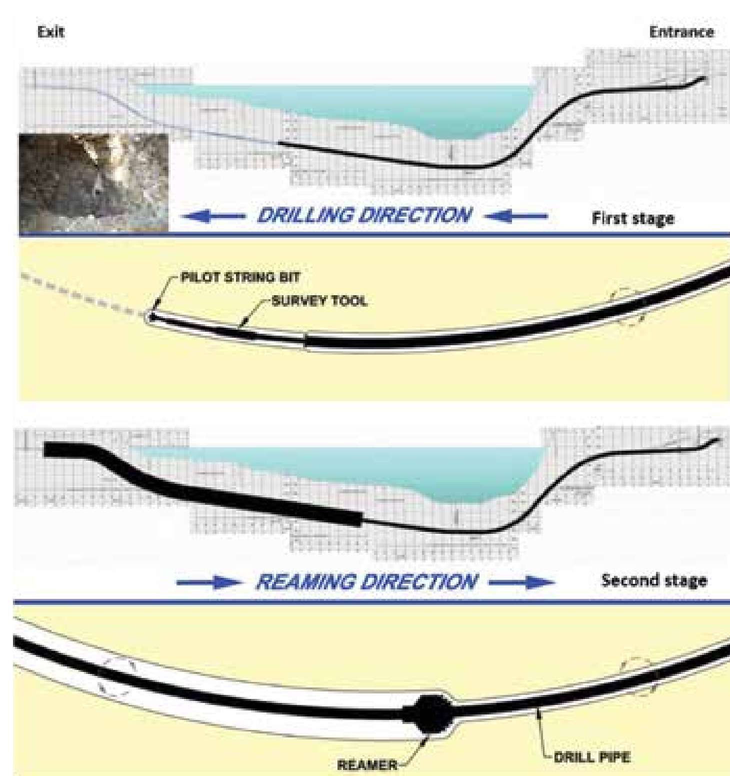

Figure 8.1 The two-stage process in the HDD.
(Courtesy of Brian Dorwart)

Table 8.1 Comparisons of Main Features of Different HDD Systems.

System	Product Pipe Diameter (in.)	Depth Range (ft.)	Bore Length (ft.)	Torque (ft-lb)	Thrust Pullback (lb.)	Machine Weight (With Truck) (ton)	Typical Applications
Micro-HDD	2-6	13	150	600	16,500-23,000	300 lb.	Cables, Gas, Water, Sewer, Laterals
Mini-HDD	2-10	15	600	950	20,000	9	Cables, Pressure Pipelines
Midi-HDD	10-24	75	900	900-7,000	20,000-100,000	18	Pressure Pipelines
Maxi-HDD	24-63	200	5,000	80,000	100,000	30	Pressure Pipelines

diameter slightly greater than the utility diameter. A circular cutting tool is attached to the drill pipe end, which is then rotated by the drilling rig, simultaneously pulling it back along the drilled pilot hole. Bentonite pumped down the drill pipe carries soil cutting to the surface.

This deals mainly with maxi rigs pulling in large diameter welded pipe. It is not applicable in mini-HDD installations. After this process, the pipe is then attached to the drill pipe with special adapters between the two. A fly cutter or barrel reamer and swivel are installed between the drill pipe and the pipe to increase the borehole to the desired diameter, smooth the hole, and ensure that the rotating action of the drill pipe is not transmitted to the product pipe. The pipe is then pulled along the drilled path and installed in position.

8.2 HDD CLASSIFICATION

HDD methods can be divided into four areas:

- Micro-HDD
- Mini-HDD
- Medium or Midi-HDD
- Large or Maxi-HDD (usually called HDD)

For comparison, Micro-HDD is for drive lengths less than 150 ft (45 m) and product line diameters up to 6 in. (152 mm). Mini-HDD is typically restricted to lengths less than 600 ft (182 m), and pipe sizes less than 10 in. (254 mm). Micro and Mini-HDD are used much more on the telecommunication industry for shallow depth installations of underground cables and gas lines. In part because of the limitation of tracking systems for Micro and Mini-HDD, the depth of these installations is limited to around 15 ft (4.5 m). In contrast, the Maxi-HDD systems have collaborated guidance systems and are able to install pipes to depths of 200 ft (60 m). The Mini-HDD systems can be launched from a curb, while the Maxi-HDD systems are giants in comparison, requiring a space of 150 ft by 250 ft (45 m by 75 m), and arrive to the jobsite in as many as ten trailers. Table 8.1 presents a summary of features for different HDD systems.

Because of similarities between type of equipment, application, and method for Micro-HDD and Mini-HDD, and between Midi and Maxi-HDD, in this manual the HDD methods will be divided into two broad categories of Horizontal Directional Drilling (HDD) and Mini Directional Drilling (Mini-HDD).

8.5 HDD MAJOR ADVANTAGES

The major advantage for HDD is the speed of installation combined with minimal environmental effect. This facilitates the construction permit process, thereby saving a lot of time and expense. Long and complicated crossings can be quickly and economically accomplished with a great degree of accuracy since it is possible to monitor and control the drilling operation. Accuracy of this method is approximately ±2 to 10 ft (0.5 to 3 m) depending on the bore length, tracking capabilities, ground conditions, and drill crew skill. Another advantage is that sufficient depth can be achieved to avoid other utilities. In the case of river crossing, the effect of buoyancy, the danger of riverbed erosion, and possible damage from river traffic is eliminated. Another advantage is that access and reception pits are usually not required for this method.

8.6 HDD MAJOR LIMITATIONS

HDD is an extremely specialized operation for which special equipment and a very high degree of operator skill are required. Because the cost of the equipment and the operation are high, the bore length should be sufficient for it to be economically feasible. Although it has been done, this type of boring can be difficult for small slopes and may not be suitable for gravity pipeline applications. Also, the type of pipe installed by this method is limited to that which can withstand sufficient axial tensile force even at the joints, i.e., PVC, steel, or high-density polyethylene (HDPE) pipe. Table 8.2 summarizes potential problems, their possible causes, and actions required to remedy the problems.

Table 8.2 Potential problems and possible solutions for HDD process.[3]

Problem	Probable Cause	Solution
Lost position of drill head	Locator showing inaccurate readings	Check locator performance. Try push and pullback of the drill head to track it.
Difficulties in product pipe pullback	Product pipe pushed into the sidewalls of the curved bore hole	Alternatively push and pull to free pipe
Drill head exits off target	Steering difficulties and/or inaccurate locator	Pull back head reasonable distance and redrill
Back reaming difficulties	Possible blockage due to cobbles or gravel	Push reamer back out. Detach pipe and reamer. Pullback with drill bit to clear obstruction
Steering difficulties	Hit bedrock or a hard layer at steep angle	Drill very slowly to pass through hard ground
Fluid migrates to surface Alignment too tight for product pipe	Fissured rock or hydraulic fracture Difficult steering section	Lower the fluid pressure Enlarge the section of the bore hole
Loss of bore hole stability	Fluid pressure fluctuation between rig and drill face	Increase applied fluid pressure to just below maximum permissible value
Groundwater seepage washes out drilling fluid	High groundwater pressure or low drilling fluid rate	Adjust drilling fluid weight and flow rate
Plugged fluid jets	Debris in drill string	Remove and clean
Separation of drill string immediately behind reamer	Damaged swivel assembly	Blind push backwards and dig up
High drill torque requirements	Worn bit/cutting head	Replace
Increased torque overnight	Collapsed hole/cohesive soil	Drill continuously or rotate periodically overnight
High pullback forces	Radius too small	Flatten drilling path curves
Warning siren and/or flashing lights	Advance Electric Strike system activated because drill head is too close to or struck a live electric underground line	Do not move. Stay on the protected mat. Always wear safety shoes and gloves.

8.7 SOIL COMPATIBILITY

8.7.1 Site Investigations

Site investigations for HDD operations can be broken into two main categories, surface investigations and subsurface investigations. The surface investigations need to address topographical information for horizontal and vertical control, including overbank profiles on centerline back 150 ft (45 m). The hydrographic characteristics of the water body needs to be accurately determined also. This would include fathometer readings 200 ft (60 m) on either side of the centerline of the bore.

Subsurface investigations consist of reviewing available geological information and correlating this information with site specific information to obtain a subsurface profile. Borings should be located approximately 50 ft (15 m) off the centerline and extend at least 30 ft (9 m) below the lowest drill depth.

Location and intervals of sampling will depend on individual job conditions, but should be enough to describe the expected subsurface conditions accurately and adequately. Boring data should include:

1. Standard classification of soils
2. Gradation curves of granular soils
3. SPT "blow counts" where applicable
4. Cored rock with RQD and % recovery
5. Unconfined compressive strength for rock samples
6. Inspection for possible contamination (Hazardous Waste)
7. Groundwater location, type, and behavior
8. Determination of electrical resistivity or mineralogical constituents.

Table 8.3 Applicability of Mini-HDD (or Midi-HDD) for various soil conditions.[2]

Soil Conditions	Applicability	
	Mini -HDD	Midi -HDD
Soft to very soft clays, silts, and organic deposits	Yes	Yes
Medium to very stiff clays and silts	Yes	Yes
Hard clays and highly weathered shales	Yes	Yes
Very loose to loose sands (above water table)	Yes	Yes
Medium to dense sands (below water table)	Yes	Yes
Medium to dense sands (above water table)	Yes	Yes
Gravels and cobbles less < 2 - 4 in. (50.5 – 101 mm) diameter	Marginal	Marginal
Soils with significant cobbles, boulders, and obstructions > 4 -6 in. (101 – 152 mm) diameter	No	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils	Yes	Yes
Slightly weathered to unweathered rocks	Marginal	Marginal

Boring path to follow is determined by:

1. Pipe length
2. Pipe diameter
3. Pipe thickness
4. Entry/Exit angles
5. Required depth below the obstacle
6. Subsurface conditions

"Borings should be located off the drilled path centerline to reduce the possibility of drilling fluid inadvertently surfacing through the borings during HDD operations. The borings should penetrate to an elevation 20 to 30 ft. (6 to 9 m) below the depth of the proposed drill path to provide information for design modifications and anticipated pilot-hole deviations during construction."[1]

8.7.2 Types of Soils

Clay is considered ideal for HDD methods. Cohesion-less fine sand and silt generally behave in a fluid manner and stay suspended in the drill fluid for a sufficient amount of time; therefore, they are also suitable for HDD.

High-pressure fluid drilling system (mini-HDD and midi-HDD) normally do not damage on-line existing utilities and thus are safe for subsurface congested urban areas. Fluid cutting systems, which are most suitable in soft soil conditions, have been used widely in sand and clay formations. Although small gravel and soft rock formations can be accommodated by higher fluid pressure and more powerful jets, steering accuracy might suffer.

Generally, mechanical drilling system (mini-HDD) can be applied in a wider range of soil conditions than fluid jetting methods. A pilot hole can be drilled through soil particles ranging from sand or clay to gravel, and even in continuous rock formations, by using a suitable drill head; however, problems might occur in spoil removal, pilot hole stabilization, and back-reaming operations. Today's technology enables large drilling operations to be conducted in soil formations consisting of up to 50 percent gravel. Applicability of mini-HDD (or midi-HDD) for different soil conditions are listed in Table 8.3.

8.7.3 Ground Movement

In mini-HDD the typical pipe size is less than 100 mm (4 in.). For small pipe line installations, the soil cutting is not removed. Instead, they remain in suspension in the drilling fluid, resulting in compaction of the soil around the boreholes as installation takes place. The drilling fluid is usually under pressure, which helps stabilize soils such as sand and soft clay.

Surface subsidence ordinarily is not a concern with mini-HDD because there is minimal overcutting of the soil. On the other hand, with maxi-and midi-HDD, ground subsidence must be taken into consideration during both the design and construction phases of a directionally drilled crossing. Care must be taken because significant forces are created by the flow rates and pressures at which the bentonite slurry is circulated through the drilling string to operate the down-hole motor and wash the cuttings from the borehole. The typical slurry flow ranges from 280 to 570 L/min (75 to 150 gpm); the typical pressure is 69 kPa (10 psi).

The pressure and high flow rates may cause the slurry to flow into a soil strata, a process called Frac-outs, causing heave of the soil or the surface. The pressure and high flow rates also may cause the soil to erode, thus leaving behind a void that may subsequently collapse and cause a surface settlement. These problems can be eliminated by ensuring that adequate depth is maintained, compatible soil conditions exist, and by closely monitoring the flow rates and pressure of the drilling fluid. The following can increase the risk of inadvertent returns (commonly referred to as "frac-outs"):[1]

- Highly permeable soils such as gravel.
- Soils consisting of loose sands or very soft clays.
- Soil and bedrock materials with very low permeability but jointed or fractured (slickensided clays or rock fractures).
- Clay soils that swell in the presence of fluids.
- Considerable elevation differences between either the entry or exit point and ground elevations along the HDD alignment.
- Disturbed soils such as fill or soils adjacent to piles or other structures.
- Areas along the HDD alignment where depth of cover is less than 40 ft (12 m).
- Locations along the HDD alignment where significant variations in density and/or composition of ground conditions are encountered (i.e., overburden/bedrock contact and other types of mixed-inter-face transition zones).
- Use of inappropriate downhole tooling or drilling practices.

8.8 MINI DIRECTIONAL DRILLING METHOD

The mini directional drilling method is used specifically for the installation of small diameter lines that need to be installed at a reasonable depth and up to 600 ft (180 m) in length. The process involves the creation of a small diameter borehole, either by mechanical cutting or fluid jetting, and then pulling back the utility through the borehole. A slurry is used to stabilize the walls of the borehole in unstable soils and to reduce the frictional drag on the cable or pipeline being installed.

The mini directional drilling method has the ability to locate the position of the drill head and steer it in the required direction. The survey systems used for locating the drill head vary with the manufacturer, but they all serve the same purpose of locating the drill head position so that correction can be made as the boring progresses.

There are several manufacturers of mini directional drilling equipment. The drilling equipment and surveying systems manufactured by these companies varies significantly. The mini directional drilling equipment presently being manufactured in the United States falls into one of the following categories:

1. Controlled fluid jetting method.
2. Fluid assisted mechanical cutting method.

8.8.1 Introduction of Controlled Fluid Jetting Method

The controlled fluid jetting technique uses high water pressure to create small diameter boreholes. The soil is cut by small diameter high pressure jets of water or liquefied clay (bentonite). The jets cut the soil in advance of the boring tool, impregnating and lining the borehole wall with clay.

This method has the capability of monitoring the path and remotely steering the boring tool in the soil. By remotely changing or biasing the direction of the cutting jets at its nose, the boring tool changes directions as it is thrust through the soil. This capability, combined with the electronic tool detection system, makes it possible to align boreholes up to 600 ft (180 m) long, depending on soil conditions. The electronic detection system can measure the tunnel position within 1 in. (25 mm) at normal utility placement depths. If the toll begins to deviate from the desired path, it can be steered to the designed path or pulled back to create a new course. Also, in case of an obstacle, the same principle of backing up to create a new course around the obstacle is applied.

8.8.1.1 Method Description

The controlled fluid jetting process is characterized by a low flow (1 to 2 gpm or 3.75 to 7.5 L/min), high pressure (1,000 to 4,000 psi or 6,890 to 27,560 kPa) fluid cutting system. The soil is cut by small diameter high pressure jets using a mixture of water and bentonite clay. This method can be differentiated from the water jetting and the slurry bore methods by the pressure and flow rates that create the cutting. Depending on soil conditions, two to five jets are used. These jets cut the soil in advance of the boring tool and impregnate and line the bore wall with the bentonite clay. The clay lining maintains a stable opening even in unstable soils such as fine sand. In addition, the clay lining makes the tunnel wall smooth and slippery, greatly reducing frictional drag on the new line as it is being installed.

The equipment is designed so that the cutting fluid's energy rapidly dissipates after it leaves the nozzle. This prevents the cutting fluid from cutting through existing utilities or over-cutting the soil. Since over-cutting is prevented, excessive soil erosion and surface subsidence is minimized.

A significant feature of this method is the remote steering capability of the boring tool. By remotely changing or biasing the direction of the bit at the nose of the boring tool, the tool can be made to change directions as it is thrust through the soil. In addition, there is an electronic tool detection system to accurately locate the position of the drill head. These two features make it possible to maintain the tool on course and provide detailed data on its exact position. If the tool begins to deviate from its designed path, it can be steered back on course. Or if required, it can be backed up 3 to 6 ft (1 to 2 m) and a new borehole created.

The tool detection system can be either a walkover system or an electromagnetic tool detection system. The walkover system consists of a transmitter located in the head of the tool and a locator unit on the surface. The locator unit is moved along the desired course in line with the tool head by a crew member. At any time during the drilling operation, the crew member can determine the path and the deviation of the tool from the course line. The usable depth of the standard walkover system is approximately 30 ft (9 m).

The alternative tool detection system consists of a series of four parallel electric cables that are positioned above the desired tunnel path and secured in place. These waterproof and traffic-proof cables may be weighted down to the bottom of the stream or canal or laid directly on top of a street or highway, thus not interfering with traffic flow. The cables transmit into the earth an electromagnetic signal that is received by the navigational instruments located in the drill head. These instruments locate the position of the drill head in the ground with respect to the center of the cables to a precision of ±0.5 in. (±12 mm) and continuously relays this information back to a computer on the operator's control console. In case of deviation from the design path, the drill head can be steered back to the design path by adjusting the orientation of the jets.

The drilling equipment for this method consists of a power unit and the drill rack. The power unit is track mounted. It provides a means to prepare and pressurize the cutting fluid. In addition, the unit provides electricity and high-pressure hydraulic fluid.

The drill stem can be in sections or it can be one continuous coil. When the drill stem is in sections, the drilling unit is carriage-mounted and can be used up to 325 ft (100 m) away from the field power unit to facilitate operations in congested areas. The unit advances the tool by pushing 10-ft. (3 m) long sections of drill pipe into the ground, producing a 1.75-in. (45 mm) diameter borehole. A three-man crew is generally required to operate the system: one person controls the steering and advances the tool from the drill unit, the second helps connect and disconnect the drill pipe, and the third operates the locator. The crew member performing the locating functions and generating the steering commands communicates with the crew member executing the commands at the drill unit by means of a two-way radio.

When the drill stem is a continuous length of coiled pipe for drilling and pulled back operations, the length of the bore is limited by the usable length of the coil pipe. The drill head is pushed forward by means of unwinding a continuous coil manufactured from a high tensile tubing used in offshore oil drilling applications. An injector/extractor assembly straightens and bends the coil as the tubing is extended or retracted. This coil tubing encloses the electric cables and control wires supplying power and steering data to the electric motor and navigational instruments located inside the drill head.

The coil tubing also transports the jetting liquid from the trailer-mounted storage tank to the rotating jet nuzzles that cut and form the tunnel. After the drill head emerges at the desired location, the rotating cutter is removed and replaced with a pulling apparatus. The cable or other conduit is attached and pulled back through the borehole as the drill head is withdrawn by rewinding the coil. If the utility to be installed requires a borehole larger than 3.5 in. (90 mm) in diameter, a hydro-jet reaming device is installed between the pulling apparatus and the drill head. As the drill head pulling the utility or conduit is being installed into the borehole, the hydro-jet reamer automatically expands the borehole to a size just large enough to allow the utility or conduit to be pulled into the location, thus assuring a snug fit of the conduit in the ground. Undesirable over-cutting of the borehole is eliminated. The lubricating effect of the bentonite keeps the pulling forces on the utility or conduit at a minimum to prevent damage.

More recently, polymer gel has been used in addition to the bentonite slurry. Since only one gallon of gel is required for 800 gallons (3,000 liters) of water, this eliminates the necessity of taking a vehicle or large towing track to mix the bentonite slurry, and makes the unit self-sufficient. The fluid helps support the hole, as well as lubricate the boring, reaming, and pipe pull-back operations.

The basic operating procedure is as follows:

1. The drill unit is properly positioned at the starting location.
2. The drill head compatible with the soil conditions is

selected and loaded onto the thrust frame with the first drill pipe.
3. The proper cutting fluid pressure is set onto the field power unit. The fluid pressure is a function of the soil conditions.
4. The tool is advanced and steered level at the proper depth, using the locator to sense the tool position.
5. More drill pipe is added as the tool advances.
6. After each new drill pipe is advanced into the ground, the tool is located and the operator specifies the steering command for the next length of drill pipe to keep the tool on course.
7. Steps 5 and 6 are repeated until the tool advances to the end of the run.
8. At the end of the run, the drilling head is removed and a reamer is attached to enlarge the borehole for the utility (if necessary).
9. The utility is attached to a swivel, which is attached to the reamer.
10. The drill pipe is withdrawn from the borehole pulling in the utility.
11. Appropriate utility connections are made.
12. All disturbed areas are restored.

8.8.1.2 Main Characteristics

1. **Type of Pipe Installed:** Any type of cable or small diameter pipeline that can be joined together to accept the tensile forces that arise from pulling the pipeline or utility through the borehole can be installed by this method. Among the most common pipes are high density polyethylene (HDPE) pipe, small diameter steel pipe, copper service lines, cable installations, etc.
2. **Pipe Size Range:** The size of pipe or utility that can be installed by this method ranges from 2 in. (50 mm) to 14 in. (350 mm). The system is also capable of installing multiple conduits having same or different diameters in the same run.
3. **Bore Span:** The system can install utilities or pipes up to 600 ft (180 m) without interruption and within the specified tolerance. However, this figure depends heavily on the type of soil and the site conditions.
4. **Disturbance to the Ground:** As discussed earlier, the small-diameter jets of liquid rapidly lose energy after leaving drill head. This prevents over-cutting of the soil, so there is not much gap between the installed line and the excavated hole. Surface subsidence is highly unlikely because no void is left under the installed path or roadway.
5. **Area Requirement:** This method does not require a bore pit for drilling purposes. However, normally a pit is required for connecting the utility at the starting point and the terminal point. The drill stem enters the ground at an angle and is guided into and out of a small connection pit. The drill head can be guided out at any desired point on the ground. Since the equipment is portable and self-contained, it can be moved on and off the site quickly. The equipment is compact and is designed to work in congested areas.
6. **Operative Skill Requirements:** The equipment is designed to be a three-person operation as discussed. Since the method is a high technology system, the operators must have a high degree of skill and must be able to operate the computer sensing devices and interpret the results, so that necessary adjustments can be made. The operator must select the drill heads compatible with the soil conditions and should be able to handle the various situations that develop in the field.
7. **Accuracy:** The drill head can be located to a precision of ±2%. The steering accuracy is up to ±6-in. (150 mm) in most cases. However, if a higher accuracy is desired, an accuracy of ±3-in. (75 mm) can be achieved by reducing the interval at which the location readings are taken.
8. **Recommended Ground Conditions:** The system is ideal for soft soils and has been extensively used in sand and clay formations and recently in hard and caliche type soils. The worst soil conditions for this method are rock and gravel.

8.8.1.3 Major Advantages

One of the major advantages for this method is that the method does not damage existing utilities. In the case of obstacles being encountered, the drill head can be guided around the obstacle. The system does not require any bore pits and only one person is required to operate the equipment. Since the method uses a continuous length of coiled pipe, there are no rods or pipes of fixed length that require connection and disconnection during boring and pull-back operations. This makes the installation faster. The method can install lines up to 16 ft (5 m) depth from the ground surface. Minimum site preparation is required because the system is easy to set up.

8.8.1.4 Major Limitations

Continuous strengthening and rewinding of the coil limit the life of the pipe material mainly due to metal fatigue. Hence the coil must be replaced every six months, or after 200 operations.

Another disadvantage is that the system can install only small diameter lines at the present time. Due to its weight and size, the unit cannot be moved through garden gates to the back yard of houses like other units. Hence, it cannot be used in congested places and is mostly used at the curbside.

8.8.2 Introduction of Fluid Assisted Mechanical Cutting

The directionally controlled fluid assisted mechanical cutting method is used for the installation of small-diameter lines. The system uses a mechanical cutting drill bit to cut the soil while the steering is accomplished by using a slanted nose piece. The equipment is being manufactured by a wide variety of firms under various trade names. These systems use a medium pressure low volume drilling fluid to assist in the drilling process. The method tracks the tool from the ground surface to monitor the accuracy and progress of the bore.

8.8.2.1 Method Description

The fluid assisted mechanical cutting system consists of a drilling frame, mechanical drilling assembly, tracking instrumentation, a transport trailer package, and associated accessories and tools. The drilling frame provides torque and thrust to the boring head. In addition, it houses the hydraulic controls, gauges to monitor the torque and thrust, drilling fluid controls, gauges to monitor the pressure and quantity of fluid flow, and an emergency engine shutdown control. The drill pipe is in lengths of 5 ft or 10 ft (1.5 m or 3 m) depending on the size of the drill rack. The procedure consists of driving one length of the drill pipe, adding another length to it, driving it in and repeating the procedure.

The mechanical drill head consists of the drill assembly with a fluid jet nozzle and a remote transmitter. The nose of the drill head consists of a slant shaped anvil that creates a bias to assist in mechanically steering the system. When it is required to bore along a straight path, the drill head is continuously rotated. If the borehole is required along a curved path the nose is positioned in the direction of the curve and is not rotated while the boring is in progress. This permits the drill head to move along a curved path in the required direction. A medium pressure low volume (1 to 2 gpm or 3.5 to 7 L/min) drilling fluid, usually bentonite slurry, is used during the drilling process. The purpose of the drilling fluid is threefold. First, it cools the drill bit, second it stabilizes the borehole, and third it keeps the cutting in suspension.

The tracking instrumentation consists of a walk-over system. In this system, the drill head is equipped with a remote transmitter located just behind the drill bit. The transmitter is powered by battery and continuously emits signals. These signals can be picked up by a hand-held electromagnetic receiver similar to the conventional cable locator. This hand-held receiver gives data on the position, depth, and orientation of the drill bit. Thus, one crew member must walk along the borehole as it progresses and transmit the information to the operator so that necessary adjustments can be made to align the borehole along its desired path. The system is capable of transmitting data to an accuracy of db. 6 in. (±150 mm) up to a depth of 30 ft (9 m). With enhancements to transmitter and receiver, 50 ft (15 m) in depth is possible.

After the drill bit exits the other end at the desired location, the drilling head is removed, and if necessary, a reamer is attached to enlarge the hole for the utility. The utility is attached to the reamer with a swivel installed between the two to ensure that the rotating action of the reamer is not transmitted to the utility. The reamer enlarges the borehole to the required size as it is pulled along the drilled path and the utility is installed in position.

The transport trailer package consists of the power unit, hydraulic pumping station, and the drilling fluid mixing tank. The units are self-powered so that they do not have depend on any external source of power. The power unit can be located to a distance of up to 100 ft (30 m) away from the drill rig, which makes the rig accessible even in tight spots (such as back yards), leaving the power unit near the main road.

8.8.2.2 Main Characteristics

1. **Type of Pipe Installed:** Any type of cable or small diameter pipeline that can be joined together to accept the tensile forces that arise from pulling the pipeline or utility through the borehole can be installed by this method. Among the most common are high density polyethylene pipe, PVC pipe, small diameter steel pipe, copper service lines, cable installations, etc.

2. **Pipe Size Range:** The size of pipe or utility that can be installed by this method ranges from 3 in. (75 mm) to 12 in. (300 mm).

3. **Bore Span:** The system can install utilities or pipes up to 500 ft (150 m) without interruption and within the specified tolerance. However, much depends on the type of soil and the site conditions.

4. **Disturbance to the Ground:** For the installation of lines up to 4 in. (102 mm) in diameter the system does not

remove soil cuttings. Rather, the cuttings remain in suspension in the drilling fluid and the soil is thus compacted around the borehole as utility installation takes place. The drilling fluid is also under pressure, which stabilizes soils like sand and soft clay. Surface subsidence does not take place because there is minimal over-cutting of the soil.

5. **Area Requirements:** This method does not require a bore pit for drilling purposes. However, normally a pit is required for connecting the utility at the starting and terminal points. The drill stem enters the ground at an angle and is guided into and out of a small connection pit. The drill head can be guided out at any desired point on the ground. Because the equipment is portable and self-contained, it can be moved on and off the site quickly. The equipment is compact and is designed to work in congested areas.

6. **Operative Skill Requirements:** The equipment is designed to be three-person operation. Since the method is a high technology system, the operators must have a high degree of skill and must be able to operate the computer sensing devices and interpret the results so that necessary adjustments can be made. The operator must select the drill heads compatible with the soil conditions and should be able to handle the various situations that develop in the field.

7. **Accuracy:** The drill head can be located to a precision of ±6 in. (150 mm). The steering accuracy is up to ±12 in. (30 mm). However, if a higher accuracy is desired, it can be achieved by reducing the interval at which the location readings are taken.

8. **Recommended Ground Conditions:** The system is ideal for soft soils and has been extensively used in sand and clay formations. The worst soil conditions for this method are rock and gravel.

8.8.2.3 Major Advantages

The major advantage of this method is its steering capability. In the case of obstacles being encountered, the drill head can be guided around the obstacle. The system does not require bore pits and only three men are required to conduct the operation. The method can install lines up to 30 ft (9 m) and with enhancements to 50 ft (15 m) in depth from the ground surface. Minimal site preparation is required because the system is easy to set up.

8.8.2.4 Major Limitations

This method cannot be used to install lines for depths greater than 50 ft (15 m) because the range of the electromagnetic receiver is limited. Because the cutting head consists of a drill bit, the system can cut through existing utilities unless one is very careful. Hence, locating all existing utilities before starting the operation is very important.

Note:

Small drill rigs (less than 25,000 lb. thrust) are often used to install small-diameter HDPE pipes [1]. Mini-HDD construction has a tremendous amount of versatility (flexibility) to provide installations around existing facilities. This type of installation allows the depth of the new facility to vary a great deal (i.e., 3 to 30 ft).

Whenever an HDD construction installation method is planned, it is strongly recommended that the utility/entity requesting the installation require an "as-built" showing a precise horizontal bore path and profile showing accurate depths of newly installed facilities. An as-built that is accurately transferred on engineering records will go a long way to help future line locators to locate accurately. This will also help a contractor's pothole to expose these facilities.

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CHAPTER 9
Maxi Horizontal Directional Drilling Methods

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Photo courtesy of American Augers

9.1 INTRODUCTION OF THE HDD METHOD

Horizontal Directional Drilling (HDD) is a technique used to directionally drill a bore hole or small tunnel under a surface obstruction such as a waterway, roadway, crossing utility or other obstruction to pull a pipe or cable through a drilled underground bore hole. HDD began in the early 1960s and can be distinguished by its relative ability to steer along the desired path of the borehole through the soil. Initially it was used to lay small lines over short distances under a surface obstruction, but with each success and continued development, it grew in capabilities. Current HDD projects lay large-diameter, long-distance lines of many different types with minimal surface disturbance. HDD developed rapidly in the United States after the late 1980s and early 1990s and has grown to be common in all types of pipe and cable installations.

Maxi HDD methods involve steerable boring systems using various techniques depending on the bore size and soil structure. In most cases for installing large product, HDD requires a pilot bore then a multi-stage process requiring multiple reaming passes, swabbing pass and product installation pass as shown in Figure 9.1. The first stage involves drilling a pilot hole approximately 6 to 13 in. (150 to 330 mm) in diameter from one side of the obstacle to the other along the design bore path. Typically, this bore path follows the intended installation path of the installed product, such as the centerline of the proposed pipeline. For large HDD, a drill pipe with diameter ranging from about 5-1/2 to 6-5/8 in. (140 to 168 mm) is used. After completing the initial pilot bore, the multi-stage process begins with a first reaming pass followed by subsequent reaming passes until reaching the desired hole diameter. A swabbing process is then used to clean all remaining solids from the bore hole. Lastly, the product pull installation is performed.

The HDD unit is setup on the jobsite by moving it into position and inclining the drill frame to the desired angle, typically between 10 to 18 degrees. The setup and bore path must account for setback distances for both the entrance and exit, to account for pipe bend to the desired depth. Next the drill unit is held in place or anchored to the ground to allow it to withstand the push and pull forces generated by the drill. Next the down hole tools including the steering head and navigation sections are connected to the front of the drill string. Once set up, the bore begins by the HDD unit rotating and pushing each drill rod one after another into the ground. The pilot hole is typically bored along the pre-designed bore path passing safely under the obstacles and boring to the surface tie-in location.

The location and configuration of a bore path are determined by many factors. Some factors are the soil type and underground conditions, the entry and exit angles, the design radius of curvature of the product pipe and drill pipe, any surface and underground obstacles and any other borepath complications, and desired vertical depth of cover over the product pipe. Figure 9.1 shows a detailed example of a bore plan. The selection of the HDD unit depends on factors such as the bore path design, the underground soil structure (ranging from compactible soil to hard rock), the bore length and product size, including physical parameters of the product pipe, the surface and underground obstacles along the bore path, the desired navigation system, and finally the jobsite area and layout.

Based on the previously discussed factors, the selection of drill unit and downhole tools determine the steering and drilling method. In the case of smaller bores in good soil, a compactible drill bit is often used where a slanted faced bit is pushed or maneuvered through the soil to cause steering; and when desiring to bore straight the rod is simply rotated into the ground. In the case of larger rigs and harder soils including rock, a rock drilling system must be used. For rock drilling and other types of difficult soils, mud motors are employed to drive

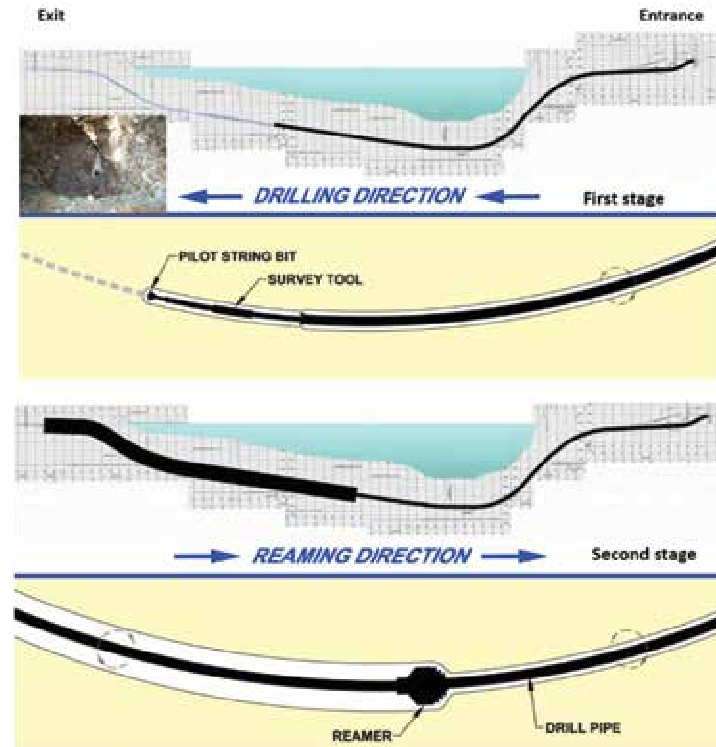

Figure 9.1 The two-stage process in HDD.
(Courtesy of Brian Dorwart)

a host of drill bits selected for the soil conditions. Mud motors are powered by drilling fluid pumped from the surface through the drill pipe. The fluid passes through a power section on the motor that drive the rotating bit. Steering is accomplished by positioning a bent sub behind the bit in the direction you want to go and sliding the rotating bit forward without rotating the drill string. The drilling fluid containing Bentonite serves multiple purposes including, cooling the bit and electronics, maintaining borehole integrity, and functioning as a conveyor to transport solids out of the bore hole to the surface. On large bores, drilling fluid may require very large volumes of drilling mud, often requiring 200 – 500 gpm (750 - 1,900 lpm) for pilot bores and 300 – 1,200 gpm (1,135 – 4,500 lpm) for backreaming. A mud cleaning system is used to separate the usable drilling fluid from the bore hole cuttings. The usable drilling fluid is pumped downhole over and over until it becomes contaminated by excessive fine cuttings at which time it is disposed of on the jobsite or transported to an appropriate dump site.

HDD steering is accomplished using one of several basic forms of navigation, wireline, and walkover navigation.In general, navigation is accomplished by determining the position and orientation of the drill head relative to the bore plan. On smaller jobs with surface access, navigation can be accomplished by walkover tracking technology which uses an electronic beacon located directly behind the bit. The beacon trans-

Table 9.1 Comparisons of Main Features of Different HDD Systems.

System	Product Pipe Diameter (in.)	Depth Range (ft.)	Bore Length (ft.)	Torque (ft-lb)	Thrust Pullback (lb.)	Machine Weight(lb.)	Typical Applications
Small-HDD	2-6	50	700	4000	40,000	<15,000	Cables, Gas, Water, Sewer, Laterals
Medium-HDD	6-24	200	2,000	4,000-20,000	40,000-100,000	<60,000	Gas, Water, Pressure Pipelines
Large-HDD	24-63	200	5,000	>20,000	>100,000	45 tones	Pressure Pipelines

mits information in the form of radio signals to the surface to indicate depth, position, and orientation of the steering tool. The walkover locator picks up this information directly over the transmitter, where it is transmitted to the HDD unit and operator. On maxi HDD systems, navigation is typically done by wireline navigation. There are multiple steering technologies for steering a pilot bore. The type of steering technology used on a bore is usually determined by the operator depending on the project conditions and operator preference. All the technologies can complete most bores to the engineered bore plan and provide an as-bored report for traceability. Communication from the steering tool to the surface is accomplished through a connected wire routed through the drill pipe passed through a collection device on the rotary on the drill unit. The steering operator views the information through a computer and uses this information to determine if a steering adjustment is needed. If steering is needed, the operator positions the steering tool to the proper roll orientation and proceeds to steer by pushing forward for a short section of pipe. If no steering is needed, the operator simply bores straight for a short distance. This process is repeated at least once or twice for each joint of drill pipe, pipe after pipe until the entire bore is completed.

Once the pilot hole is complete, backreamers are attached to the drill pipe. Depending on the size of the product pipe, one or more backreamers in increasing diameters are sequentially pulled or pushed through the hole to enlarge it near its final diameter. Depending on the jobsite and available space, typically prior to the pipeline pullback operation, the pipeline must be made up in one full length. Typically for large pipelines, the pipe must be welded, X-rayed, coated, and tested, etc. before installation. Then it is normally positioned on rollers in line with the drilled hole to minimize coating damage, control excessive stress and limit excessive bend radius from being imposed on the pipe as it is being pulled into the bore. Finally, numerous downhole pulling tools are attached between the drill pipe and the pipeline. These include special adapters, a backreamer and a swivel. There are many types of backreamers for different soil conditions. The backreamer cleans and smooths the final borehole to prepare it for the controlled movement of the pipeline. The swivel is used to ensure that the rotating action of the drill pipe and backreamer are not transmitted to the pipeline. The pipeline is then slowly pulled along the borehole and installed in its final position.

Note that HDD processes described in this document are highly abbreviated; additional operational and safety details are outlined in various references [1-6] and in the HDD equipment manufacturer's operations manual and through training available at most major equipment dealers.

9.2 HDD CLASSIFICATION

HDD methods and capabilities can be divided into three classifications.

- Small-HDD
- Medium or Midi-HDD
- Large or Maxi-HDD

For comparison, Small HDD is for bore lengths less than 700 ft (210 m) and product line diameters up to 8 in. (200 mm). Small HDD are typically used on product ranging from telecommunications cable and small pipe to small distribution pipe, done in good soil conditions and limited depth, and use walkover tracking. Medium-HDD can install pipe and cable up to 2,000 ft (600 m) in both good soil and rock for pipe sizes up to 16 in. (400 mm) or more. These rigs typically use walkover tracking but can also be setup with wireline guidance systems. Finally, Large HDD or Maxi rigs are used to install pipes to depths of 200 ft (60 m) and well over a mile (1.5 km) in length in some cases. The differences between the smallest and largest rigs are vast. Small HDD rigs can often be hauled to the jobsite behind a one-ton truck on a single trailer and be maneuvered and setup in the backyard behind a house. Large HDD rigs might require a space of 150 ft by 250 ft (45 m by 75 m) for setup and arrive to the jobsite in as many as five to ten semi-trailers. Table 9.1 presents a summary of features for different HDD systems.

9.3 HDD EQUIPMENT DESCRIPTION

The major piece of equipment required for the installation of the pipeline is the directional drilling rig. This discussion will focus primarily on Large HDD used to install pipelines over long

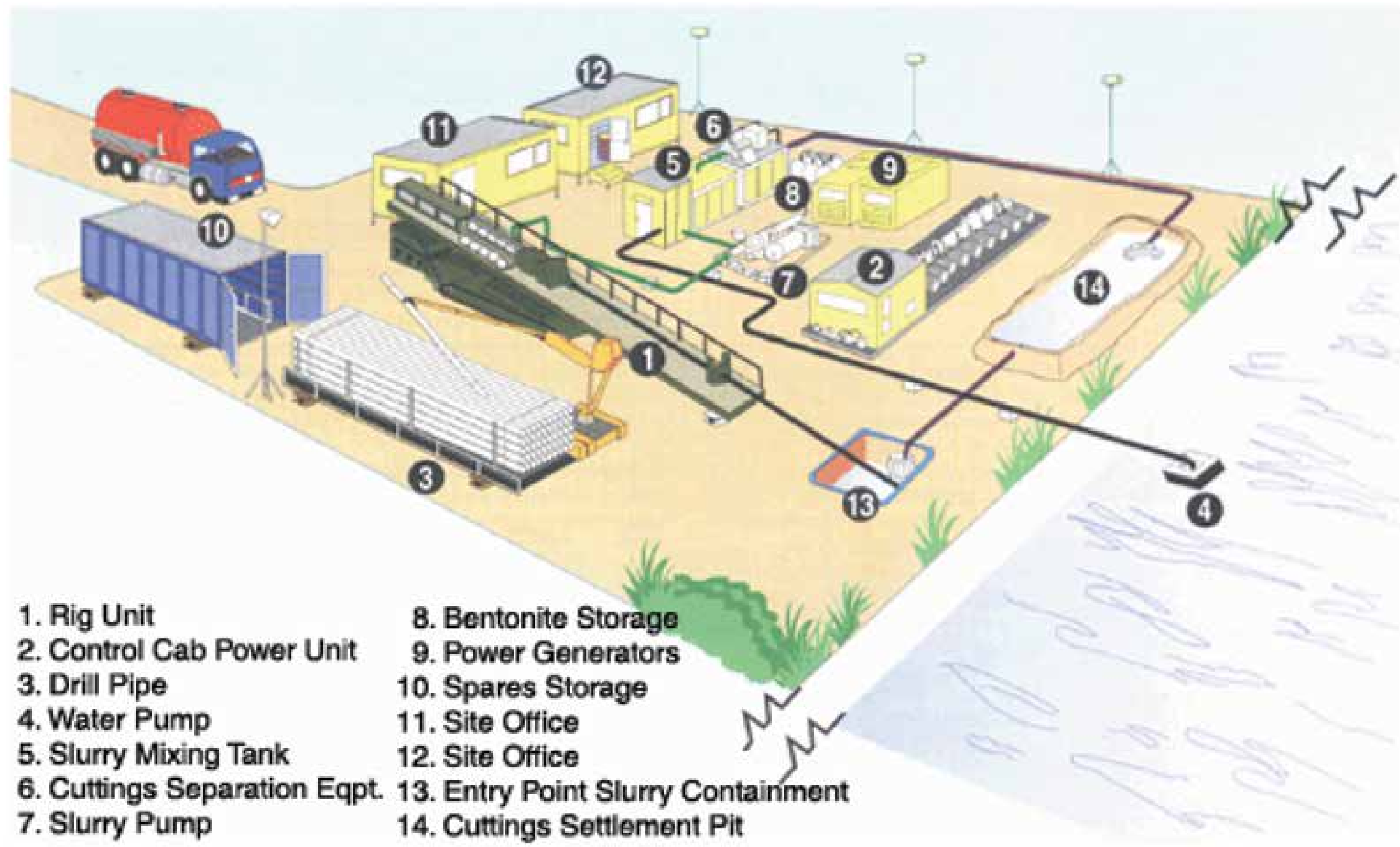

Figure 9.2 A representative maxi HDD site layout.

distances. Most large HDD rigs are built for the job and designed for relatively easy transportation and rapid site erection. Typical large to very large HDD drilling rig are comprised of:

1. Drill frame 50 to 80 ft (15 to 25 m) long and 8 ft (2.5 m) wide, weighing in excess of 45 tones with a pullback capacity of 500 tones (2,225 kN). It can provide up to 100,000 ft-lbs. (135,581 N-m) of dynamic torque.
2. Two skid-mounted power packs, which provide all the power and fluid flow/pressure requirements.
3. 100-to-250-ton units are often configured on tracks with an onboard engine. This configuration is popular for mobilization and setting up in tight or hard to access locations.
4. Drilling fluid cleaning systems are sized for the differing sizes of maxi systems. These mobile systems mix and clean the drilling fluid as well as provide the volume of fluid the expanding bore hole requires. A maxi drill site also requires multiple 20,000-gallon freshwater storage tanks for makeup water and drilling fluid storage.
5. Control cab. This houses the drillers control console and survey instruments.
6. Multiple pipe racks. A skid-mounted rack that is 40 ft (12 m) long by 8 ft (2.5 m) wide positioned alongside the rig for storing pilot string and drill pipe.
7. Two containerized workshops/messing facilities.

9.4 HDD MAIN CHARACTERISTICS

1. **Type of pipe installed:** The type of pipe installed is limited to one that can be joined together, so that it can accept sufficient axial tensile forces to permit it to be pulled through the borehole. Steel pipe is the most common type of pipe being used for large pipeline jobs, at present. However, high density polyethylene pipe and fusible PVC is becoming more common and often used as well, especially for smaller diameter pipe.
2. **Pipe size range:** The size of pipe that can be installed by this method ranges from 3 in. (75 mm) to more than 54 in. (1,375 mm). Multiple lines can also be installed in a single pull but is typically limited to small diameter pipes. The installation procedure is the same with the bundle being pulled back as a single unit along the pre-reamed profile. A significant multiple line crossing is a 2,800-ft

(850-m) of five separate lines ranging in size from 6 in. (150 mm) to 16 in. (400 mm).

3. **Bore span:** The bore span ranges from 1,000 to 12,000 ft (300 m to 3,600 m). Longer bores are less common and require emerging techniques to complete. Typically, shorter lengths are only economically feasible because of highly difficult soil conditions and for large pipe sizes.
4. **Ground disturbance:** Ground disturbances must be taken into consideration both during the design and the construction phase of a directionally drilled crossing. Planned execution of drilling fluid is required on every HDD project. It is important that fluid management is elevated to critical status to ensure project success. Highly trained and skilled drilling fluid personnel with the right sized cleaning and pumping systems are paramount to project success and profitability. Keeping the drilling fluid clean and free of solids is important to removing solids from the bore hole so productivity can be carried out throughout all phases of the installation. Solids in "clean" drilling fluid causes premature wear on fluid stream components and increases the risk of inadvertent returns. Inadvertent return mitigation includes keeping correct drilling fluid, pre-bore soil profiles, annular pressure monitoring, rapid pressure loss recognition, containment procedures and eliminating unplanned stops in the boring process. Ground disturbances can be a detriment to HDD installations, but proper planning and execution can minimize these disturbances.
5. **Area requirements:** The directional drilling process does not necessarily require access pits or receiving pits but can be required in certain conditions. Preferably, the rig working area should be reasonably level, firm, and suitable for the movement of large heavy equipment. Typically, an area of 250 ft (75 m) in length by 150 ft (50 m) in width is considered sufficient for the operation. A suitable access road should be provided. At the exit end of the bore, sufficient room is required for return fluid systems, drill pipe handling and storage, exit rig if utilized and the equipment to properly handle the product pipe as it is installed in the borehole. Modern Maxi HDD jobsites are being squeezed for space and will require careful execution to keep the site safe.
6. **Operator skill requirements:** The directional drilling method is a sophisticated system and hence requires highly skilled operators. The operator must be knowledgeable concerning downhole drilling, impact of drilling in various geological formations, sensing and recording instrumentation, interpreting computer display information, etc. As mentioned in the Ground Disturbance section, highly trained and skilled fluid maintenance personnel are critical to project success.
7. **Accuracy:** The accuracy of installation will depend on the survey system being used, jobsite and underground conditions and the relative skill of the operator. Steering systems have evolved to improved accuracy, but they still require competent operators to achieve these accuracy levels.
8. **Ground conditions:** Clay is considered to be an ideal material for the directional drilling method. However, cohesionless sand and silt generally behave in a fluid manner and are able to stay in suspension in drill fluid for a sufficient period of time to be conveyed out of the borehole. However, as HDD equipment capabilities and experience have progressed, even the hard to bore conditions such as compacted gravel, solid and broken rock and even deposited cobble, though difficult can be successfully bored with proper planning and operation.

9.5 HDD MAJOR ADVANTAGES

The major advantage of HDD is the ability to efficiently install pipe and cable under obstructions such as roadways and rivers, resulting in limited surface obstruction and reduced environmental damage. Often HDD is about the only feasible method of installation available for certain applications. This facilitates the construction permit process, thereby saving a lot of time and expense. Long and complicated crossings can be quickly and economically accomplished with a great degree of accuracy since it is possible to monitor and control the drilling operation. The HDD method has progressed to being able to meet increasingly stringent accuracy specifications on difficult extended installations. Crew skill levels have greatly increased due to the number of projects completed every year. Advancements in both the HDD equipment and the guidance equipment have extended the lengths and sizes of product that can be successfully completed with HDD.

9.6 HDD MAJOR LIMITATIONS

HDD operation requires highly trained operators, controlling highly complex special equipment with a very high degree of operator skill required. Because equipment costs and the operating costs are high, the bore diameter and length should be of sufficient size to be economically feasible. Although it has been done, this type of boring can be difficult for highly controlled pitch bores and may not be suitable for gravity pipeline applications. Also, the type of pipe installed by this method is limited to that which can withstand enough axial tensile force even at the joints, i.e., mainly steel or high-density polyethylene (HDPE) pipe or fusible PVC (Polyvinyl Chloride). Table

Table 9.2 Potential problems and possible solutions for HDD process[3]

Problem	Probable Cause	Solution
Lost position of drill head	Locator showing inaccurate readings or interference from other sources	Check locator performance. Try push and pullback of the drill head to track it. Calibrate locator for high noise floor before launch.
Difficulties in product pipe pullback	Product pipe pushed into the sidewalls of the curved bore hole	Alternatively push and pull to free pipe. Use a larger reamer diameter.
Drill head exits off target	Steering difficulties and/or inaccurate locator	Pull back head reasonable distance and re-drill if possible. Slow down bore to improve path following.
Backreaming difficulties	Possible blockage due to cobbles or gravel, improper drilling mix or flowrate, or too rapid pull rate.	Push reamer back out. Detach pipe and reamer. Pullback with drill bit to clear obstruction. Slow down and allow reamer to improve soil mixing and cuttings flow.
Steering difficulties	Hit bedrock or a hard layer at steep angle.	Drill very slowly to pass through hard ground.
Fluid migrates to surface Alignment too tight for product pipe	Fissured rock or hydraulic fracture Difficult steering section, improper fluid mix or flowrate	Lower the fluid pressure Enlarge the section of the bore hole and allow time for fluid flow to re-establish.
Loss of bore hole stability	Fluid pressure fluctuation between rig and drill face or incorrect fluid mix recipe	Increase applied fluid pressure to just below maximum permissible value. Add proper additives.
Groundwater seepage washes out drilling fluid	High groundwater pressure or low drilling fluid rate	Adjust drilling fluid weight and flow rate.
Plugged fluid jets	Debris in drill string	Remove and clean
Separation of drill string immediately behind reamer	Damaged swivel assembly	Blind push backwards and dig up.
High drill torque requirements	Worn bit/cutting head	Replace
Increased torque overnight	Collapsed hole/cohesive soil	Drill continuously or rotate periodically overnight.
High pullback forces	Radius too small	Flatten drilling path curves

9.2 summarizes potential problems, their possible causes, and actions required to remedy the problems.

9.7 SOIL COMPATIBILITY

HDD has grown in capabilities to drill in most any soil type ranging from the soft clay loam to the hard rock formations, including the most difficult formations of flowing sand to glacial till. Each requires the correct HDD unit capabilities, down hole tooling and operator experience. Drilling in these more difficult formations often presents many unforeseen obstacles. Operators will be challenged to create solutions on the fly as these challenges arise. Occasionally even the most experienced crew with the best equipment can fail in some situations.

9.7.1 SITE INVESTIGATIONS

Site investigations for HDD operations can involve a great number of factors, but for purposes of this introduction, can be broken into two main categories, surface investigations and subsurface investigations. The surface investigations need to address topographical information for horizontal and vertical control, including surface survey profiles along the intended path, the setup and exit sites, and the pipeline fabrication area. Subsurface investigations might include vertical borehole sampling to determine soil type geological characteristics, water table and other factors that affect success of the bore. Location and intervals of subsoil sampling will depend on individual job conditions but should accurately and adequately describe the expected subsurface

Table 9.3 Applicability of Mini-HDD (or Midi-HDD) for various soil conditions.[2]

Soil Conditions	Applicability	
	Small-HDD	Medium-HDD
Soft to very soft clays, silts, and organic deposits	Yes	Yes
Medium to very stiff clays and silts	Yes	Yes
Hard clays and highly weathered shales	Yes	Yes
Very loose to loose sands (above water table)	Yes	Yes
Medium to dense sands (below water table)	Yes	Yes
Medium to dense sands (above water table)	Yes	Yes
Gravels and cobbles less < 2 - 4 in. (50.5 – 101 mm) diameter	Marginal	Yes
Soils with significant cobbles, boulders, and obstructions > 4 -6 in. (101 – 152 mm) diameter	No	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils	Yes	Yes
Slightly weathered to unweathered rocks	Yes	Yes

conditions. Boring data should include:

1. Standard classification of soils
2. Gradation curves of granular soils
3. Cone penetrometer or "blow counts" for characterizing soil strength if applicable
4. Cored rock samples with RQD (Rock Quality Designation) and % recovery
5. Unconfined compressive strength for rock samples
6. Inspection for possible contamination (Hazardous Waste)
7. Groundwater location, type, and behavior
8. Determination of electrical resistivity or mineralogical constituents.

Boring path to follow is determined by:

1. Pipe length
2. Pipe diameter
3. Pipe thickness
4. Entry/Exit angles
5. Required depth below the obstacle
6. Subsurface conditions

Soil sample borings should be located off the planned bore path centerline to reduce the possibility of creating weak spots in the soil for drilling fluid to surface through the test holes during HDD operations. The test holes should penetrate to an elevation 20 to 30 ft (6 to 9 m) below the depth of the proposed drill path to provide information for design modifications and anticipated pilot-hole deviations during construction.[1]

9.7.2 TYPES OF SOILS

Soil type and subsoil conditions play a heavy role is the difficulty of any bore. Clay loam soil and other types of compressible soils are considered ideal for HDD methods. Though some types can result in difficulty with the incorrect drilling fluid mixture. Drilling in flowing sand, high water table and unstable soils can result in complicated bores because of the difficulty in maintaining a quality borehole. Consulting with your drilling fluid supplier and proper fluid component selection can solve most issues. With today's HDD drill, boring through moderate strength rock that is contained in homogeneous formations is commonly done with good success. Drilling through subsurface conditions that change frequently, contain rock shelves, or cobbles and boulders laid down by glaciers are among the most problematic drilling conditions. It typically requires extensive planning, highly experienced drill crews and properly executed "in process" adjustments to complete the project. The prudent operator would be wise to incorporate every good practice available to be successful.

9.7.2 Underground Obstacles

One common complication encountered more often today is the congestion caused by other underground lines. In most cases these must be avoided, and by regulation and best practice must be exposed by "daylighting" or opening up an excavation to identify the exact position of the obstacle and observe the drill head pass by the obstacle line. Local regulation will control this process and the operator must be familiar with any regulations. In case where the line cannot be exposed high-pressure fluid drilling systems normally do not damage existing utilities and thus are safe for subsurface congested urban areas.

9.7.3 Drilling Fluid Disposal and Inadvertent Returns

Drilling fluid is a vital part of the HDD process of installing pipe and cable. This is especially true with large HDD where many tens of thousands of gallons of fluid may be pumped down hole during the bore. Any loss of drilling fluid which unintentionally flows to the soil surface above the bore path is typically called Inadvertent Returns. Job owners, local regulators and the public have become highly sensitive to the environmental concerns of Inadvertent Returns. Recent research[6] indicates that in most cases HDD drilling fluid is benign from a contamination perspective, though it can cause water pollution from turbidity. This is especially true in highly sensitive areas such as wild rivers and other bodies of pristine water where any drilling fluid is considered a pollutant and to be avoided if possible. Care must be taken because significant forces are created by the flow rates and pressures downhole where drilling fluid is circulated through the drilling string to operate the down-hole motor and convey the cuttings from the borehole. The pressure and high flow rates may cause the slurry to flow into a soil-strata, causing heave of the soil or leakage to the surface; or can result in underground soil erosion, leaving behind a void that may subsequently collapse and cause a surface settlement. Typically for small HDD, in non-sensitive areas, this is not a problem if proper techniques are used. But for larger bores using medium-and large-HDD, the high fluid flow rates and long distances can result in erosion or hydraulic fracture of the soil above the drill string, resulting in fluid being expressed to the surface Therefore, soil hydraulic fracture susceptibility and/or soil erosion and subsidence must be taken into consideration during both the design and construction phases of a directionally drilled crossing. These problems can be generally reduced by ensuring that adequate depth is maintained, compatible soil conditions exist and by closely monitoring the flow rates and pressure of the drilling fluid. The following can increase the risk of inadvertent returns (commonly referred to as "frac-outs").[1]

- Highly permeable soils such as gravel.
- Soils consisting of loose sands or very soft clays.
- Soil and bedrock materials with very low permeability but jointed or fractured (slickensides clays or rock fractures).
- Clay soils that have a tendency to swell in the presence of fluids.
- Considerable elevation differences between either the entry or exit point and ground elevations along the HDD alignment.
- Disturbed soils such as fill or in soils adjacent to piles or other structures.
- Areas along the HDD alignment where depth of cover is shallow.
- Locations along the HDD alignment where significant variations in density and/or composition of ground conditions are encountered (i.e., overburden/bedrock contact and other types of mixed-inter-face transition zones).
- Use of inappropriate downhole tooling or drilling practices.
- Poor fluid management practices and execution.

9.8 SUMMARY

Maxi HDD has progressed to a "standard" method for longer and larger pipeline, electrical, water and sewer installations. The number of operational systems, experienced crews and equipment advancements have greatly improved the success and economic viability of the method.

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CHAPTER 10
Pipe Ramming Methods

10.1 INTRODUCTION

The Pipe Ramming method is a trenchless installation method that uses a pneumatic or hydraulic percussion hammer to drive a new steel pipe into the soil along the design path (horizontal direction). There are a variety of different methods used to remove the soil/spoils from the inside of the steel pipe that had been hammered in place such as auguring, jetting, or inducing air. The most common application for the pipe ramming method is often in conjunction with pipeline construction. It is also used under railway and highway embankments in certain ground conditions to help limit installation risk related to certain soil conditions. The diameters vary from small (approximately 12 in. +/-) to large (approximately 144 in. +/-) but these crossings are usually 200 ft or less in total length. Shorter distances allow the effective use of pipe ramming to install the steel pipe on the intended design path. Pipe ramming can be used in longer installations; however, line and grade can venture from the intended design path as distance increases.

In critical line and grade installations, a pilot tube can be used in advance of pipe ramming to help maintain line and grade. Pipe ramming can be used after

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Photo courtesy of HammerHead Trenchless

the pilot tube installation to drive large diameter steel pipes on line and grade. The combination of pilot tube and pipe ramming has opened markets to use the pipe ramming technology on projects that typically would have used other trenchless methods due to the previous limitations concerning line and grade control on longer runs. In short trenchless crossings less than 150 ft in distance, pipe ramming has proven to be cost effective in comparison to other installation techniques. Additionally, pipe ramming is much quicker to set up than other installation methods. This method is most valuable for installing larger pipes at shorter distances and installing at shallower depths. It is suitable for use in most ground conditions except solid rock.

Figure 10.1 Hydraulic leveling jacks being used to support casing for line and grade.

10.2 DESCRIPTION OF THE PIPE RAMMING METHOD

Most of all pipe ramming is completed using an open-ended steel pipe to allow the spoil to enter the casing. However, there have been a few exceptions where a pipe can be driven by having the leading end of the pipe closed or in a wedge cone shape. This pipe ramming method displaces the soil and is only used in small diameter applications from 2 to 6 in. When the pipe is driven with the leading edge closed, it behaves like a rod having a diameter equivalent to the pipe diameter. This can be compared with the compaction method. For pipes 8 in. (200 mm) or larger, the leading end is usually left open. In this case, a band is installed around the outside and inside edge of the leading section of pipe called a soil shoe. This serves a dual purpose: (1) it reinforces the leading edge, and (2) it decreases the friction both inside and outside the casing. The band installed on the inside edge of the leading section reinforces and reduces the amount of soil that enters the inside of the pipe. Reducing the amount of soil coming inside of the steel pipe will help during the cleanout process, and reduces friction inside the pipe. Once the type of leading edge is determined, the main work pit is constructed and the lengths of pipe to be rammed are chosen. The longer the pipe lengths, the less welding that is required to assemble the steel pipe together.

The size of the main work pit required can vary based off the lengths of steel pipe. This versatility plays a critical role when choosing a trenchless method. If sufficient room is available, the main work pit is constructed to enable the entire length of steel pipe to be driven as one single unit. In this case, the welding is accomplished above ground before the start of the ramming. Longer lengths (60 ft or 18 m and above) may require down pressure at various locations along the pipe. This down pressure will eliminate any bouncing of the pipe, and it will also hold the pipe to the line and grade established during the setup. Anything longer than 80 ft in distance at one time can present multiple challenges in dealing with the pipe weight and the engagement of the steel pipe into the soil. Recoil from the hammer in longer lengths of pipe can reduce the effectiveness of the installation.

Most projects will start by ramming a 20-ft or 40-ft length of steel pipe on line and grade. The contractor will then weld up the remainder of the pipe and try to drive it to reduce welding and clean out time. In the case where the work area is congested, the pipe is installed in various lengths depending on the available size of the main work pit.

Once these lengths are established, pipe supports are chosen. For situations where the line and grade are not critical, the pipe can be supported by construction equipment (such as backhoes, cranes, and side-boom tractors), wood or block supports, or it can be supported directly on the pit floor. In cases where the line and grade are critical, the pipe is supported by adjustable hydraulic cradles, launch cradle platforms, I-beams, or auger boring machine tracks. Once the pit is constructed, the pipe supports are set, the pipe to be driven is set to proper line and grade, collets are installed, and the ramming tool is locked into collets which are locked into the steel pipe.

After providing adequate support to the pipe and hammer, the hammer is cinched into place by welding eyebolts/lugs on to the steel pipe being driven. These eyebolts/lugs are used to hold the straps, chains, or hoists linked to the ramming tool. In some cases, the tool itself is locked into the collets/drive cone in the steel pipe, without the use of straps. After the tool is fired in forward position the striker locks the hammer into the collets and steel pipe.

Compressed air supplied from an air compressor is used

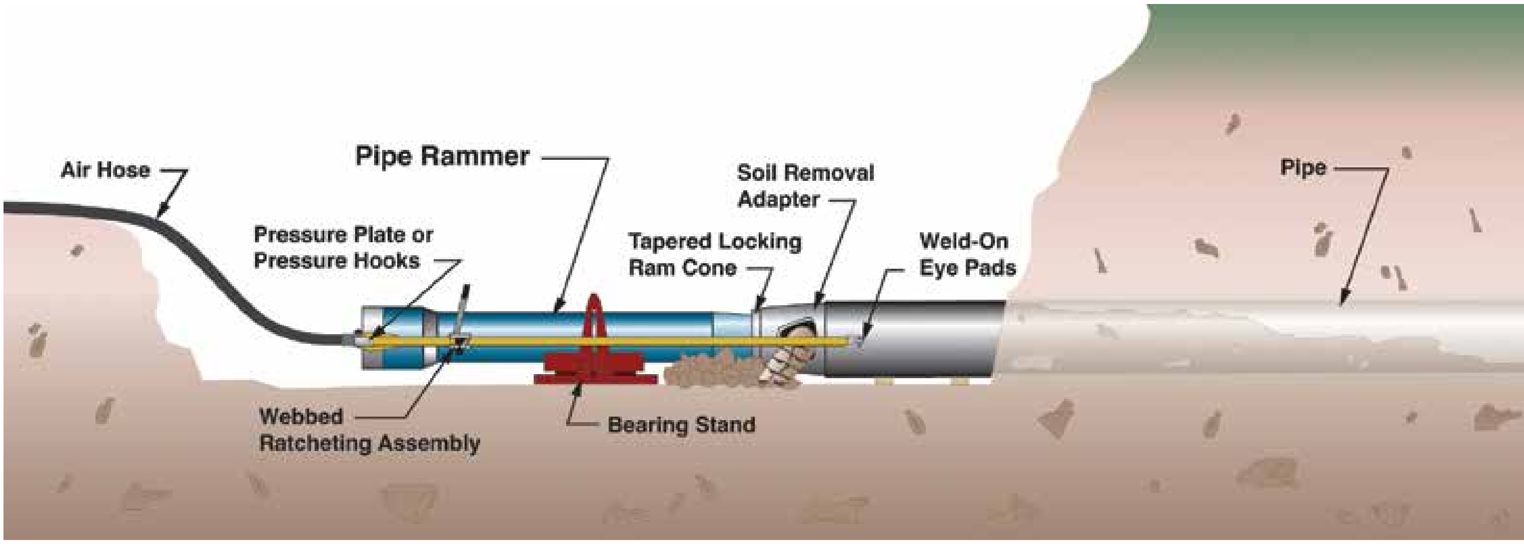

Figure 10.2 The Pipe Ramming Process.

as a power source for the pneumatic hammers. However, as with percussion tools, the power source could also be hydraulic. With the power source connected and the line and grade checked the ramming operation can begin. For ramming operations with the pipe face closed, the soil is compressed around the pipe as it is being rammed. These installations are typically used for small diameter installations less than 6 in. There is no spoil removal in this case. For the open face pipe, the leading edge cuts a hole equal to the diameter of the leading-edge band. The spoil enters the pipe, is compacted, and forced to the rear of the pipe. In some cases, a spoil removal port is installed or cut in the steel casing near the hammer. The spoil can then be extruded or washed through the opening in the port as the ramming proceeds. Lubrication to the front of the head or displaced throughout the inside of the steel casing can help the tooling drive the pipe further, remove the spoils from inside the casing, and lubricate the OD of the steel casing. In certain soil conditions (such as stiff clays or sands), small 3/8-in. or 1/2-in. galvanized steel pipe is installed on the top of the rammed pipe being installed at a point approximately 24 in. (600 mm) behind the front edge of the pipe. This line is used to supply polymer, bentonite, or other drilling lubricants inside and/or outside of the pipe to facilitate spoil removal, reduce friction, and maintain the integration of the hole being cut. These fluids can also potentially aid in soil removal once the ramming is completed. After each section of the pipe is rammed, another segment is welded on and the procedure is repeated until the whole length of the pipe is completed.

Cleaning out the pipe can be done by a variety of methods. These methods include using a pipe cleaning pig or plug, high-pressure air or water, jet cleaning, pumping, HDD, or auguring. Many of the large diameter (66 in. to 144 in.) pipe rams have utilized skid steers and mini excavators to clean the spoils from the pipe. After completing the clean-out process the pipe is ready to be used for the utility installation.

10.3 SOIL COMPATIBILITY

10.3.1 Soft Clays, Silts, and Organic Deposits

Inner and outer rings at normal positions, outer ring is 3 in. (76 mm) wide, welded around the casing with bottom 6-10 in. (152-254 mm) not covered. The inner ring is 3 in. (76 mm) wide and welded around the entire circumference of the casing. Drilling fluids should be used to maintain the whole integrity and reduce friction, usually on the inside only. In some cases, usually on small casing (2-6 in.), the steel pipe can have a closed end. This closed end front of the steel pipe is designed to look like a cone. This design functions much like the smaller piercing tools.

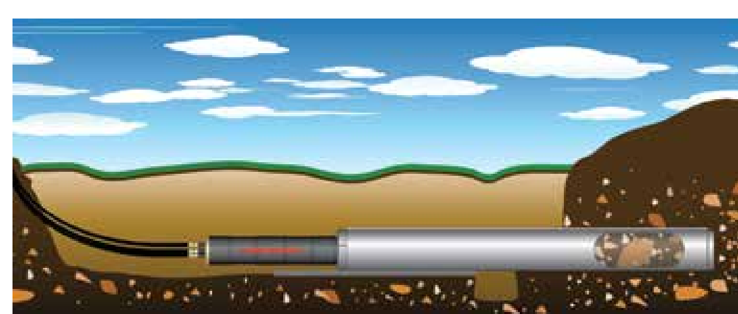

Figure 10.3 Another illustration of a pipe ramming method.

10.3.2 Medium to Very Stiff Clays and Silts

Inner and outer rings on the casing. Depending on the grade requirements, the outer ring can vary. One option is to increase the width of the outer ring from the standard 3 inches to 10, 12 or even 14 in. (76, 254, 300 or 350 mm). This type of adjustment makes a slightly longer overcut on the leading edge which can allow the pipe to maintain a truer grade. Also, an outer band can be placed further back on the casing (12-18 in. or 300-460 mm). By moving the band back, the front of the casing can get a secure "bite" into the clays before being overcut. Both options should help get the critical first section of pipe started on the required grade (which is the most important part). All the measurements mentioned above can be adjusted as needed. In addition, different types of bentonite or polymers can be used. These lubricants, applied through a small pipe on top of the casing, will help reduce the friction created during ramming, and they will help keep the hole open as the casing advances. (This is like the fluid used in HDD).

10.3.3 Hard Clays and Shales

Inner and outer rings placed at the normal location. Use drilling fluids on the outside of the pipe. Occasionally, the pipe that is carrying the fluid for the outside of the pipe being driven can be designed so a portion of the fluid is directed inside of the pipe. By placing fluids inside, the spoils removed will move back through the casing with less friction.

10.3.4 Loose Sands Below Water Table

In addition to an outer and inner ring, a plug may be installed. This plug can be made by filling the front 8-10 ft. of the casing with sandbags. These sandbags will slow down the soil and water entering the casing, thereby helping to prevent a potential void outside of the pipe during the installation, otherwise caused by the soil flowing freely into the pipe being driven by the hammer. Care should be taken to watch the grade of the pipe as it goes in to make certain the pipe is not nose diving as it enters the bank with the plug installed. Dewatering may be necessary at the entry and exit pit for this soil condition so that the face of the main work pit can be opened to start the pipe into the ground.

10.3.5 Medium to Dense Sands Above Water Table

Inner and outer rings are in normal positions. Bentonite should be used both inside and outside of the casing. The use of fluids is important in sands above the water table. During the ramming process, the casing temperature increases. This temperature increase tends to dry up the small amount of moisture that exists in the sand. Once the water has dried up, the sand can become cemented in some cases. This cemented sand can halt the ramming or cause the casing to veer off the designed line and grade. Thus, bentonite (and even plain water) can help prevent the formation of cemented sands. The use of the plug described earlier is optional, depending on whether the sand has the possibility of flowing or running.

10.3.6 Medium to dense sands below water table

See "10.3.4 Loose Sands Below Water Table."

10.3.7 Gravels and Cobbles (2-4 in. or 50-400 mm) or Larger (Weathered to Unweathered Rocks)

Use an outer and an inner band on the lead section of pipe. These bands can be beveled to create a sharper edge on the casing. This band helps direct the cobbles and the soil inside of the pipe. It also acts like a knife or splitter against the rock. This sharp edge then helps split or fracture the cobbles that are encountered. Generally, drilling fluids are not necessary in rocky situations.

10.3.8 Cemented Soils

Inner and outer rings are normal position with a sharp beveled edge. Drilling fluids are optional since the integrity of the hole is usually not compromised. In this type of soil, different types of auger or drilling teeth can be used. These teeth (like those on a rock bit used in auger boring) can be welded around the outside of the casing. The spacing of the teeth is variable depending on the rigidity of the soil. These teeth act as chipping points on the leading edge of the pipe. The points aid in fracturing the cemented soils encountered. This is one of the hardest soils faced when ramming.

10.4 PIPE RAMMING MAIN CHARACTERISTICS

10.4.1 Type of Pipe Installed

The type of pipe installed by the pipe ramming method is limited to steel due to the application of loads on the pipe. The thickness of pipe used varies; however, a thicker than normal size is recommended. Check with equipment manufacturer to see what pipe wall is recommended based on the tool to be used.

10.4.2 Pipe Size Range

Pipe ramming is job-specific according to needs of the project. Design, final product, soil conditions, contractor experience, length of crossing, and tooling all help to determine the appropriate size for each project. Pipe Ramming technology has improved and is able to install pipes from 8 in. up to 144 in. Multiple applications of pipe ramming have been utilized with this rapidly advancing industry.

Table 10.1 Recommended Wall Thickness for Pipe Ramming.

Outside Diameter of Pipe		Recommended Wall Thickness - Bores Exceeding 65 ft (20 m)	
in.	mm.	in.	mm.
4	100	0.188	4.78
6	150	0.27	7.1
8	200	0.27	7.1
10	250	0.27	7.1
12	300	0.27	7.1
14	350	0.31	8
16	400	0.31	8
18	450	0.39	10
20	500	0.39	10
22	550	0.47	12
24	600	0.47	12
26	660	0.55	14
28	710	0.55	14
30	750	0.62	16
32	815	0.62	16
36	900	0.62	16
42	1,050	0.7	18
48	1,200	0.7	18
54	1,350	0.78	20
60	1,500	0.87	22
66	1,650	1	25
72	1,800	1	25
78	1,980	1	25
84	2,100	1	25
96	2,400	1	25
108	2,700	1.25	31.75
120	3,000	1.25	31.75
144	3,600	1.5	38.1

**Minimum wall thickness recommendations should be verified and designed/modified for specific ground and project conditions.*
**Recommended minimum wall thickness is greater than required in the AREMA Manual.*
**Check the AREMA Manual, AREMA requires a thicker-walled casing for E80 loading.*

10.4.3 Crossing Span

Pipe ramming can install pipes up to 400 ft (120 m) in length. However, the bore span, diameter of pipe, and most importantly the soil conditions must be considered to determine the length that is able to be installed and provide an acceptable outcome for the installation to line and grade requirements.

10.4.4 Disturbance to the Ground

In most trenchless installation methods, there is a cutting head to overcut and allow the pipe to slide into place. By using the pipe ramming method, the pipe is installed without excavating at the face of the steel pipe. This method generally does not result in any significant disturbance to the ground since there is no over-excavation in the soil. The leading edge of the pipe performs the cutting action. The cutting action thus forms a soil plug at the leading edge of the pipe. The soil plug is particularly helpful in unstable or free flowing soils because the pressure at the cutting face compacts the soil around the hole and prevents excessive loosening of the soil. In addition, any obstructions that are encountered, such as boulders or debris, are absorbed by the steel pipe, and not displaced outside of the pipe (generally), thereby avoiding any swelling or sinking of the ground surface. However, as with any trenchless method, extreme care must be taken to avoid disturbing the surface or existing utilities. It is possible for the ground to heave during the installation if an obstruction is encountered and there is limited cover. It is also possible that if an obstruction is encountered in certain soil conditions in an embankment that the embankment can move laterally. These are not common and only occur in very specific conditions, however the risks should be discussed prior to commencing the installation or finalizing the design of the project.

10.4.5 Area Requirements

The pipe ramming method requires a main work pit to install the pipe. The size of the pit is a function of the length of the pipe being installed, the size of the ramming tool selected, the depth at which the pipe is placed, and the size of the working area available. This method requires adequate surface area. There should be enough to accommodate the main work pit and, occasionally, enough room for a spoil stockpile. In addition, room should be made available for the equipment required to place the ramming tool and the pipe, a location for the power source, a welder, and a place for lubricating equipment and supplies (if fluids are being used).

10.4.6 Operator Skill Requirements

The pipe ramming method requires a high degree of operator skill. The operator must decide what kind of set up is needed for the crossing, what type of soil shoe should be built, how to remove the spoils, whether lubrication should be used, what type of lubrication should be used, and what lengths of pipe should be rammed. In addition, the operator must provide proficient line and grade alignment, and avoid damaging any existing utilities throughout the crossing. The operator must be able to recognize the existing utilities that have been located and whether these utilities should be exposed (it is encouraged that all utilities to be crossed should be exposed and that size and placement are verified so they will not conflict with the installation). The operator must recognize when obstructions are encountered and how to address the obstructions while still being able to complete the installation. The more the operator is informed about the Geotechnical Evaluation and potential obstructions, the better they can create a contingency plan to address any potential issues.

10.4.7 Accuracy

The accuracy of the pipe ramming method to a great extent depends on the initial setup. Once the ramming has begun, there is a limited amount of control in changing the direction of the pipe installation. Accuracy of line and grade for this method is approximately ±1% over 100 ft. Proper pipe set up and monitoring the line and grade at the commencement of hammer operations will increase the potential accuracy for the installation. Occasionally, a wedge or shoe can be placed in larger diameter pipes at the leading edge and at the required location to help redirect the pipe. This shoe is generally made of metal or wood. Also, grade control of the pipe can be aided by raising or lowering the back end of the pipe, thereby allowing the front end to move in the opposite direction. The removal of the soil at various times during the installation of the pipe reduces the weight of the casing. This reduction in weight can reduce friction, and can help continue driving the pipe further in length. When critical line and grade are required a pilot tube/pipe ram combination will always provide the most accurate results.

10.4.8 Recommended Ground Conditions

The pipe ramming method can be used in almost all types of soil conditions. The leading edge of the pipe being driven with the outer and inner bands will cut through stiff clays, compacted soils, and cobbles or boulders. In the case of unstable soils, the pressure created at the leading edge of the pipe, along with the soil plug that is formed will help reduce the chance of a void or the loss of an excessive amount of soil.

10.5 PIPE RAMMING MAJOR ADVANTAGES

The pipe ramming method is an effective method for installing medium to large diameter pipes. The versatile pit sizes, varying lengths of pipe that can be installed, and ability to handle almost all types of soil conditions, make this method a practical and economical technique for installing pipe. This method may not require any thrust block/structure because the ramming action is self-contained in the hammer with a striker providing the percussive force. The pipe ramming method is also multi-functional. A single ramming tool and air compressor can be used to install a wide variety of pipe lengths and diameters. Ramming can also be

Figure 10.4

Figure 10.5

used for vertical pipe driving, angular ramming, or pipe replacement. Ramming tools are frequently used to assist directional drilling operations both during pullback and to install conductor barrels, drill stem recovery, pipe removal/bore salvage, and pullback assist. Figures 10.3, 10.4, and 10.5 illustrate ramming methods used throughout the underground trenchless industry.

The Pipe Ramming method has the advantages of simple construction procedures, potential reduced construction time, good quality, high efficiency, and low cost.

10.6 PIPE RAMMING MAJOR LIMITATIONS

The major disadvantage of the pipe ramming method is the minimal amount of control over line and grade. Therefore, the initial setup is of major importance. Distances over 200 ft should take extra precautions to ensure the installation is completed. Also, in the case of obstructions like boulders or cobbles, the pipe may be deflected, especially with small diameter pipe. Therefore, enough information on the existing soil conditions must be available to determine the proper size of pipe to be used. Also, the hammer produces significant noise and vibration at the face of the steel casing.

REFERENCE

Pipe Ramming Projects

1. *ASCE Manual and Reports on Engineering Practice No. 1152020.*
2. *Simicevic, J., Sterling, R. L. Guidelines for pipe ramming. Trenchless Technology Center of Louisiana Technological University Technical Report, 2001b.*

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Photo courtesy of Trenchless Technology

CHAPTER 11
Pipe Bursting

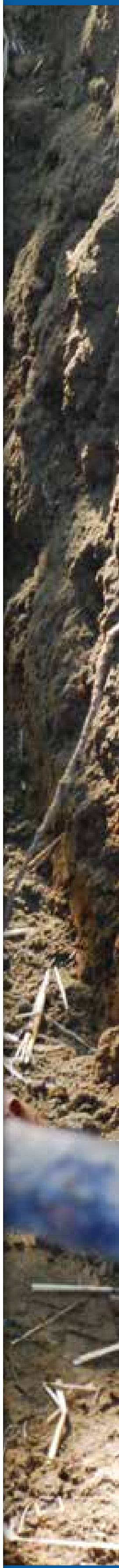
11.1 INTRODUCTION

Pipe bursting is a proven method for replacing and/or rehabilitating existing pipelines that have met the end of their useful design life or do not meet desired capacity. The method was established in the late 1970s and was patented by British Gas. After the patents expired in 2005, the method has become one of the more popular trenchless pipe replacement and rehabilitation techniques. Advancements in the pipe bursting method have led to growth throughout the water, sewer, and gas industry.

Pipe bursting is, generally, considered to be both a pipe rehabilitation technique and a pipe replacement technique. For this reason, the method is associated with other trenchless pipe rehabilitation techniques like Cured-In-Place-Pipe (CIPP) or sliplining. In municipal rehab/ replacement bids pipe bursting might be an alternate approved method bidding against CIPP. Pipe bursting is unique in that the pipe is replaced with the same diameter or larger diameter pipe in its place. The pipe bursting process breaks, splits, cuts, and eventually displaces the existing pipe into the surrounding soil while pulling in a completely new replacement pipe.

11.2 BENEFITS OF ALL TRENCHLESS REHABILITATION/REPLACEMENT TECHNOLOGIES

New construction and utility replacement is becoming more challenging every day as the easements



become more congested. Trenchless methods are on the rise to limit inconvenience to the public and provide pipe replacement methods for cost effective alternatives to methods like open cut. Pipe bursting offers the following advantages over open cut replacement.

- Using the existing pipe corridor to replace pipe on the same alignment and grade. New easements are not required.
- Reduces financial costs associated with restoration, paving, and backfilling an excavated trench
- Typically, reduces social costs associated with disruptions to traffic patterns, commerce, and use of public and private spaces
- Reduces time required to complete the work
- Reduces stress on the community with ever increasing construction projects disrupting life
- Reduces carbon footprint required to replace an existing pipe
- Reduces the impact to trees and landscaping
- Safer due to less excavations, open trenches, and utility strikes

11.3 BENEFITS OF PIPE BURSTING OVER OTHER TRENCHLESS REHABILITATION TECHNIQUES

Pipe Bursting is technically considered a replacement method, however many times pipe bursting is included as a rehabilitation technique for comparison purposes, because it offers many of the same advantages as those techniques. Pipe bursting offers advantages that the other trenchless rehabilitation techniques do not. Unlike all the other trenchless technologies used for rehabilitating/replacing existing pipe, pipe bursting offers the following additional advantages the other trenchless methods do not.

- Pipe bursting installs a new factory manufactured pipe.
- Pipe bursting is the only trenchless technology that can increase or maintain the diameter from the existing pipe to the new pipe, and therefore, significantly increasing the pipes capacity.
- The condition of the existing pipe has little impact on the pipe bursting process, therefore, virtually any pipes in poor condition are a candidate for pipe bursting.
- Only trenchless technology that provides a new product pipe with a 100-year design life in the same path.
- Pipe replacement materials can handle high pressure requirements.
- Pipe replacement materials meet the requirements of potable water and natural gas standards.

11.4 PIPE BURSTING DESCRIPTION

Pipe bursting is defined as a trenchless pipe replacement and/or rehabilitation method in which an existing pipe is fractured or split by using an internally applied mechanical force. This internally applied force takes place where an expander or cutting mechanism contacts the existing pipe material. The existing pipe material is displaced along with the surrounding soil to create a temporary cavity through which a new pipe is pulled through the displaced pipe and soil, simultaneously. The expander and cutting mechanisms are pulled and/or pushed through the existing pipe material by a combination of a winch and pneumatic hammer or hydraulically powered static bursting machine.

The displacement of the existing pipe material and surrounding soil permits the pipe bursting method to create cavities large enough to pull in new pipe sizes equal to or greater than the previously existing pipe diameter. The limitation of potential upsizes is a function of the physics of the important variables involved in each pipe segment to be pipe burst. The most important variables in this equation are as follows.

- Amount of force applied by the equipment;
- Existing pipe material being replaced;
- Depth, Soil conditions surrounding the existing pipe;
- Amount of upsize desired;
- Amount of time the temporary cavity will stay open
- Length of segment to be replaced; and
- Repair Sections, Pipe Material Changes, and Estimated Pipe Drag for desired launch pits.

The combination of these variables effects the extent to which the pipe bursting equipment can displace the existing pipe material and soil to create and hold open the temporary cavity long enough to pull in the new pipe. Unfortunately, the predictability of how the various combinations of variables will interact among each other is limited. While engineering firms and equipment manufactures continue to address pipe bursting needs throughout the country there is no equation or formula that will always dictate the success or failure of a pipe bursting project. In many cases, the data for the potential variables is impossible to properly quantify.

Consequently, the potential amount of upsize that can be expected in an area is generally based upon the experience of the owner, engineer, and contractor. Of course, the limits of those expectations have been probed by trial and error over the years and norms have been established based upon experience. The International Pipe Bursting Association (IPBA), a division of National Association of Sewer Service Companies (NASSCO) has published a simple table categorizing the degree of difficulty for some of the combinations of variables. These categories are as follows.

- Class A – Minimal difficulty – Size-On-Size Pipe Bursting – Refers to replacing an existing pipe with a new pipe of similar ID. An example would be pipe bursting an existing 8-in. VCP with a new 8-in. HDPE pipe.
- Class B – Moderate difficulty – Single-Upsize Pipe Bursting – Refers to increasing an existing pipe with a new pipe that has an ID larger by approximately one nominal size. An example of this would be pipe bursting an existing 8-in. VCP with a new 10-in. HDPE pipe.
- Class C – Comprehensive difficulty – Double/Triple-Upsize Pipe Bursting – Refers to increasing an existing pipe with a new pipe that has an ID larger by approximately two (double) or three (triple) nominal sizes. Examples of this would be pipe bursting an existing 8-in. VCP with a new 12-in. or 14-in. HDPE pipe.
- Class D – Developmental – Undefined – Triple Upsize or More – With other complicating variables. An example would be pipe bursting an existing 8-in. VCP with a new 16-in. HDPE pipe in densely compacted clay soils.

While the table was, purposefully, an oversimplification of the combination of variables and their impact on probabilities of success, the point of the IPBA Classification Chart is to communicate that there are variables that impact the probabilities of success and that owners, engineers, and contractors should understand and acknowledge these potential issues and risks when considering this method. The IPBA Classification Chart clarifies the concepts involved but is not necessarily useful as a scientific decision matrix, as it is sometimes applied.

11.5 PIPE BURSTING METHODS

Pipe bursting systems are generally classified into two categories: (1) pneumatic pipe bursting systems and (2) static pipe bursting systems. Contractors choose between these two methods based upon preference, availability, application, existing pipe material, pipe material being installed, soil conditions, noise, and risk mitigation.

Pneumatic Pipe Bursting Systems

Pneumatic pipe bursting systems utilize a combination of a pneumatic hammer powered by compressed air, a winch, and an expander at the front or rear of the pneumatic hammer. The hammer, as it is called, is driven through the existing pipe by a relatively large piston fired by compressed air. The compressed air is fed to the back of the hammer by air hose running from an appropriately sized air compressor. The cone shaped expander is attached to the new pipe being pulled into place and set against the pneumatic hammer, either at the front or rear of the hammer. The percussive action of the hammer drives the expander through the existing pipe. The expander maintains contact with the existing pipe and soil and is the mechanism for creating the temporary cavity for pulling in the new pipe. The winch cable is attached to the pneumatic hammer to provide additional pulling force and to keep the expander in contact with the existing pipe and maintaining the proper direction. The tension supplied by the winch and cable is necessary to keep the hammer properly aligned in the pipe so that the system does not navigate outside the existing pipe corridor.

The hammer, expander, and new pipe are inserted into the existing pipe at one end, while the winch is typically located at the opposite end of the pipe segment to be replaced. The winch cable is snaked through the existing pipe and attached to the hammer. An excavation pit is required at one end to accommodate the bursting process. This excavation is typically called the insertion or entry pit. Once the hammer, expander, and new pipe have reached the end of the segment to be replaced, the hammer and expander must be removed. Depending upon the type of pneumatic equipment used, an additional excavation might be required to remove the equipment at this end. This excavation is typically called the retrieval pit. A retrieval pit to remove hammer and burst head are not needed in many cases which provides another benefit to the bursting method.

One of the reasons why pneumatic pipe bursting systems are popular, is the ability for the hammer to reverse back down the newly installed pipe back to the entry pit, and expander to be removed without excavating a retrieval pit. This adaptation is called a Reversible Pneumatic Pipe Bursting. This adaptation is configured in such a way that the hammer fits inside the pipe being installed with an expander seated at the front of the hammer. The winch is placed over a manhole entry point with the cable snaked down the manhole and through the pipe with a boom section and pully system. The expander is pulled into the manhole where it can be accessed and removed. The point where the cable is attached to the hammer can also be accessed and disconnected. The expander is brought out through the manhole and the hammer is pulled back through the newly installed pipe and removed at the insertion pit. Consequently, the entire process can be completed with only the one excavation, at the insertion pit. The potential economies realized by avoiding this second excavation are as follows.

- Saved time in performing the excavation and hauling off material from the retrieval pit
- Saved costs of backfilling the retrieval pit – compaction and/or backfill stone
- Saved costs of restoring the retrieval pit – landscaping,

grassing, asphalt, concrete
- Eliminated inconvenience to public for excavation

While pneumatic pipe bursting systems are used more often than static pipe bursting systems, it is important to note that pneumatic pipe bursting systems are only suitable for installing HDPE pipe. If any other new pipe material is desired, static pipe bursting systems will be necessary. In a majority of cases HDPE is specified as the pipe replacement for water, sewer and gas.

Static Pipe Bursting

Static pipe bursting systems are classified by hydraulic fed pulling system utilizing a rod, cable, or chain. The pulling machine pulls on an expander which transfers this force into to the existing pipe and surrounding soil. The existing pipe and soil are displaced to create the temporary cavity through which the new pipe, attached to the expander, is pulled through.

The hydraulically powered static machine is typically set at one end of the segment where chain, rods, or cable are snaked through the existing pipe and attached to the expander and new pipe. The excavation where the machine is placed, the rods/cable enter the existing pipe and are pushed through the pipe, is typically called the machine pit. The excavation where the expander and the pipe are attached to the rods or cable and inserted into the existing pipe is called the insertion pit. Once the machine rods or cables are attached to the expander, the expander and pipe are pulled through the existing pipe by brute force.

The cable static burst systems are typically used on small diameter pipes (6 in. or less) with short pull lengths. Some cable systems can be used though manholes in much the same way a reversible hammer can be used to eliminate excavation of a machine pit. For larger diameters and longer pull lengths more power is generally desired and therefore both a machine pit and an insertion pit excavation are required to accommodate the larger equipment.

Static pipe bursting systems are required when any pipe material other than HDPE, is desired or required. The pneumatic systems can only be used with HDPE pipe, because of the percussive force involved. The percussive force of a pneumatic system is simply incompatible with the other new pipe materials. Static pipe bursting systems are also advantageous when cutter attachments are required or desired for repair clamps and non-fracturable pipe. Additionally, static pipe bursting systems are desirable in high water table scenarios and to mitigate risk.

11.6 PIPE BURSTING APPLICATIONS & SIZE RANGES

Courtesy of HammerHead Trenchless

The pipe bursting method can be used to replace most pipe, in most any condition, used for most any application. The method can also be used for potable water applications, in addition to non-potable applications. The versatility of the process and the pipe materials installed is quite extensive. Pipe busting can be used for pressure applications and gravity applications, ranging in size from 4 to 54 in.

- Pressure Pipes
    - o Water Pipes (Potable), Water Pipes (Non-Potable), Sewer Force Main Pipes, Gas Pipes, & Process Pipes
- Gravity Pipes
    - o Sanitary Sewer Pipes, Storm Sewer Pipes, & Process Pipes

11.7 EXISTING PIPE MATERIALS THAT ARE CANDIDATES FOR PIPE BURSTING

The very first feasibility test to be considered when evaluating the pipe burst process for use in a particular application is existing pipe material. It is important to verify that the pipe bursting process has the capability to either fracture the existing pipe material easily or cut/split the existing material with an attachment. Proper consideration of the existing pipe material would guide the installer on tool selection and potentially render the method infeasible if there is no proper way to cut or split the existing material. For this reason, the existing pipe material is categorized into two types for consideration.

Fracturable Pipe Material

- Asbestos Cement Pipe
- Cast Iron Pipe
- Concrete Pipe
- PVC Pipe (Thin Wall)

- Truss Pipe
- Vitrified Clay Pipe (VCP)
- Pitch/Orangeburg Pipe

Non-fracturable Pipe Material

- Corrugated Metal Pipe
- Cured-In-Place Pipe (In addition to the existing host material type)
- Ductile Iron Pipe
- Hobas
- Polyethylene/HDPE
- PVC (Thick Wall)
- Steel
- Wood

11.8 EXISTING PIPE MATERIALS THAT ARE CHALLENGING CANDIDATES FOR PIPE BURSTING

There are some existing pipe materials that are best avoided when considering the pipe bursting process. For various reasons, there are materials that do not fracture, cut, or split very well with current pipe bursting tooling or they are heavily reinforced. These materials are as follows.

- Long Runs of Corrugate Metal Pipe (CMP) & Corrugate Plastic Pipe (CPP)
- Ball & Socket Ductile Iron Pipe
- Bar-wrapped Non-cylinder Reinforced Concrete (BWRCP)
- Mechanical Joint Ductile Iron Pipe
- Pre-stressed Concrete Cylinder Pipe (PCCP)
- Reinforced Concrete Cylinder Pipe (RCCP)

11.9 POTENTIAL ISSUES RELATED TO THE EXISTING PIPE MATERIAL

As discussed in section 11.8, there are some existing pipe materials that are best avoided regarding the pipe bursting process. While the pipe bursting process has been used to replace the existing pipe materials discussed in section 11.7, there are additional considerations that designers, engineers, and owners should consider when choosing the pipe bursting process to rehabilitate some of these materials. Designers, engineers, and owners should consider and acknowledge the following when specifying the pipe bursting process.

Fracturable Pipe Materials

Fracturable pipes are very good candidates for pipe bursting and can be accomplished with either a pneumatic or static pipe bursting configuration. These materials pose little challenge to the pipe bursting process and are the best candidates for the pipe bursting process. Consequently, the pipe bursting process can be specified for "fracturable" pipes with a high degree of confidence and little further concern regarding the existing pipe material. These materials are good candidates for upsizing, subject to normal subsurface limitations.

Non-Fracturable Pipe Materials

Non-fracturable pipes can still be good candidates for the pipe bursting process, but typically require specialized tooling to cut or split the existing pipe material. Depending upon how each of these materials react to the cutting/splitting process, can dictate the machine size needed to replace the host pipe. Complications and limitations need to be known and potentially addressed when evaluating a pipe bursting project. The following potential issues related to pipe bursting some non-fracturable pipe should be considered and acknowledged by the designer, engineer, and owner prior to specifying the method.

CIPP, Hobas, PVC & Wood Pipes

Cured-in-Place-Pipe, Hobas, PVC, & Wood pipes typically cut and/or split with little effort. With these materials, simply adding a cutting edge to the expander contacting the material is all that is needed to cut or score the existing pipe material just as the force of the expander begins the displacement process. Once the material is cut or scored and force is applied, the material is easily displaced into the surrounding soils to allow the new product pipe to be pulled through the newly formed void. The displacement of these pipe material does not impede or damage the new product pipe and does not typically impede the potential ability of the pipe bursting process to upsize subject to the normal limitations of upsize. When considering pipe bursting Cured-in-Place-Pipe, considerations must be given to the existing material of the host pipe containing the CIPP liner, in addition to the presence of the CIPP liner. Additionally, uncured or improperly cured liners can be problematic.

HDPE & Orangeburg Pipes

Cutting or splitting Polyethylene (HDPE) or Pitch/Orangeburg pipes is slightly more complicated by the tendency of the material to bunch up like an accordion during the pipe bursting process. This bunching up can eventually cause so much drag on the process that progress is impaired or stopped short of the desired length. Upsizing can exacerbate the problem, consequently adding more potential to stop short of the desired length. The accordion affect does not typically damage the new product pipe being installed.

Corrugated Metal Pipe

Cutting or splitting Corrugated Metal Pipe (CMP) is extremely challenging and complicated by the tendency of the material to bunch up like an accordion during the pipe bursting process. This material is much more likely to bunch up than Polyethylene (HDPE) or Pitch/Orangeburg pipe. The potential for the accordion affect to eventually cause so much drag on the process that progress is stopped short of the desired length must be anticipated. The remedy for the accordion effect on CMP is to bring more powerful equipment than would have typically been required for a different pipe material as the extra power is intended to extract the wadded-up CMP. Also, limiting pull lengths to less than 60 linear ft (lf) is recommended. Of course, lengths of less than 60 lf limits the number of applications where the method can prudently be used. Since CMP is so likely to bunch up and stop progress, projects where unplanned stoppages are cost-prohibitive or undesirable are best avoided. Discretion over valor in project selection must be used to avoid problems. The accordion affect does not typically damage the new product pipe being installed, however limiting any upsize in highly recommended. A planned excavation pit to remove the CMP pipe being pushed out by expander is essential to complete a pipe burst. If excavation cannot be completed for a stuck burst then alternative trenchless methods should be used to avoid risk.

Ductile Iron Pipe

Cutting or splitting ductile iron pipe is extremely challenging compared to other pipe materials due to the difficulty of displacing or pushing the material away from the new pipe being installed. This material tends to split but remain intact in close enough proximity to hinder installation of the new pipe. Unfortunately, the cutting and splitting process causes the ductile iron pipe to form jagged and serrated edges that could damage the new pipe being pulled into place. Upsizing exacerbates the problem by forcing the proximity of new pipe closer to the jagged and serrated edges. A recommendation from pipe bursting equipment manufacturers is to only replace with same size pipe or 1 upsize. Anything more than that has an exponentially high risk of damage to newly installed pipe. Another potential solution to mitigate the effect of the jagged and serrated edges on the newly installed product pipe is to select a pipe material that is more resistant to abrasion, like Restrained Joint Ductile Iron Pipe (RJDIP) or steel casing. Any new installed plastic pipe could encounter damage throughout the replacement.

11.10 IMPACT OF SOIL AND BEDDING SURROUNDING THE PIPE

Designers, engineers, and owners should consider and acknowledge the following when specifying the pipe bursting process. Subsurface soil conditions and bedding materials surrounding the existing pipe can have an impact on the pipe bursting process. Over the course of pipe bursting various pipe segments, most issues related to subsurface soil conditions can be overcome by adding larger more powerful pipe bursting equipment or changing between pneumatic and static pipe bursting systems as discussed in section 11.5.

The most favorable subsurface soil types for the pipe bursting process are those that are easily compactable by the expander and the force of the equipment. Additionally, the best soils remain compacted long enough for the new pipe to be brought through the temporary cavity prior to returning and causing pipe friction. Conversely, the least desirable soils are difficult to compact and/or do not remain compacted long enough for the new pipe to be brought through the temporary cavity, causing additional pipe friction.

Of course, there are a host of subsurface soil types between the most desirable and the least desirable soil types. The size and power of the equipment is typically adjusted to accommodate the more difficult soil types. Additionally, lubricating systems can be used to reduce pipe friction for those soils that will not remain compacted, when necessary.

High ground water situations complicate the excavations of insertion and machine pits and occasionally require the use of static pipe bursting systems. Pneumatic pipe bursting systems are negatively impacted by extremely wet scenarios. Potentially, excess water combined with vibration from hammer percussion can cause the temporary cavity created by the expander to close quicker than expected causing much more drag on the new pipe being installed. Also, the piston cycle that drives the hammer percussion can permit wet sandy soils to get in front of the expander and obstruct progress of the unit. Additionally, weight and vibration from hammer percussion can cause grade variations when combined with weaker excessively wet soils. Static pipe bursting systems mitigate the impact of these issues and are often preferred over pneumatic pipe bursting systems in high ground water situations, because they have no percussive action and no piston cycle. Thus, there is minimal vibration to cause the temporary cavity to collapse or vibration/hammer combination to cause grade variations in weaker soils. Additionally, static bursting systems lack the piston cycle of a hammer that permits wet sandy soils to get in front of the expander and obstruct progress of the unit. Depending upon the type of soil, the presence of groundwater can either act as additional lubrication or cause additional friction.

Even when the least desirable subsurface soil conditions are encountered, it is rare that the subsurface soil conditions are difficult enough to impact the pipe bursting process beyond a few minor equipment adjustments or additions. Generally, the surrounding soils simply add a level of difficulty or risk to the overall process, especially when upsizing.

Existing pipe that was bed or backfilled with non-compressible material such as those listed below are problematic for the pipe bursting process.

- Concrete
- Flowable fill
- Cement stabilized sand
- Caliche

Existing pipe that was originally laid in trenches where the rock was drilled and shot with explosives to facilitate pipe laying activities is generally not problematic for the pipe bursting process, as the excavation was larger than the new pipe installed. Successful drilling and shooting with explosives, generally, resulted in plenty of trench width to accommodate pipe bursting and even upsizing. However, there are instances where the rock was not adequately removed by blasting and excavation during the original installation of the existing pipe. This may result in pinch points that have the potential to cause unplanned stoppages. Solid rock removed by rock trencher or rock saws would likely be problematic for the pipe bursting process to overcome, as there might not be room to allow for soil displacement. It is critical to understand the trench originally made to lay the pipe as to understand the limitations of upsizing the existing host pipe. Careful considerations should be made when looking to upsize a pipe in a rock trench based on original excavation.

11.11 REPLACEMENT PIPE MATERIAL INSTALLED BY THE PIPE BURSTING PROCESS

As the popularity of the pipe bursting process has grown over the years, so too have the options available to install the replacement pipe. Manufactures have adapted, designed, and developed a host of variations of their products to accommodate the pipe bursting method. Pipe products currently available as options to pipe bursting personnel are broken up into a handful of categories, as follows.

- Continuously (Fused) Replacement Pipe
    - High Density Polyethylene (HDPE)
    - Fusible PVC (FPVC)
- Pull-In-Segmental Replacement Pipe
    - Restrained Joint PVC (RJPVC)
    - Restrained Joint Ductile Iron Pipe (RJDIP)
    - Ductile Iron

Pipe selection is typically based upon several factors including application, required pressures, depth of install, cost, efficiency of installation, and available space, and desired tooling. As discussed in the pipe bursting equipment section, reversible pneumatic pipe bursting allows the process to be completed with only one excavation, which is convenient, efficient, and less costly. Consequently, it is important to note that the pneumatic pipe bursting system is limited to only HDPE pipe. To receive the benefit of the reversible pipe bursting method, HDPE pipe must be specified. Static pipe bursting equipment is required for any of the pipe material, other than HDPE.

Internal Bead Removal

HDPE pipe comes in 40- or 50-ft lengths and is typically butt fused together prior to installation. Proper butt fusion results in an external and internal bead of material at each joint. Occasionally, there has been some discussion of the need to remove the internal bead as there is some potential for this bead to impede flow. The bead can be removed during the butt fusion process by employing a device that can reach and cut the bead located 40 to 50 feet away. This process must be done prior to fusing the next joint. Studies have been done that both refute and support the impacts that fusion beads have on the flows.

HDPE Pipe Stretch

Occasionally, HDPE pipe is stretched during installation by the pipe bursting method. Pipe stretch is experienced most often when the surrounding soil forming the temporary cavity will not stay open long enough to allow the new HDPE pipe to be pulled into place. The resulting pipe friction pulls back on the HDPE pipe while the expander and pipe progress towards completion. In section 11.10 Impact of Soil and Bedding Surrounding the Pipe, the best and least favorable soils for pipe bursting were discussed. Soils that do not compact well and remain open during the process are more likely to cause pipe friction that would result in pipe stretch. Conversely, soils where the temporary cavity stays open longer are unlikely to cause pipe friction that would result in pipe stretch.

If pipe stretch is encountered, eventually, the HDPE pipe will return to the proper final form and be restrained the length of the run as the soil re-compact around the pipe. Thus, pipe stretch when experienced is typically a short-term issue. The issue regarding pipe stretch is how long to wait before making connections and tie-ins. The concern, of course, is that the pipe may move after connections and tie-ins are made, if done too quickly, and cause quality control issues. For this reasons, owners, engineers, and manufacturers have required or recommended long relaxation periods prior to making connections and tie-ins after the pipe bursting process is complete. Unfortunately, the relaxation periods published by manufacturers are based upon the absolute worst-case scenarios with safety factors that are designed to remove liability and are not applicable for all installations in all parts of the country.

In reality, some soil types and some equipment types are prone to stretching the pipe and some are not. Pipe bursting performed with pneumatic pipe bursting systems do not typically experience pipe stretch. Static pipe bursting systems, especially in high groundwater and sandy soils where pipe friction is more significant, are more likely to experience pipe stretch. Consequently, one rule for all installations is not practical. Realistically, the risk of quality control issues related to pipe stretch on most projects is negligible. Experienced pipe bursting personnel can tell when they are experiencing pipe stretch and make the appropriate adjustments, if necessary.

Since experienced pipe bursting professionals know when connections and tie-ins can be safely made, the relaxation periods recommended by manufactures are loosely followed. The inconvenience, disruption, and hazard of leaving holes open longer to accommodate relaxation periods are often forgone for the convenience and public safety of reconnecting services, backfilling pits, and removing temporary bypass systems.

Courtesy of TT Technologies

HDPE Pipe Scratching or Gouging

Another topic of discussion related to HDPE pipe is the acceptable amount of pipe scratching or gouging allowed during the pipe bursting process. Manufactures recommend less than 10% of the pipe wall be scratched or gauged to ensure the integrity of the any HDPE pipe, regardless of how it is installed.

HDPE pipe installed by the pipe bursting process should be no thinner than SDR 19. Because the pipe bursting process involves dragging the HDPE pipe along asphalt and concrete surfaces and eventually through displaced pipe fragments and soils, owners, engineers, and contractors have found that SDR 19 is advisable just to withstand the rigors of the installation process. Again, SDR 19 is the minimum thickness installed for typical gravity installations less than 16' deep. Due to availability and manufacturing preferences, a slightly thicker HDPE pipe of DR 17 is the most often installed pipe for gravity applications by pipe bursting. For pressure applications and deeper depths, thicker HDPE pipe of DR 13.5, 11, and 9 are used.

Rarely, does the typical pipe bursting process result in scratching or gouging the HDPE more than the manufacturers recommended 10% rule. Commonly, the HDPE pipe is fused together up to several blocks from the installation site. Consequently, the HDPE pipe is often drug significant distances from the fusing site to the installation site. While the distance has little effect on the HDPE pipe, the occasional need to pull around corners necessitates pulling against backhoe buckets or truck tires to snake through neighborhoods. This manipulation of the pipe with various equipment is a source of potential risk for gouging. Therefore, the pipe should be inspected at these pinch points. The remedy for pipe gauges or scratches of more than 10% is to cut out or remove the damaged section for the length of pipe and re-fuse the pipe back together less that section.

Existing Pipe Materials That May Damage HDPE Pipe

Also, some existing pipe materials through which the new pipe is being installed by pipe bursting, have a potential for damaging the new HDPE pipe. As discussed in section 11.9, pipe bursting through ductile iron pipe has the potential to damage any pipe material, especially HDPE. Please review section 11.9 for further discussion on the topic.

11.12 FEASIBILITY TESTS FOR PIPE BURSTING

When considering the pipe bursting process among alternatives, there are factors that should be taken into consideration to determine if pipe bursting is even a feasible option for the project in question. Designers, engineers, and owners should consider and acknowledge the following feasibility tests when specifying the pipe bursting process. These considerations should be analyzed on a line segment by line segment basis before any work begins on a segment.

For a small project consisting of a small number of segments, the elimination of a segment due to infeasibility may eliminate the pipe bursting process for the entire project. Conversely, a larger project with several segments under consideration may have one or two segments that do not meet the feasibility test. The infeasibility of one or two segments does not necessitate the elimination of the entire project from consideration of pipe bursting. While most large pipe bursting projects can be done in their entirety, it is not uncommon on large pipe bursting projects to discover a segment or two that are not good candidates for pipe bursting, for whatever reason. The remedy for such situations is to consider alternatives for the one or two exceptions on the project and complete the remainder by pipe bursting, as planned.

Unplanned Stoppages & Excavations

With any pipe segment to be replaced by pipe bursting, there is a potential for unplanned or unexpected stoppage of the pipe bursting equipment. Some of the causes of unplanned and unexpected stoppages are as follows.

- Unknown obstructions
  - Concrete Encasement or Thrust Blocks
  - Previous Repairs with Unknow Fittings or Pipe Materials
  - Dense Roots or Soils
  - Solid Rock Pinch Points
  - Conflict Structures
  - Unknown Utility Conflicts
  - Non-compactable Soils or Bedding
- Equipment Failure
  - Air Hose Issues
  - Hammer Issues
  - Winch Cable Issues
  - Hydraulic Issues

It is important to note that the only remedy for an unplanned or unexpected stoppage is an unplanned excavation. Since there is a potential on any and every pipe burst project for an unplanned stoppage and, therefore, an unplanned excavation, attempting to pipe burst segments where unplanned excavations are unacceptable or cost prohibitive should be avoided. Specifically, pipe bursting under structures, buildings, railroads, and interstate highways where unplanned excavations would be nearly impossible or unacceptable should be avoided. The rule of thumb is to hope for the best, but plan for the worst. Ask the question, "What would happen if we had an unexpected excavation at the absolute worst spot along the segment to be pipe burst." If there is no good answer to that question, avoid pipe bursting the segment.

The previous list of unknown obstructions can sometimes be known obstructions. When known, these obstructions can often be removed by excavation prior to pipe bursting to allow the equipment to pass. In the instances of dense roots and soils, the appropriate power may be all that is needed, so long as the possibility of unplanned excavation at that point is acceptable and not cost prohibitive.

Soil Displacement

Potential utility conflicts are another potential feasibility issue when considering pipe bursting. Since the pipe bursting process displaces the existing pipe and soil into the surrounding soil, it is possible that nearby utilities could be impacted, especially when upsizing the existing pipe. For utilities that cross or run perpendicular to the pipe segment to be burst, the remedy is to simply remove the soil between the two pipes by excavation or potholing. Pipe running parallel to the pipe segment to be pipe burst are more troublesome and likely to render the method infeasible as it would be impractical to excavate or pothole between the two pipes for a long stretch.

Another issue related to pipe and soil displacement is the potential impact on surface improvements such as asphalt and concrete paving. This issue, typically called heave, is especially relevant for shallow pipes that are being upsized. The issue is not typical for pipe deeper than 3 or 4 ft, unless double or triple upsizes are involved. The outcome of heave for surface improvements generally manifests itself in humping/heaving the surface in much the same manner a mole (rodent) humps a yard. The remedy for heave is to drive a heavy vehicle or piece of equipment over the heaving trench to disperse and suppress the heaving force. If this method doesn't work, pipe bursting personnel could pre-saw cut the asphalt or concrete to limit the area of damage to the narrowest width possible. Eventually, the affected asphalt or concrete improvement would have to be replaced for the length and width of the heave. Obviously, some of the cost benefit of the pipe bursting method over traditional open cut would be lost. However, pipe bursting the segment could still result in less surface repair, less backfill, and compaction costs. Also, many of the social benefits could still be realized.

Limitations Regarding Existing Pipe Material

Additional feasibility tests related to existing pipe material was discussed in section 11.9. A list of existing pipe materials that are generally not considered good candidates for pipe bursting was provided.

Also, other consideration should be given to limits on upsizing and cutting and splitting ductile iron pipe and corrugated metal pipe. Review the variables and concepts related to the IPBA's Classification Chart discussed in section 11.4.

11.13 POTENTIAL IMPACT OF PRE-EXISTING POINT REPAIRS, MJ FITTINGS, MJ BENDS, & REPAIR CLAMPS

The pipe bursting process can be affected by pre-existing point repairs, fittings, bends, and repair clamps. Their presence is best mitigated by cutting attachments. Since static pipe bursting systems offer the best options for cutting attachments and cutting rods, static pipe bursting systems are often used to remedy the impact of these potential issues.

Pre-CCTV of the existing line segment prior to pipe bursting is the most effective way to determine the presence of these issues. Knowing if and where these issues are present is the key to mitigating the impact on the pipe burst process. While it is not always possible to perform pre-CCTV, it is recommended wherever possible. It is generally possible to pre-CCTV gravity systems, while pressure systems are rarely televised.

Pre-Existing Point Repairs

The material used to perform pre-existing point repairs is an important determinant in how that point repair would interact with the pipe bursting equipment and what type of equipment and cutters would offer the best solution. Point repairs made with fracturable material do not pose near the challenge of those performed with non-fracturable material. Pneumatic pipe bursting systems will typically navigate through fracturable point repair pipe and fittings, and hang up on non-fracturable pipes and fittings. Static pipe bursting systems with cutter attachments will potentially cut through non-fracturable point repair pipe and fittings.

With any pre-existing point repair there is some potential for an unplanned excavation, no matter what the material or the pipe bursting system used. For the pipe bursting equipment to cut or break the pre-existing pipe and fittings, it is necessary for the point repair to remain in place without shifting to allow cutters and pipe bursting equipment to have the desired effect. If the point repair pipe and fittings shift, in unison, prior to the pipe bursting equipment cutting or breaking the material, it could result in winch cable breaks or jamming of the pipe bursting equipment. Cable breaks or jamming of equipment would require a rescue excavation.

The shifting of the point repair, in unison, is facilitated by the lack of resistance exerted on the backside as it encounters the pipe bursting equipment on the opposite end. Weak soil or weaker pipe material may be the culprit. For example, a point repair performed with ductile iron pipe on a clay or PVC pipe line, would likely shift because the clay or PVC pipe material would not provide enough resistance to hold the stronger ductile iron point repair in place long enough for the cutter to have effect.

MJ Fittings & Repair Clamps

Mechanical Joint (MJ) fittings and repair clamps can only be traversed by cutter rods attached to static pipe bursting equipment. These are often found in pressure pipes at unknown and multiple locations. For this reason, most pipe bursting personnel will choose static pipe bursting equipment with specialized tooling to cut for pressure applications. When MJ fittings and repair clamps are associated with point repairs there is still the potential for unplanned excavations due to shifting.

MJ Bends

Mechanical Joint (MJ) bends are not generally candidates for pipe bursting, even with static pipe bursting systems and specialized tooling. Generally, the static bursting rods would not likely navigate through the bend to reach the insertion pit to permit the process to begin. Consequently, MJ bends are typically excavated and used for machine or insertion pits. The pipe bursting process would still be economical, so long as the spacing between excavations was significant or advantageous.

11.14 LINE & GRADE CONSIDERATIONS

The line and grade produced by the pipe bursting process is a function of time, space, force, friction, distance, & subsurface soil conditions. Designers, engineers, and owners should consider and acknowledge the following when specifying the pipe bursting process. In general, the pipe bursting process produces the line, grade, and invert elevations equivalent to that existing in the host pipe prior to pipe bursting. There is little control that can be exerted over the process by pipe bursting personnel, outside of the excavations. Consequently, owners, engineers, and contractors cannot expect the pipe bursting process to correct previously existing line, grade, and elevation issues. Therefore, it should be assumed that previously existing sags or humps will remain after the pipe bursting process is complete.

Line

Pipe bursting equipment can navigate bend radius of any existing pipe segment where manufactured bends were not installed to make the bend radius during the original installation. In other words, the bend radius was accomplished by deflecting the pipe joints and without the installation of manufactured fittings. For long or severe bend radii, the static bursting system works best, as there is significant play or give in the rods to navigate the radius. Pneumatic systems can navigate smaller radii. The limitation for the pneumatic systems is the likelihood that the cable would saw through the existing pipe wall when tension is applied by the winch. This could cause a deviation in alignment or cause the winch cable to break.

Grade

Pipe segments with sags or grade issues continue to be pipe bursting project candidates. Isolated sags or grade issues can be ignored or corrected depending upon necessity. Isolated sags of 25-50% are often considered incidental and not corrected, while more significant sags would be corrected for gravity applications. For pressure applications, sags and grade issues are generally ignored as they have little impact on the utility of the pipe.

Occasionally, the pipe bursting process will bridge a short sag, thereby reducing or eliminating the amount of sag. While reducing or eliminating short sags is possible, this result cannot be expected with any amount of consistency. Since there is some possibility of reducing or eliminating a sag or grade issue, corrective action is often done after the segment is pipe burst, if necessary. Corrective action, in the form of a point repair, completed prior to pipe bursting could potentially worsen the problem. Please see the preceding section, 11.13 Point Repairs, for further discussion.

While very uncommon, it is possible for the pipe bursting process to result in sags or humps that did not exist prior to pipe bursting the pipe segment. Again, this issued is a function of physics and is typically beyond the control of pipe bursting professionals performing the work.

One potential cause for the creation of a new hump is the presence of solid rock, concrete footers, or other firm subsoil under the existing pipe that when struck by the expander or hammer causes the invert of the new pipe to ride upwards forming a hump. Creating humps that did not previously exist is so rare that they are not typically anticipated. There are few mitigation techniques to remedy the issue beyond excavating and lowering the newly installed pipe after the fact, if necessary. The remedy may include chipping out the solid rock, concrete, or other subsoil to facilitate lowering the pipe.

One potential cause for creating new sags is the presence of poor subsoil under the existing pipe that when passed allows the weight of the expander and hammer to settle, forming a sag for the length of the poor subsoil. The weight and vibration of large pneumatic pipe bursting systems combined with slow progress due to pipe friction are generally the cause of this issue. The relative light weight of static pipe burst systems at the point of contact with the existing pipe mitigates the effects of this risk.

The only opportunities for pipe bursting personnel to control line and grade during or after the pipe bursting process are in the excavations related to accessing the existing pipe with equipment, service reconnections, grade repairs, obstruction removal, or unplanned stoppages

- Retrieval pits
- Machine pits
- Insertion pits
- Service reconnections
- Grade repairs
- Obstruction removals
- Unplanned stoppages

Inside these excavations, pipe bursting personnel are responsible for installing the newly installed pipe at the proper line and grade. These excavations are a critical quality control points that must be monitored for proper installation techniques. Often these excavations include couplings or saddles that are potential sources of grade problems. Proper bedding of the new pipe and fittings is critical for supporting the pipe and the additional weight of the backfill material to be installed.

Additionally, if the pipe bursting equipment does not enter the existing pipe at the proper angle in the insertion pit or pull pit, there is some potential for grade issues just outside the pit. The key is to have the hammer (if applicable) and expander enter the existing pipe with only a slight angle. If the angle is too steep, the hammer and expander could submerge below grade as they enter the existing pipe just outside the excavation. The remedy for this issue is to dig a long enough pit that allows the bend radius of the fused pipe to permit the hammer and expander to lie flat in the excavation as they enter the existing pipe. Digging pits too short will not allow enough length for the hammer and expander to overcome the bend radius of whatever fused pipe material being installed. Static bursting or using segmental pipe material would be less prone to grade issues related to pit length.

Effects on Spring Line of Newly Installed Pipe for Upsizes

While the invert elevations remain consistent with pre-existing invert elevations after the pipe bursting process is complete, the spring line of the newly installed pipe would be at a higher elevation than pre-existing elevation when upsizing. The spring line is defined as an imaginary horizontal line at the mid-point of the circular pipe wall. The spring line is generally considered the lowest point on the pipe wall where service reconnections can be installed. The raising of the spring line on gravity systems can be of significance when existing lateral elevations need to remain at the pre-existing elevations due to flatness of the existing service line. The remedies for these scenarios are laying back far enough on the service lateral to regain appropriate elevations or moving the service connection downstream on the main.

11.15 PRE-INSTALLATION PROCEDURES

Prior to pipe bursting any pipe segment, utility locates should be made and pre-CCTV inspections should be completed, whenever possible. These two measures will be essential in determining likely excavations and risks.

Utility Locates

For most segments to be pipe burst, utility locates simply guide strategy on where to best perform pit excavations and where potholing will be necessary to avoid damaging existing utilities. In most instances, utility locations are a routine part of the process and have little impact on completing the work.

As previously discussed in the Soil Displacement Section, paralleling utilities, near the segment, could be damaged by soil displacement. Therefore, utility locates are essential for locating potential utility conflicts that can easily be addressed or potentially render the pipe bursting method infeasible.

Pre-CCTV

As previously discussed in the Pre-existing Point Repair section, it is not always possible to pre-CCTV all pipe segments prior to pipe bursting, especially in pressure applications. However, when possible, it is best to pre-CCTV inspect the pipe to identify potential issues that could be mitigated if known. Pre-CCTV inspection is used to identify the following potential issues.

- Pre-existing sags or humps
- Verify/identify the existing pipe material
- Identify changes in pipe material
    - o Point Repairs
    - o Changes from one pipe material to another
- Identify MJ fittings, MJ bends, & repair clamps
- Identify cross bored utilities
    - o Gas lines
    - o Water lines
- Identify locations of service connections and manholes

11.16 POST INSTALLATION QUALITY CONTROLS

After the pipe bursting process is complete there are several post installation procedures that can be performed to assure quality. These procedures are industry standards that vary from application to application. These procedures would most likely be performed regardless of the method chosen to rehabilitate or replace an existing pipe.

Gravity Pipes

Gravity pipes, including sanitary sewer and storm sewer, are typically post-televised with CCTV equipment after the pipe has been installed, services reconnected, and manholes installed or rehabilitated. The post-CCTV inspection will verify the quality of the pipe and fitting installation along with grade issues. Additionally, depending upon the region, a low-pressure air test is often required to ensure an air-tight system.

Pressure Pipe

Pressure pipe, including water, sewer, and gas, are rarely post-televised. However, some type of low-pressure air test or hydrostatic test is generally, required to ensure an air-tight system. Additionally, potable water lines require bacteriological testing to ensure safe drinking water for public consumption.

Contingency Allowances or Pay Items for Unplanned Stoppages & Excavations

As previously discussed, there are numerous possibilities for unplanned excavations with any pipe bursting project. While rare, these unplanned excavations are generally due to subsurface conditions that were not previously known or knowable to the owner, engineer, or contractor. When unknown or unknowable subsurface conditions cause unplanned excavations for rescue, grade repair, utility repair, or pipe repair, a cost is incurred by one of the parties to make the appropriate corrective action. The question then becomes which party incurs the damages. Contractors would rely on precedent from contract law that would transfer the risk of unknow subsurface conditions to the owner. Consequently, it is highly recommended that engineer or owner establish unit prices or contingencies allowances in the bid form to allow for these possibilities.

While it would be difficult to provide unit price items in the bid form for every possible unplanned excavation that may or may not be necessary, some potential issues are more common than others and should be considered as a standard pay item for pipe bursting. Some of the more standard pay items would be as follows.

- Point Repairs for Removal of Existing Point Repair or Fitting, If Necessary
- Point Repairs for Grade Adjustment, If Necessary
- Rescue Pit for Removal of Concrete Encasement, If Necessary
- Resolve Utility Conflict, If Necessary

These contingency allowances or pay items would not relieve the contractor of responsibility for quality issues for which the contractor has control. The contractor has control over the pipe installed in the pits.

CHAPTER 12
INTRODUCTION TO SAFETY FOCUSED ON TRENCHLESS TECHNOLOGIES

There are multiple methods involving limited excavating, which are deemed Trenchless Technologies. OSHA has not created any specific standards that reflect any one underground technology. The closest that OSHA comes is CFR1926.800 which relates to tunnels and systems that require employees to enter portals to assist in technologies that are considered "Underground Construction."

OSHA makes specific training recommendations that assist those companies involved with underground construction in identifying exposures and hazards not normally recognized in typical, above ground construction. A good example is illumination and access/egress to those construction sites. Each site listed, that follows this introduction, has specific, OSHA Compliance-related construction standards that are required during operations. There are further mandatory requirements that at times are focused on operations, though not necessarily safety oriented. However, companies and manufacturers of Trenchless Technology Equipment make their technology as safe as possible, oftentimes exceeding OSHA requirements. In many cases, Trenchless Technology has exceeded OSHA CFR Standards and improved safety of employees, specifically regarding the elimination of anyone making entry into a shaft, portal, or tunnel while operations are in progress. The safety recommendations should be reviewed to ensure OSHA Compliance and, most of all, safety of those who practice those Trenchless Technologies.

Contractors should make recommendations of best practices so industry safety recommendations, remain in line with actual field operations.

CHAPTER 4
AUGER BORING

1. OSHA Standards that focus on Auger Boring

1) CFR1926.650 is the primary standard that affects safety regarding the excavation of bore pits and their receiving pits. Different requirements are based on depth. Sloping is common for shallow bore pits 12 to 15 ft or less. Trench boxes and engineered systems are necessary for pits of any depth if the ground is unstable, or depth is greater than 12 to 15 ft.

Additional subparts, throughout the CFR1926 Construction Standards, apply to the actual process of Auger Boring and installation of casing or process pipe. There is a National Emphasis Program on excavations of any type and OSHA is sensitive to access to a bore site, preferring the entire site and installation to be secured with barricades or, in many cases, fence. Engineered systems are also preferred.

CHAPTER 5
PIPE JACKING AND UTILITY TUNNELING

CFR1826.800 is the primary construction standard focused on any trenchless process that includes employees actively involved in an underground construction process.

This standard requires documentation of mandatory training regarding air monitoring, ventilation, illumination, communications, flood control, mechanical equipment, personal protective equipment, explosives, fire protection and emergency procedures, including evacuation plans and check-in/check-out procedures. Excavation of pits/portals typically require engineered systems. As with auger boring, multiple construction standards apply in addition to the requirements

Courtesy of United Rentals

of CFR1926.800. However, the Underground Construction Standard is very detailed regarding the process and application of those additional standards. Air quality/ventilation are good examples, as well as hoisting, unique to underground construction.

CHAPTER 6
MICROTUNNELING

This process is unique and typically engineered throughout the entire process from beginning until ending. Engineered plans will require specific excavation requirements that fall under 1926 CFR 650-653. Installation of shoring, access and egress and installation of the microtunneling equipment fall under multiple CFR 1926 Construction Standards that include:

Subpart C – General Safety and Health Provisions

Subpart D – Occupational Health and Environmental Controls

Courtesy of United Rentals

Subpart E – Personal Protective and Life Saving Equipment

Subpart F – Fire Protection and Prevention

Subpart H – Materials Handling

Subpart I – Tools, Hand and Power

Subpart J – Welding and Cutting

Subpart K – Electrical

Subpart M – Fall Protection

Subpart O – Motor Vehicles, Mechanized Equipment

Subpart P – Excavations

Subpart X – Stairways and Ladders

Subpart AA – Confined Space in Construction

Subpart CC – Cranes and Derricks in Construction

CHAPTER 7
GUIDED BORING METHOD

Excavation of the bore pit will fall under CFR 1926.650-53, Subpart P – Excavations. There are options in Subpart P that allow the Competent Person to use multiple methods to secure the pit area: Sloping and benching, Trench Box, and Engineered System. All systems are dictated by the depth of the bore. Regardless of which system is in use, the Competent Person is responsible for compliance with Subpart P and the OSHA Standard that applies. As with other underground methods, there are similar exposures that require elimination and control. Recognizable exposures are a priority but during the actual boring operations many possibilities will apply to the recommended procedures provided by the manufacturer of the machine being used.

CHAPTER 8
SLURRY BORING

Slurry Bores are typically a method of Trenchless Technology that requires an excavated bore pit as well as a receiving pit. Subpart P – Excavations is the primary standard that applies to this method. As with other Trenchless Technologies, multiple standards, as well as manufacturer recommendations apply to the actual process while performing the actual bore.

CHAPTER 8
SMALL HORIZONTAL DIRECTIONAL DRILLING

Directional Drilling is a method that in most cases does not require a "bore pit." The Boring Machine is set up onsite at ground level and the bore stems are directed by the machine operator who is typically seated above the ground or designated to stand on a synthetic mat to operate the machine. The stems are directed by the operator and guide provided in the first cutting head so that the bore can be tracked from above ground with a specialized receiver that verifies the line and grade. There are multiple exposures to the operator and 99% are addressed by the equipment manufacturer. Locating existing utilities is a priority before beginning the Directional Drill. OSHA Standards that apply are similar to other trenchless operations. Subparts C, D, E, F, G (very important), H, I, J, K, O and P play major roles in operational safety. As with operational safety, precautions for employees are a major priority. Public safety is also a major priority with this method. Fencing off the area where the Directional Drill is operating helps to assist the operator with maintaining isolation while performing his work.

CHAPTER 9
MAXI DIRECTIONAL DRILLING

As with Small Directional Drilling, the major priority for large directional drilling is following the manufacturer's recommendations for operation of the drill. Isolating is a top priority so exposure to the public or any unauthorized individual is eliminated. OSHA Standards apply to the safety of employees and focus on recognizable hazards as operations are set up and the drilling is initiated. Much of the work is performed using additional machines to assist. Loaders, Forklifts, multiple trucks, and equipment manned by qualified personnel is mandatory. Overall, supervision is critical to the safety and success of a large diameter directional drilling operation. As with most highly professional trenchless operations, OSHA Compliance is typically not an issue. The large diameter Directional Drilling crews are small, 4-5 individuals per crew, and they are highly trained in all aspects of safety in their daily operations. There are multiple OSHA CFR 1926 Standards that apply, and most exposures are eliminated by following the manufacturer's recommendations.

CHAPTER 10
PIPE RAMMING

Pipe Ramming is a Trenchless Technology that requires exposing a solid face at the portal which could be vertical or horizontal. Vertical would imply an excavated portal and horizontal would require a sloped face to be excavated to a flat face. In either case, the actual ramming of the pipe or casing, causes a tremendous amount of vibration and noise. As the pipe or casing is rammed into the ground, material is not excavated but enters the casing and or pipe creating a tight tube of material that is removed using pressure provided by a large diameter air compressor attached at the receiving pit. As the air builds volume and pressure, the earthen tube is forced back to the ramming pit. However, on occasion the earthen tube is removed with augers. OSHA Standard Subpart P – Excavations, applies to the excavation, and Subpart G – Signs, Signals and Barricades, is also a priority, as many Ramming Bores are under roads, streets, and railroads. Subparts C, D, E, H, I and O also focus on multiple exposure during pipe ramming operations.

CHAPTER 11
PIPE BURSTING

Pipe Bursting is a method of enlarging an existing pipe installation using a cable attached to a hammerhead cylinder, approximately 10% larger than the pipe to be burst. In the case of pipe bursting, the operation is applied to existing pipe with the intention to enlarge the existing installation. In most cases, there is an existing structure or structures involved in the operation of pipe bursting. Wenches, downhole rollers, and various other types of equipment are installed in that or those existing structures. Entry into existing structures requires OSHA Compliance with CFR 1910.146 - Confined Space Entry requirements along with CFR 1926.650-653 - Excavation Standards, to set up a pit to initiate the process of bursting 8-in. to 36-in. pipe between manholes. As with other Trenchless Technology methods, multiple CFR 1926 Standards apply to exposures during the pipe bursting operations. Subparts C, D, E, F, G, H, I, J, M, O, P, X and Subpart CC could apply based on size and depth.

The NUCA Trenchless Technology Committee would like to thank Olson Construction Law, P.C. and Thomas Olson for his diligent review of this manual, as well as his detailed compilation of this chapter covering the legal rights and obligations in trenchless construction.

CHAPTER 13
THE LEGAL RIGHTS AND OBLIGATIONS

INTRODUCTION

The importance of knowing the legal rights and obligations related to trenchless utility construction cannot be overemphasized. Trenchless construction presents more risks and uncertainties than any other type of construction. Indeed, because the work is *underground*, you can't see it, and the most one can do is infer from other information what conditions may be encountered where the work is performed. Contractors and engineers alike hope that the conditions encountered at any given location will be consistent with the *local geographic conditions, past utility projects in the area, and/or subsurface investigations performed prior to the project.* Notwithstanding quality work from civil and geotechnical engineers, and proper site investigations from contractors, trenchless construction *necessarily* involves a certain amount of *educated guesswork*. As a consequence, the project as *built* is normally, if not commonly, different than *as designed*. This gap almost invariably causes *increases in both time and dollars*, and can often implicate the very *constructability* of the project.

The most effective way that both engineers and contractors can manage this uncertainty is to *perform their respective work as legally required*. In theory, this should simply mean *do what the contract says*. If only it were that easy. The problem in doing this is multifaceted. First, you cannot *do what the contract says* unless you know *what the contract says*. It takes time to learn this, and typically, neither engineers nor contractors have taken this time. Second, even if one does take the time, many of the important *legal rights and obligations are not actually written in the contract*. They are instead *implied at law based on court decisions*. Without some guidance from an attorney, it is difficult to access this information. Third, many times if the contractor or engineer understand what *its rights and obligations are*, they do not understand the other party's *corresponding rights and obligations*. Finally, even when engineers and contractors do know what the contract says, too often one or both of the parties ignore it. The net result of these three problems is that engineers and contractors *lack a common roadmap for designing, bidding, constructing, and managing the change issues that invariably arise*.

The purpose of this chapter is to provide an initial roadmap to help overcome each of these three problems. Set forth below is a *limited summary of important legal rights and obligations*. This includes those *expressly written* into contracts as well as those *impliedly made a part of contracts by law*. This chapter is admittedly not intended to be more than a *limited summary*. This means that there are rights and obligations not addressed herein, and, for those addressed, more analysis will be required to adequately manage specific project issues. Furthermore, rights and obligations admittedly vary from state to state, as well as on a federal level. To fully understand the rights and obligations in the states where you work, be it generally speaking or in the context of a specific project issue, you will need to consult with an attorney.

A. ENGINEER'S DUTIES[1]

1. The Duty to Conduct an Adequate Site Investigation

The engineer's job is to design the project. Since trenchless construction is, by definition, *underground*, a good design *must account for the underground soil conditions*. It is therefore necessary for the engineer to *perform an adequate subsurface investigation* during the design phase of any trenchless construction project. Unfortunately, there seems to be a growing trend for owners and engineers to believe that it is better to do a more limited subsurface investigation. This belief is based on the notion that the project will ultimately be cheaper with less information provided. In practice, there are a variety of reasons why this belief is mistaken.

a. Engineers are subject to professional standards to conduct an adequate subsurface investigation.

Engineers are subject to *professional standards on the scope of their site investigation for project design*. The U.S. Department of Transportation prepared a report to "provide guidance to design and construction engineers involved with the geotechnical aspects" of project work.[2] The report "contains information on subsurface information acquisition" [and] "disclosure of subsurface information in contract documents."[3] The report received "extensive review" from "the FHWA, the States, and industry" as well as "regional engineers."[4] The report provided the following "guidance to design and construction engineers":

- "Early recognition of geotechnical problems during the design state is still the best way to reduce the risk of geotechnical construction problems."[5]
- The task of performing a subsurface investigation is normally the owner's responsibility "to avoid the latent costs for pre-bid subsurface investigations by bidders."[6]
- "The resulting design implies, and the subsurface data describes, the conditions on which bidding and construction will be based. The representation of these results also provides the basis for application of the DSC [Differing Site Conditions] clause."[7]
- "Type I DSC claims usually occur when the agency [owner] does not conduct an adequate subsurface investigation and prepares plans based on assumptions as to the nature of the subsurface condition."[8]
- "An adequate site investigation is needed to minimize the potential for construction problems, change orders, and claims."[9]
- "Accepted standard procedures from ASTM, AASHTO, or as established by the agency should be followed in the investigation process."[10]
- "The agency spends months in project development to collect information about subsurface conditions at the project site. The agency's engineers assess the reliability and representativeness of the available data in project design. The contractor, on the other hand, has a limited time during bidding in which to assimilate all the available data and develop his interpretation."[11]
- "The inclusion of geotechnical information in the contract provides both the agency and contractor a consistent geotechnical baseline for determination of what constitutes a differing site condition."[12]

B. HOW TO CONDUCT AN ADEQUATE SUBSURFACE INVESTIGATION

(i.) Collection of Pre-Existing Information and Documents about Geological Conditions

The first step for any subsurface investigation should be *collection of pre-existing information and documents about geological conditions*.[13] "The initial phase of a geologic and site reconnaissance investigation is to collect existing geologic background data through coordination and cooperation."[14] "Available technical data . . . from personal communication should be reviewed before any field program is started."[15] To be clear, the engineer's investigation "should encompass" a "[r]eview of available information, both regional and local, on the geologic history, rock, soil and groundwater conditions *occurring* at the proposed location and *in the immediate vicinity of the site*."[16]

The best source to obtain pre-existing subsurface information is often *local agencies*.[17] AASHTO long ago established and set forth relevant guidelines for *engineer's subsurface investigations*.[18] "[I]ndividuals with local knowledge" of the geotechnical conditions should always be consulted.[19] Subsurface investigations should normally be "based on the result of previous work."[20] The project owner may have "considerable information" of the local conditions based on project work at "an adjacent site."[21] Given that, the engineer should meet with "[m]unicipal engineering and water service offices" specifically.[22] Since these offices oversee design of local street, sewer, and water projects, they can provide relevant information about *other projects* because they frequently maintain subsurface and groundwater information *derived from them*.

(ii.) Site Testing

In addition to pre-existing information, "[a]n adequate site investigation should include sufficient amounts of boring, sampling and testing to identify potential sources of construction problems which were identified during terrain reconnaissance or site inspection."[23] The subsurface testing, which most commonly consists of *borings*, provides the necessary design and construction information on "the type of materials to be encountered and their in situ condition."[24] Borings should be of sufficient quantity and location to provide a soil profile for the pipe zone, the area typically one to two diameters above and below the proposed depth of the alignment.

C. CONSEQUENCES FOR FAILURE TO CONDUCT AN ADEQUATE INVESTIGATION

As noted earlier, "[a]n adequate site investigation is needed to minimize the potential for construction problems, change orders and claims."[25] Without that, the project design will lack the requisite foundation to ensure the project can be constructed as bid. For the owner, that means paying more to the contractor. The engineer may itself be liable for *failing to conduct an adequate site investigation*.[26] One of the largest and most common type of extra costs that *would have been avoided with an adequate investigation at the design phase is delay*. Subsurface investigation at this stage normally means that equipment sits idle waiting for an accurate determination of the conditions. This delay then continues while a determination is made, be it by the engineer and/or the contractor, on whether the equipment on site can complete the bore as-designed and as-bid.

2. Duty to Disclose the Subsurface Investigation Information and Documentation

When an engineer undertakes a sufficient subsurface investigation, it can properly design the project. For the contractor to properly bid the project, the engineer needs to share the subsurface investigation information and documentation with the contractors during the bidding phase. This makes common sense: the more a contractor knows about the conditions in which it will perform its work, the more accurately it can determine how to perform this work and at what cost.

Notwithstanding this, it is common throughout the country that owners and engineers do *not share in whole or in part what they have learned from the site investigation*. When the FHWA surveyed state highway agencies on this issue, many disclosed that they do not share all of their geotechnical information and documentation with bidders.[27] The rationale behind not sharing this with contractors is the same as the engineer not getting it in the first place: the less the owner says, the less liability it has. For the reasons discussed above, not getting the information and/or not sharing it with the contractor will normally *increase the project cost*, which both the owner and engineer may be obligated to pay.

But beyond cost, there is also an important *legal issue*. In most states, it has long been held that an owner has an *implied duty to disclose information in its actual or constructive possession to contractors during the bidding phase that is material to the contractor's performance*.[28] Indeed, the federal government has stated that "[a]ll pertinent subsurface information should be disclosed in the contract documents."[29] Since this duty to disclose is implied, you will not find it written in any contract. But that does not change the fact that it has the same legal force and effect as if it was *expressly written into the contract*.

This obligation should normally require the owner and engineer to disclose *all* information and documentation gained from the project's subsurface investigation. An engineer cannot satisfy this obligation unless it ensures that the geotechnical firm provides all of the testing information. While "there is no single procedure for recording the data, the record should reflect all details of the investigation."[30] "[B]ecause the . . . boring logs provide fundamental facts on which all subsequent conclusions are based, the necessity of recording the maximum amount of accurate information cannot be over-emphasized."[31] This may include soil conditions "perceived" as well as "observed" in testing.

> The subsurface observed in the soil samples and drill cuttings or *perceived* through the performance of the drill rig (for example, rig chatter in gravel, or sampler rebounding on a cobble during driving) should be described in the wide central column on the log labeled 'Material Description,' or in the remarks column if available.[32]

While the engineer it not likely to have performed the testing itself, given that this testing will affect the project design and the related cost, failure to ensure that the geotechnical firm provides this information will impact both the owner and the engineer. This has long been the law of the land. Indeed, as explained in *Christie v. U.S., 237 U.S. 234, 240 (1915):*

> [W]hen obstructions were met, the [drilling] apparatus was moved elsewhere until a place was found where the drill would penetrate, and the result was recorded as if taken at the place staked out. . .. [T]he specifications contained only the record of the completed borings and did not show any record of sunken logs, or of cemented sand and gravel, or conglomerate impenetrable by the drill [encountered at the staked locations].[33]

This disclosure obligation should also include *information obtained from other projects.* The United States Department of Transportation has detailed how to communicate this in the contract documents:

> Use specific plan notes to communicate experience with the type of subsurface condition at a specific project site to all prospective bidders. An example follows:
>
> > Although boulders in large quantities were not encountered on this site in borings, which are numbered BAF-1 through BAF-4, previous projects in this area have found large quantities of boulders. Therefore, the contractor should be expected to encounter substantial boulders quantities and excavations.
> >
> > The contractor should include any perceived extra costs . . . in this area in the bid price.[34]

If it is discovered later by the contractor that the owner and engineer failed to properly disclose what they *knew or should have known*, the owner *should be*, and the engineer *may* be, liable for all of the related extra costs in time and money.[35]

3. Duty to Identify Potential Utility Conflicts and Prepare Utility Relocation Schedules

Another issue related to the duty to conduct an adequate subsurface investigation is the engineer's duty to identify potential utility conflicts and prepare utility relocation schedules. Untimely utility relocation is all too common on construction projects and has been identified by the Federal Highway Administration ("FHWA") as the "leading cause of highway project construction cost-and-time overruns."[36]

To evaluate utility problems, the FHWA assessed what is the standard operating procedure for DOTs in "all 50 States, the District of Columbia, and the Commonwealth of Puerto Rico."[37] The FHWA determined that the primary reason for Utility Problems was DOTs failing to properly investigate the existence and location of utilities:

> Most State DOTs do not adequately investigate underground utilities, especially vertical or depth (z coordinates), resulting in utility conflicts either being misidentified or not identified at all during the preconstruction phase. This results in contractors unexpectedly encountering utilities during construction, a situation that often increases project cost or causes delay, or sometimes both.[38]

To address this problem, the FHWA described two key obligations that DOTs are required to satisfy on *federally-assisted programs.* First, public owners must obtain accurate and complete utility information through subsurface utility engineering ("SUE").[39] This enables owners to obtain accurate horizontal information (x and the y) as well as vertical information (the z) which utilities are unable to provide.

Second, owners should execute agreements with the utility companies for the timing of utility relocation.[40] The agreement should describe both *what* relocation work the utility must perform and *when* they are required to perform it.[41]

On projects that are not funded through federal assistance programs, it is important for the engineer to review the applicable statutes, regulations, and ordinances, as well as the language of the contract. These documents may include similar obligations that require the owner to pay for utility-related delays and/or provide the owner with the right to require utility relocation within a stated period of time after notification. Moreover, there may be applicable state utility accommodation manuals, DOT memoranda, and/or franchise/easement agreements that provide additional rights and obligations related to utility relocation. Given the severe impacts to projects with inadequate utility investigations, even in the event there are no statutes, regulations, or ordinances, engineers should still look to perform as detailed an investigation as possible.

4. Duty to Clearly Communicate the Contract Requirements

On private construction projects, there is an opportunity for contractors to *negotiate contract terms and conditions*. Contractors can therefore ensure that the contract language *clearly sets forth what they are required to do*. By sharp contrast, on public construction projects, contracts are offered on a *take it or leave it basis*. Contractors have no opportunity for input on how the contract requirements are written. As a consequence, courts have established some general rules to protect contractors if the engineer fails to clearly set forth the contract requirements.

An engineer fails to clearly communicate contract requirements when the language is *ambiguous*. A contract is ambiguous when a word, phrase, or provisions has, or is susceptible of, at least two reasonable but conflicting interpretations or meanings. It is important for engineers to understand that, as a general rule, "technical terms and words of art are given their technical meaning when used in a transaction within their technical field."[42] This means that for an engineer to accurately communicate trenchless construction requirements, it needs to study the trenchless industry. If the engineer fails to do this, the language used may be ambiguous, and the ambiguous contract language will be interpreted in favor of the non-drafting party.[43] The contractor's interpretation will therefore prevail notwithstanding that it may be contrary to what the engineer intended. In short, the only "intent" that is enforced is that which is clearly written in the contract.

It is equally important for engineers to ensure that the geotechnical firm has communicated its findings clearly. Unless the geotechnical firm clearly defines what it means by the terms it uses to describe what was encountered in the soil borings, the contractor may rightfully misunderstand what was intended. "Clear and accurate data are required to describe the soil profile and sample locations."[44] Terms used should only be those from standard soil classification systems commonly used such as ASTM and AASHTO,[45] or otherwise defined in an attached glossary of terms. While terms such as "cobble" or "boulder" are commonly known and used, there may be some *size* variance depending on the source used. By contrast, a term such as "stone" is not normally used to define a material type. As such, it would not be appropriate to use this term, certainly not without a size definition. In addition, since the N-values set forth in the boring logs help the contractor determine the requisite jacking forces, it is important that the borings clearly communicate whether the N-values are "corrected" for hammer efficiency and overburden pressure, or not.

5. Duty to Properly Design

In most states, an owner *impliedly warrants* that the *methods and materials specified by the plans and specifications will produce an acceptable result if the contractor follows them*. "In general, then, design specifications created by the Government contain an implied warranty that if the contractor adheres to the specifications, the result will be acceptable to the Government."[46]

Whether this implied warranty of design applies turns on whether, and to what extent, the contractor has *discretion* on how to complete the work.

> Design specifications "set forth in precise detail the materials to be employed and the manner in which the work [is] to be performed," from which the contractor is "not privileged to deviate . . . but [is] required to follow… as one would a road map.[47]

Assuming the warranty applies, if the work cannot be performed because of a design deficiency, or can only be performed at a greater cost, the owner is normally liable to pay for the impact in time and dollars.

In addition, whether the implied warranty applies or not, contractors should *not* be responsible to determine whether the project can be constructed as designed.

> The Government is responsible for the correctness, adequacy and feasibility of the Specifications and the other party [contractor] has no obligation to check and verify the work product of the party who assumes responsibility for the preparations of the specifications…It is inconsistent for the Government to insist that there is nothing wrong with the specifications and, on the other hand, contend that the contractor should have found out what was wrong with it."[48]

The above rule means that contractors should not refuse to bid a project if they have questions about the design adequacy. It also means that contractors need not include a contingency in their bids.

Engineers should also recognize that they too may have financial responsibility for the consequences of an inadequate subsurface investigation and related design. Owners have the right to seek reimbursement from the engineer, which normally results in the engineer having to file a claim under its Errors and Omissions Insurance. The owner has every right to do this, particularly where the cost of the project is higher than it would have been but for the engineer's failure to properly perform its investigation and design work. As noted above, a typical example of such cost is *delay*. In addition, there is a growing legal trend, which has been established by court decisions, whereby engineers may be *independently liable to the contractor for negligence as well as the owner for breach of contract*.[49] This relates to improper design, as well as the failure to *obtain and communicate information an engineer knows will be relied upon*.[50] Engineers should also recognize that there can be very real advantages for a contractor to seek reimbursement directly from the engineer.[51]

The engineer's protection against a defective design is not malpractice insurance: it is a greater understanding of both soils and the trenchless technology. My experience of over 30 years is that engineers all too often lack sufficient geotechnical training. I have heard engineers say they did not study geotechnical engineering in college because it is based more on art than science.

> In test boring, the sensitivity required in knowing when to take the 'sample,' and the interpretation of the picture it presents, makes this an art rather than a science. Adequate and reliable interpretation of the exact nature of soil samples is often a matter of judgment.[52]

As a consequence, when an engineer designs an underground project, it often lacks the requisite geotechnical understanding. To overcome this, other than taking the time to get formal training in geotechnical engineering, engineers need to *more fully utilize the services of a geotechnical firm*. The extra cost of such services pales by comparison to a project that is under-designed.

Engineers can better appreciate the need for accurate soils information by learning more about trenchless technology. Soil boring logs are "considered the most reliable reflection of subsurface conditions."[53] That is why test borings are the "primary source" of subsurface information.[54] It is also why "[p]articular protection is given by the courts to the right of bidders to rely upon drill hole data in the contract data, recognized to be the most reliable and the 'most specific indicator' of subsurface conditions."[55] Borings provide the trenchless contractor the basis to calculate the *required jacking forces* to complete the bore. This begins with the soil's *density*. "Density" is defined by the *N-values* obtained from the *standard penetration test*. "N-value" is defined as the "number of blows per foot required to drive a standard penetration test (SPT) soil sampler into the ground during geotechnical exploration."[56] *Blows per foot* (BPF) measures soil density to arrive at the N-value. With this information, the contractor can then quantify the required *jacking forces*, defined to mean the "total force required to overcome the *face pressure* [on the machine] and the *frictional resistance component along the pipe* to allow forward movement."[57] What trenchless methodology to use also turns on the soil *type* as well as the presence of *obstacles* such as *boulders*. In addition, the length of the bore, and the pipe size and thickness are also relevant. A pipe-ramming project provides but one example to understand this this.

> The recommended framework for the evaluation of rammed-pipe drivability commences with the selection of the preferred pipe geometry, including the alignment, length, diameter, and wall thickness of the pipe and the design soil profile.[58]

If the soils accurately encountered are not actually shown in the pre-bid test results, there is a good chance the technology selection will be wrong. Similarly, if the specified pipe is under-designed (e.g., thickness not enough to bear required jacking forces, size too small to allow removal of boulders through pipe), it is likely that the chosen technology will be inadequate. Fortunately, there is a lot of literature available for engineers to consult.[59]

6. Duty to Cooperate

As odd and unfair as it may seem, many of the important contractual duties for engineers, like contractors, are not actually written in the contract. In addition to the implied duties discussed above, the engineer also has an *implied duty to cooperate*. "The contracting party impliedly obligates himself to cooperate in the performance of his contract and the law will not permit him to take advantage of an obstacle to performance which he has created or which lies within his power to remove."[60]

This implied duty has fostered several subsidiary duties that vary throughout the country. These subsidiary duties include:

- Implied duty to provide adequate and timely site access;[61]
- Implied duty to make timely decisions;[62] and
- Implied duty to reasonably inspect.[63]

B. CONTRACTOR'S DUTIES AND RIGHTS

1. Duty to Investigate the Site

Generally speaking, bidders have a contractual duty to "visit the site" and have "thoroughly informed themselves of all conditions and factors which would affect the work and the cost thereof." This *requires* the contractor to *visually* inspect the site, local geographic conditions, and all site information provided as part of the contract documents (whether treated as part of the "contract documents" or not).[64] This duty also includes reviewing site information that the contract documents state are "available upon request." And to be safe, contractors should also get in the habit of asking *in writing* for all site data that the owner has. Whether legally obligated to do this or not, this request avoids a situation where the engineer or owner says: "All you had to do was ask for it, and we would have given it to you." As I said, make sure this is done *in writing* so there is a clear record. If the owner or engineer provides documentation that it neither included nor referenced in the contract documents, and this information makes the project more expensive, make sure you follow up your initial production request with one asking that all bidders be told of this *per addendum*.

Generally speaking, the contractor's site investigation duty *does not require* the contractor to make its own independent test borings, laboratory tests or detailed subsurface investigation.[65] The policy behind this is simple: as noted earlier, a contractor is not required "prior to submitting a bid and entering into the contract, to conduct its own investigation in order to ascertain the truth or falsity of the defendant's positive assertions regarding subsurface conditions encountered in drilling."[66] This is also true "to avoid the latent costs for pre-bid subsurface investigations by bidders."[67]

There are also other limitations that may apply to the contractor's site investigation duties. First, the contractual duty to *examine the site* may not impose the "duty of making a diligent inquiry into the history of the locality."[68] Second, the contractor may not be required to contact *other contractors in the area*.[69] Third, the contractor may not be required to "have checked back with other officials of the Owner concerning the information given it by the [owner's representative]."[70] Fourth, even if the contractor had *general knowledge of the area's site conditions* that may not defeat its justifiable reliance on the site-specific soil borings because "no one can know with certainty what will be found during subsurface operations."[71] Finally, a contractor's general knowledge of an area's site conditions does not mean that the contractor had reason to know the soil conditions of a particular site, inasmuch as local soil conditions in the area may vary, and inasmuch as the owner made a distinctive and definite representation as to conditions to be encountered.[72]

Thus, while the owner may provide the contractor with the *right* to conduct subsurface testing, a contractor normally does not have the *obligation* to do so. If the contract does afford contractors the right to conduct subsurface testing, they should carefully consider whether to exercise this right. If the contractor does conduct its own subsurface testing, it may find that the soil conditions are not as bad as shown in the owner's soils data, and therefore be able to submit a lower bid. By contrast, if the contractor finds that the soils are worse, it faces the dilemma of either increasing its bid (in which case it will probably not be awarded the project) or ignoring the information. The problem with the latter approach is that if it does get the project, and the soil conditions prove to be different than shown in the owner's soil data, the contractor may not be able to recover for the related extra costs. The engineer would have the right to respond that the contractor *could not and should not have relied upon the owner's data*.

2. Duty to Seek Clarification

When bidding a project, if the contractor discovers contract language that is either in conflict or obviously unclear, it should *seek and obtain written clarification before submitting a bid*. This duty is often *expressly set forth in standard contracts*,[73] though the duty is otherwise impliedly required.[74] If it fails to do so, and a dispute develops later, the contractor is legally required to meet whichever contract interpretation provides the most benefit to the owner at the cheapest cost. An easy example is where the soils report states that cobbles and boulders were encountered during soil borings, yet the boring logs do not show either. The language is in conflict, for which the contractor should seek written clarification. If the contractor fails to do so, and the contractor encounters cobbles and/or boulders, the contractor may not be entitled to any additional compensation. The same may also be true if the engineer uses different language in different locations of the specifications to describe the required work. If the contractor fails to seek a written clarification, it is no defense for the contractor to claim that this would result in the owner unfairly receiving a financial windfall (i.e., the contractor having bid it based on the cheaper cost interpretation).

To ensure that there is no misunderstanding later, the contractor should always seek clarification in writing. Most contracts then require the engineer to issue an addendum to clarify the issue. That way, all contractors are bidding on the same basis when pricing the work. If the engineer fails to correct the flaw and notify all bidders, then the contractor should not be held responsible for the contract interpretation that most benefits the owner.[75]

3. Right to Rely on Subsurface Data

We can all agree that the only reason owners provide subsurface data is so contractors *will rely upon it when bidding the project*.[76] Notwithstanding this fact, engineers are increasingly attempting to include contract language that purports to limit or disclaim the contractor's right to rely. Examples include:

- The soils data is "for information only."

- The actual site conditions "may vary."
- The owner "does not warrant or guarantee the accuracy" of the soils data.
- The "contractor is responsible for conclusions to be drawn" from the soils data.

There is no question *why* engineers are doing this: they want the owner to avoid liability if the subsurface conditions *actually encountered* are different than *indicated by the pre-bid soils data*. From an equitable standpoint, this is unfair. If the owner got the benefit of a lower price based on the conditions shown in the pre-bid data, it should have to pay if the actual conditions prove to be different and more costly. As a consequence, some courts have expressly recognized that such disclaimers are *unenforceable*. As explained in *Appeal of Herman H. Neumann*, 69-2 BCA ¶ 7965, ASBCA No. 13074, 1969 WL 959 (1969):

> We agree that where the Government makes a positive assertion as to subsurface conditions, it is not relieved of liability by general contractual provisions requiring the bidder to investigate the site or satisfy itself of the conditions, or stating that the Government does not guarantee the accuracy of the information furnished.

Likewise, in *Appeal of Bay West, Inc.*, ASBCA No. 54166 (2007), the court explained:

> It has long been the rule that contract borings are the most significant indicator of subsurface conditions.... This is so even though the contract advised bidders they were responsible for making their own determination of the characteristics of the native soils and contained disclaimers in the Physical Data clause.

In addition, in *Metcalf Const. Co., Inc. v. U.S.*, 742 F.3d 984 (Fed. Cir. 2014), the court stated:

> The information was given Appellant by the Government. It had a purpose - to aid the Appellant in formulating its offer. Appellant had a right to rely on such information. If the Government did not want that information to be used and relied upon then it should not have taken the borings, prepared boring logs and given them to the contractor for use as an aid in preparing its offer. The Government may not by means of a broad disclaimer leave without remedy an otherwise valid contractor grievance under the Differing Site Conditions clause.

One of the additional reasons cited by courts for not enforcing disclaimers is that it would be *inequitable* to allow engineers to rely upon soil borings to *design* the project but not for contractors in *bidding* the work.[77]

Admittedly, the law on this issue is not settled throughout the country. Many courts have not addressed the enforceability of site data disclaimers. Other courts have, but their opinions vary. Contractors should therefore seek outside legal guidance on how courts have addressed the issue where they bid work. Engineers should also carefully consider whether to try to limit the contractor's right to rely on subsurface data. Doing so will likely cause projects to be *more expensive, not cheaper*.

> Reliance is affirmatively desired by the Government, for if bidders feel they cannot rely, they will revert to the practice of increasing their bids. The purpose of the changed [differing site] conditions clause is thus to take at least some of the gamble on subsurface conditions out of bidding. Bidders need not weigh the cost and ease of making their own borings against the risk of encountering an adverse subsurface, and they need not consider how large a contingency should be added to the bid to cover the risk. They will have no windfalls and no disasters. The Government benefits from more accurate bidding, without inflation for risks which may not eventuate. It pays for difficult subsurface work only when it is encountered and was not indicated in the logs.[78]

4. Duty to Continue Work

When a contractor encounters differing site conditions or other changes to the scope of its work, it is important to note that the contract will almost certainly require the contractor to continue working as directed by the owner/engineer (notwithstanding a disagreement regarding who is responsible for the related cost and time impacts). Typical contract language provides: "Contractor agrees to continue performance of the work and shall proceed in accordance with the directives of the Engineer, under protest, in the event of a dispute or controversy."

As a consequence, although the contractor may want to stop work or refuse to work until an agreement is reached, the contractor risks termination and potential claims for the cost to have another contractor perform the work and/or resulting project delays. When this happens, there are can be many consequences. First, you will almost surely be sued by the owner for increased project costs. This is a result of the takeover contractor nearly always costing more. Takeover contractors are being brought in during the middle of the project, and often under tight deadlines, which increases the cost to the work. Second, even if you were justified in your request for additional compensation, you will still have a termination on your record which will impact your bonding and ability to win bids on future projects. Finally, if you do not wish to pay for the increased costs due to the denied request for extra compensation, you will be spending money on attorney's fees to defend your position. It is therefore in the contractor's best interest to proceed with the changed work after ensuring it has properly preserved its right to additional time and/or compensation under the contract.

5. Duty to Satisfy Procedural Requirements to Obtain Additional Compensation and/or a Time Extension

Trenchless contractors know that it is more likely than not that the conditions and/or the related work will be different than anticipated and bid. The difference may be above ground (i.e., lack of timely and complete site access) as well as below grade. Contractors also know that this difference normally increases the cost and time required to complete the work, and sometimes the ability to even complete the work as designed. Given how tight margins are, contractors cannot afford to bear the related financial impact.

When the cost and time required to complete trenchless work changes for reasons outside of the contractor's control, there is typically a contractual basis to obtain a contract adjustment for the related impact. What is required in virtually all public design documents is that the contractor provide *immediate written notice before it proceeds with work in the affected area.*[79] The purpose for this requirement is so the engineer has the opportunity to timely evaluate whether the project work has changed and, if so, whether and to what extent this change may affect the cost and time in completing it, as well as in some circumstances, the very feasibility of completing the project as designed. The engineer also then has the opportunity to determine whether the project can and should be re-designed in response. The importance of the contractor providing the required notice can therefore not be disputed.

Unfortunately, contractors too often fail to provide the notice as required. Either no written notice is provided until the work has continued, or even until after the work is completed, or if notice is timely provided, it is given verbally. The problem in not providing the notice exactly as required is that it puts at risk the contractor's legal right to obtain additional compensation and/or a time extension. Although court decisions vary throughout the country, many courts have held that a contractor waives it right to obtain a contract adjustment if it fails to provide the required notice. In *Joel Lehmkuhl Excavating v. City of Troy*, 2005 WL 994607 (Ohio Ct. App. 2d Dist. 2005), for example, the court stated: "[W]here a construction contract specifically provides that any claims for extra work must be made in writing prior to executing the work, the clause is valid and binding upon the parties, and no recovery can be had for extra work without a written directive, unless the written requirement is waived by the City."

In addition to the initial written notice requirements, there are also other procedural requirements. Examples include additional notice "if the contractor seeks compensation," obtaining a change order before proceeding, filing a formal claim if the engineer disputes the contractor's position, providing supporting documentation for the amount of additional compensation/project delay, updating project schedules, appealing the engineer's claim denial, requesting mediation, demanding arbitration, filing a bond claim, filing a mechanic's lien, etc. Failure to meet any of these or other procedural requirements can result in the loss of all rights to a contract adjustment.[80]

In short, it is not enough that the project conditions and/or work may have changed: contractors must satisfy all related procedural requirements or risk not obtaining an adjustment in price and/or time. And, even though a lawyer may be able to help you overcome some failures to meet procedural requirements, it is both more expensive and detrimental to project working relationships.

To ensure that contractors do what is necessary to obtain contract adjustments, they must first learn what is *expressly set forth in the contract.* Contractors need to know not only what these clauses are, but *how they can and should be applied.* Given that the clauses are meant to apply to many factual situations, it is often unclear how they should apply to a *particular situation.* Contractors also need to know the additional legal rights that are not actually written in the contract, but apply with the same force and effect. These arise from the *implied duties* of the owner and engineer discussed earlier. If the owner and/or engineer fail to satisfy implied duties, contractors normally have corresponding rights. To make this easier, contractors should consider obtaining legal assistance to prepare a *flowchart addressing how and when each of the rights apply, as well as the related procedural requirements that must be met.* The goal should always be to do what the contract says. To be sure, a contractor cannot do what the contract says unless it knows what the contract says.

CONCLUSION

Public construction is an inherently risky profession due to the nature of public bidding. Trenchless construction is even more so due to the simple fact that it is simply not possible to guarantee what conditions will be encountered until they are uncovered. This risk leads to disputes regarding extra compensation, extra time, or any number of additional reasons. If contractors and engineers more fully understand their corresponding *rights and obligations* set forth in both the contract and in common law of whatever jurisdiction they are in, disputes would be a much less common proposition.

REFERENCES

1. *With limited exceptions, the duties discussed in this section are those of the project owner. However, given that the engineer is generally treated as the owner's representative under most, if not all, public construction contracts, the duties are presented as those of the engineer. See, e.g., Section 9.01.A of the EJCDC C-700 Standard General Conditions of the Construction Contract (2007). If the engineer fails to properly perform its duties on behalf of the owner, the owner is deemed to have breached those duties, and the contractor should be entitled to additional time and compensation. Increasingly, as discussed below, the engineer may also be liable to pay the owner and/or the contractor.*
2. *See May 2, 1996 memorandum from the U.S. Department of Transportation to Regional Administrators re: "Transmittal of Geotechnical Engineering Notebook Issuance GT-15 Geotechnical Differing Site Conditions."*
3. *Id.*
4. *Id.*
5. *See U.S. Department of Transportation, GEOTECHNICAL ENGINEERING NOTEBOOK GEOTECHNICAL GUIDELINE NO. 15, "Geotechnical Differing Site Conditions" (1996) at 4.*
6. *Id. at 7.*
7. *Id.*
8. *Id.*
9. *Id. at 10.*
10. *Id.*
11. *Id. at 13.*
12. *Id.*
13. *See ASTM Standard D420-98 (2003), "Standard Guide to Site Characterization for Engineering Design and Construction Purposes" at 3.*
14. *See U.S. Corps of Engineers Engineering and Design, ENGINEER MANUAL "GEOTECHNICAL INVESTIGATIONS" EM 110-1-1804 (2001) at 3-1.*
15. *See ASTM Standard D420-98 at 4.1.*
16. *Id. at ¶ 5 (2009) (emphasis added).*
17. *See U.S. Corps of Engineers Engineering and Design, ENGINEER MANUAL "GEOTECHNICAL INVESTIGATIONS" EM 110-1-1804 (2001) at 3-1. ("[C]ontacts should be made with . . . local agencies to identify available sources of existing geologic information applicable to the project.").*
18. *See AASHTO MANUAL ON SUBSURFACE INVESTIGATIONS (1988).*
19. *Id. at 21.*
20. *Id. at 19.*
21. *Id. at 28.*
22. *See U.S. Corps of Engineers Engineering and Design, ENGINEER MANUAL "GEOTECHNICAL INVESTIGATIONS" EM 110-1-1804 (2001) at 3-2.*
23. *See U.S. Department of Transportation, GEOTECHNICAL ENGINEERING NOTEBOOK GEOTECHNICAL GUIDELINE NO. 15, "Geotechnical Differing Site Conditions" (1996) at 10.*
24. *See U.S. Army Corps of Engineers Pub. No. EM 1110-101804, GEOTECHNICAL INVESTIGATIONS MANUAL (Jan. 2001) at 5-7.*
25. *See U.S. Department of Transportation, GEOTECHNICAL ENGINEERING NOTEBOOK GEOTECHNICAL GUIDELINE NO. 15, "Geotechnical Differing Site Conditions" (1996) at 10*
26. *See, e.g., Zontelli & Sons, Inc. v. City Nashwauk, 373 N.W.2d 744, 753 (Minn. 1985).*
27. *Id. at Appendix B, "FHWA Survey of Geotechnical Information Included in Bid Documents by Highway Agencies" (December 1994).*
28. *See, e.g., Appeals of Pitt-des Moines, Inc., 96-1 BCA ¶ 27941, 1995 WL 575160 (ASBCA 1995).*
29. *See U.S. Department of Transportation, GEOTECHNICAL ENGINEERING NOTEBOOK GEOTECHNICAL GUIDELINE NO. 15, "Geotechnical Differing Site Conditions" (1996) at 15.*
30. *See U.S. Army Corps of Engineers Pub. No. EM 1110-1-1804, GEOTECHNICAL INVESTIGATIONS MANUAL (Jan. 2001) at 13-3.*
31. *Id.*
32. *See U.S. Department of Transportation Pub. No. FHWA NHI-0509-037, GEOTECHNICAL ASPECTS OF PAVEMENTS REFERENCE MANUAL (2015) at 4.7.1 (emphasis added).*
33. *See Christie v. U.S., 237 U.S. 234, 240 (1915).*

34. See GEOTECHNICAL "DIFFERING SITE CONDITIONS," GEOTECHNICAL GUIDELINE NO. 15, U.S. Department of Transportation, Federal Highway Administration 16 (Apr. 30, 1996)

35. See, e.g., State Bank of Townsend v. Maryann's, Inc., 204 Mont. 21, 664 P.2d 295 (1983).

36. See NATIONAL UTILITY REVIEW: UTILITY COORDINATION PROCESS, U.S. Department of Transportation, Federal Highway Administration 2 (Oct. 2018).

37. Id. at 2.

38. Id. at 3.

39. Id. at 11-12.

40. Id. at 15-17; see also 23 CFR § 645.113.

41. Id.

42. See Restatement (Second) of Contracts § 202(3)(b) (1981).

43. See, e.g., Turner Const. Co., Inc. v. U.S., 367 F.3d 1319 (Fed. Cir. 2004).

44. See U.S. Army Corps of Engineers Pub. No. M1110-1-1804, GEOTECHNICAL INVESTIGATIONS MANUAL (Jan. 2001) at 8-3.

45. See CIVIL ENGINEERING HANDBOOK (2003 2nd ed.) at 15.1.

46. See Ehlers-Noll, GmbH v. U.S., 34 Fed. Cl. 494, 499, 503, 40 Cont. Cas. Fed. (CCH) ¶ 76874 (1995).

47. See Aleutian Constructors v. U.S., 24 Cl. Ct. 372, 378, 1991 WL 211506 (1991), quoting, J.L. Simmons Co. v. U.S., 188 Cl. Ct. 684, 412 F.2d 1360, 1362 (1969).

48. See Appeal of R.C. Hedreen Co., 77-1 BCA ¶ 12329, ASBCA No. 20599, 1977 WL 2266 (1977).

49. See, e.g., Jim's Excavating Service, Inc. v. HKM Associates, 878 P.2d 248, 255 (Mont. 1994).

50. See, e.g., State Bank of Townsend v. Maryann's, Inc., 204 Mont. 21, 664 P.2d 295 (1983).

51. While a contractor may be barred from obtaining payment from an owner due to the contractor's failure to meet applicable procedural requirements, these requirements do not apply to a negligence claim against an engineer. In addition, a claim is more likely to settle with the engineer's insurer able to pay in addition to the owner. And, finally, if the case does go to trial, a contractor is rightfully less concerned that a jury will side in favor of the engineer as it might be with the owner where it resides.

52. See Cunin, Soils Part II: Test Borings, The Construction Specifier 63 (1968).

53. See United Contractors v. U.S., 368 F.2d 585, 597 (Ct. Cl. 1966).

54. See Pleasant Excavating Co. v. U.S., 229 Ct. Cl. 654, 656 (Ct. Cl. 1981).

55. See Foster Const. C. A. & Williams Bros. Co. v. U.S., 435 F.2d 873, 888 (Ct. Cl. 1970).

56. See North American Society for Trenchless Technology, "Glossary of Microtunneling Terms and Definitions" at 5.

57. Id. at 4 (emphasis added).

58. See Meskele and Stuedlein, Driveability Analyses for Pipe-Ramming Installation (2011) at 11.

59. Hammerhead Trenchless Equipment, for example, publishes a list of recommended pipe wall thickness for pipe ramming projects based on pipe diameter and the length of the pipe ram. This has been accepted for the "minimum" wall thickness requirements.

60. See Gulf, M & O. R. Co. v. Illinois Cent. R. Co., 128 F. Supp. 311 (N.D. Ala. 1954), judgment aff'd, 225 F.2d 816 (5th Cir. 1955).

61. See, e.g., Guerini Stone Co. v. P.J. Carlin Const. Co., 248 U.S. 334, 340 (1919) ("It is sufficiently obvious that a contract for the construction of a building, even in the absence of an express stipulation upon the subject, implies an essential condition that a site shall be furnished upon which the structures may be erected.").

62. See, e.g., Horton Indus., Inc. v. Village of Moweaqua, 142 Ill. App. 3d 730, 492 N.E.2d 220 (5th Dist. 1986) (contractor not responsible for cost of owner and engineer being slow in making decisions).

63. See, e.g., Kenneth Reed Const. Corp. v. U.S., 201 Ct. Cl. 282, 475 F.2d 583 (1973) (contractor entitled to compensation for inspector requiring tolerance beyond that specified).

64. See, e.g., Appeal of Lee R. Smith, 66-2 BCA ¶ 5857, ASBCA No. 11135, 1996 WL 498 (1966) ("We are not aware of any case where the Change Conditions

clause has been interpreted as charging a contractor with knowledge of the conditions at the site that could not be discovered by a visual examination of the site.").

65. *See, e.g., I.A. Const. Corp. v. Department of Transp., 139 Pa. Commw. 509, 591 A.2d 1146, 1148 (1991).*
66. *See Morrison-Knudsen Co., Inc. v. U.S., 170 Ct. Cl. 712, 345 F.2d 535, 539 (1965). See also Douglas Northwest, Inc. v. Bill O'Brien & Sons Const., Inc., 64 Wash. App. 661, 828 P.2d 565, 577 (Div. 1 1992) ("[G]overnment cannot make the contractor the insurer of all government mistakes.").*
67. *See U.S. Department of Transportation, GEOTECHNICAL ENGINEERING NOTEBOOK GEOTECHNICAL GUIDELINE NO. 15, "Geotechnical Differing Site Conditions" (1996) at 7.*
68. *See Midwest Dredging Co. v. McAninch, 424 N.W.2d 216 (Iowa 1988), quoting, U.S. v. Spearin, 248 U.S. 132, 137 (1918).*
69. *See Pat J. Murphy, Inc. v. Drummond Dolomite, Inc., 232 F. Supp. 509, 521 (E.D. Wis. 1964), aff'd, 346 F.2d 382 (7th Cir. 1965).*
70. *Id. See also Betancourt & Gonzalez, S.E., 95-1 BCA ¶ 27,455, DOTCAB No. 2789 (1995).*
71. *See Kaiser Industries Corp. v. U.S., 169 Ct. Cl. 310, 340 F.2d 322, 329 (1965). Accord PT & L Construction v. Dept. of Transportation, 108 N.J. 539, 531 A.2d 1330 (1987).*
72. *See Douglas Northwest, Inc. v. Bill O'Brien & Sons Const., Inc., 64 Wash. App. 661, 828 P.2d 565, 576-77 (Div. 1 1992) (court rejected the government's argument that the contractor could not rely upon the soil borings because the conditions encountered were consistent with the general area). See also Kit-San-Azusa, J.V. v. U.S., 32 Fed. Cl. 647, 651 (1995), aff'd as modified and remanded, 86 F.3d 1175 (Fed. Cir. 1996).*
73. *See, e.g., Western Elec. Corp. v. New York City Transit Authority, 735 F. Supp. 1205 (S.D.N. Y. 1990) (bid documents required bidders to seek clarification).*
74. *See, e.g., Gardner-Zemke Co. v. State, 109 N.M. 729, 790 P.2d 1010, 1016 (1990) ("If [contractor [reasonably did not understand the meaning of blow counts, as it claims, it had a duty to ascertain what the term meant and its significance.").*
75. *See, e.g., Hunt Constr. Group, Inc. v. U.S., 281 F.3d 1369, 1374-75 (Fed. Cir. 2002).*
76. *See Metcalf Const. Co., Inc. v. U.S., 742 F.3d 984 (Fed. Cir. 2014).*
77. *See, e.g., Baltimore Contractors, Inc. v. U.S., 12 Cl. Ct. 328, 334 (Cl. Ct. 1987); Foster Const. C. A. & Williams Bros. v. U.S., 435 F.2d 873, 888 (Cl. Ct. 1970).*
78. *See PT & L Const. V. Department of Transportation, 108 N.J. 539, 531 A.2d 1330 (1987), quoting, Foster Constr. C.A. & Williams Bros. Co. v. U.S., 193 Ct. Cl. 587, 435 F.2d 873, 887 (19780).*
79. *The standard "Differing Site Conditions" clause, for example, that is required in all FHA and DOT projects, provides as follows:*
80. *During the progress of the work, if subsurface or latent physical conditions are encountered at the site differing materially from those indicated in the contract or if unknown physical conditions of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract, are encountered at the site, the party discovering such conditions shall promptly notify the other party in writing of the specific differing conditions before the site is disturbed and before the affected work is performed.*
81. *See, e.g., Section 1109.11.C of the Iowa Department of Transportation Standard Specifications for Highway and Bridge Construction (2012) ("In all cases, if this notification [for extra compensation] if not given, or if after notification is given, the Engineer is not afforded facilities for keeping strict account of actual costs as defined for force account construction, the Contractor thereby agrees to waive the claim for extra compensation for this work."); P & D Consultants, Inc. v. City of Carlsbad, 190 Cal. App. 4th 1332, 119 Cal. Rptr. 3d 253 (4th Dist. 2010) (The court denied the contractor's claim notwithstanding that changes were verbally discussed with owner's representative. "Whether a claim modification is oral or through conduct, a party contracting with a public agency is charged with the knowledge of public contracting law…The purpose of including a written change order requirement in a municipal works contract is obviously to protect the public from the type of situation that occurred here.").*

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