Surface Profiling and Jacking Force for Guided Boring Method

by

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Abstract

Trenchless technology has brought a new era in pipeline installation, overtaking the customary open cut method. Several new trenchless installation techniques are now practicable, among which Guided Boring Method (GBM), pipe jacking and microtunnelling are most remarkable. All these methods are performed by jacking. In spite of having numerous advantages, some aspects associated with the aforementioned trenchless methods still need to be improved.

GBM is one of the most widely used trenchless techniques to install pipelines with grade precision. As the length of these projects is much smaller than other underground projects (e.g. tunnels), conducting geotechnical investigations for GBM becomes unjustified at times due to budget constraints. Therefore, contractors will often conduct GBM installations without performing a proper subsurface investigation, which may lead to unusual consequences for the project. Since GBM consists of an initial pilot tube installation with further borehole reaming, the drilling parameters during the pilot tube installation phase may be used for subsurface profiling, which can help operators select proper drilling tools for the next reaming stages. This thesis firstly investigates the effectiveness of drilling parameters during pilot tube installation for subsurface profiling using drilling indices for a GBM project in Edmonton, Alberta, Canada. Five drilling indices, which were proposed for vertical drilling, were used to obtain the comparative strength of soil throughout the drive length. The results indicated that all five indices could identify the soil transitions in the drive length. However, the indices can only give a comparative strength measurement of soil throughout the drive length, not the exact soil strength.

Vertical earth load on pipes is an important aspect as it largely affects the jacking force during pipe jacking/microtunnelling. Although there are several methods to determine the normal load, the difference among them in calculating vertical normal stress is quite large. This thesis secondly provides an extensive analysis of nine different models/standards and their variations based on experimental data as well as a parametric study. The results show that some models predicted normal stress values close to the measured value, while the others give much higher or lower value than the measurement. This discrepancy of calculated normal stress using different methods from measured value is due to the variation of assumptions of different parameters. To identify the influence of parameters on normal stress, a parametric study is also conducted to reveal the effect of different parameters on arching factor as well as normal earth stress.

To my beloved

Mother

Father

&

Younger Sisters

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List of Symbols and Abbreviations

C_t: Load coefficient for jacked pipe

$W_e, W_t, W_{s,}$: Vertical soil load (kN/m)

B_d, B_t, B_{Th}: Bore diameter (m)

$\gamma_{b,}\gamma_{,}\omega$: Bulk unit weight of soil (kN/m³)

A: Area removed by drill bit (m^2)

ACPA: American Concrete Pipe Association

AI: Alteration index

ASCE: American Society of Civil Engineers

ATV: Abwasser Technische Vereinigung

B^*: Modified silo width (m)

B, b: Silo width (m)

c: Cohesion (kPa)

CIPP: Cured-in-Place Pipe

CPAA: Concrete Pipe Association of Australasia

CPT: Cone Penetration Test

D, D_p, d_p: Pipe diameter (m)

D_s: Diameter of shield (m)

E: Energy used for drilling (kJ/m)

GBM: Guided Boring Method

H, h: Overburden depth (m)

H_1: Depth of water table from surface

HDD: Horizontal Directional Drilling

HDPE: High-density Polyethylene

ISTT: International Society for Trenchless Technology

k, C_a: Arching factor

K, K_1: Ratio of active lateral unit pressure to vertical unit pressure

LED: Light Emitting Diode

MSE, E: Mechanical Specific Energy (kPa)

MTBM: Microtunnel Boring Machine

N: Rotation speed (rps)

NASTT: North American Society for Trenchless Technology

PCH: Powered Cutter Head

PJA: Pipe Jacking Association

PRH: Powered Reaming Head

PTMT: Pilot Tube Microtunnelling

r: Radius of the pipe (m)

RMR: Rock Mass Rating

RPM: Revolution per Minute

S_d: Somerton index

SPT: Standard Penetration Test

T: Rotation torque (kN·m)

TBM: Tunnel Boring Machine

V: instantaneous penetration rate (m/s)

VAF: Vertical Arching Factor

V_L: volume of the loosened soil (m^3)

W, F, WOB: Weight on bit (kN)

α: Percentage volume of the lost ground

β: Bulking factor

γ' : Submerged unit weight (below water table) (kN/m^3)

δ: Angle of wall friction

μ' : Coefficient of friction between fill material and sides of trench

σ_n: Normal stress

ϕ: Angle of internal friction of soil (°)

1. Chapter 1: Introduction

1.1. Background

The customary method for installing new pipelines has been to dig a trench along the pipeline and place the pipes. This process of installing pipe is referred to as open cut or open trench method. Open cut method requires following a strict set of rules in regards to detouring roads, storing excavated materials, managing ground water and restoring surfaces. These additional activities not only have an effect on cost, but also have a significant impact on the environment (Gottipati 2011). Trenchless technology, which refers to "a group of construction methodologies used to install, rehabilitate, and replace underground infrastructure while minimizing ground excavation, construction site footprint, and other social and environmental costs" (Olson 2013), has emerged as an alternative to traditional open cut methods. Several trenchless techniques are used for different purposes, among which Guided Boring Method (GBM) or Pilot Tube Microtunnelling (PTMT), pipe jacking and microtunnelling, are most remarkable. Pipe jacking and microtunnelling, as well as pilot tube auger boring, are from the same family of pipeline installation techniques (ISTT 2015). The common factor among all the aforementioned methods is they are all performed by jacking.

GBM or PTMT is a three-step installation process. Initially, pilot tubes are installed into the ground by jacking so as to maintain accurate alignment of further installation. In the second step, auger casings are installed, and in the third step, product pipes are installed. A launch shaft and a reception shaft are prepared to facilitate the jacking process. Due to budget constraints, trenchless personnel often commence GBM installations without performing geotechnical investigations. The shorter length of installation also influences the decision to take this risk. But sometimes the appearance of a soil layer completely different from the natural one makes the installation immovable. Since GBM or PTMT is a staged installation process, recorded drilling parameters at the first stage can provide ground information, which can help the next stages of installation.

Pipe jacking is a trenchless technique where "powerful hydraulic jacks are used to push specially designed pipes through the ground behind a shield at the same time as excavation is taking place within the shield" (PJA 1995). This process has no theoretical limit of drive length. The length can be extended up to several hundreds of meters using intermediate jacking stations. Microtunnelling is often used for installing small diameter pipe, a pipe size that does not allow the worker to enter into the pipeline. The fundamental of pipe jacking and microtunnelling is the same: they both push up specially designed pipe through the ground. For microtunnelling, the alignment of pipe is controlled remotely from the surface. Vertical earth load determination plays an important role in jacking force selection during pipe jacking or microtunnelling. Several standards and models can determine what that normal stress is, but no agreement has yet been made on which standard produces the best results in this field. A comparison among those standards and models based on field results is necessary to provide trenchless personnel a reliable process of determining vertical earth load.

1.2. Objectives

The objectives of the study can be summarized as below:

- To investigate the effectiveness of drilling indices in subsurface profiling for GBM. These indices have turned out to be effective during vertical drilling. The idea is to apply these indices during horizontal drilling and observe how it uses field data to identify the transition of soil layers. The best index in subsurface zoning will also be analyzed.

- To explore suitable standards or models for normal stress calculation during pipe jacking/microtunnelling. The discrepancy among currently used standards and models in predicting vertical earth stress propels the necessity of validation. Nine standards and models will be investigated and compared against measured field values for specific soil type to determine the most precise process of calculating normal stress. A parametric study will also be conducted to illustrate the effect of different parameters on normal stress.

1.3. Methodology

Based on literature review, five commonly used vertical drilling indices (Alteration index [Pfister 1985], Somerton index [Somerton 1959], Γ-hardness parameter [Bingham 1965], Mechanical Specific Energy [MSE] [Teale 1965] and Energy used for drilling [Pfister 1985]) were chosen for subsurface profiling for GBM application. Instrumentation was conducted to measure the jacking force and torque for a GBM project in Edmonton, Alberta, Canada. The field data was then used to calculate the selected five indices and plotted against time to see the variation of values. The potential of drilling parameter-based indices were later analyzed using those plots.

Based on literature review, nine methods, including the American Concrete Pipe Association (ACPA) (1987), Concrete Pipe Association Australasia (CPAA) (1990), Abwasser Technische Vereinigung (ATV) (1990), O'Rourke et al. (1991), Thomson (1993), Pipe Jacking Association (PJA) (1995), Bennett (1998), American Society of Civil Engineers (ASCE) (2000) and Staheli (2006), were selected for calculation of normal stress during pipe jacking/microtunnelling. The field data from Norris (1992) and Marshall (1998) was used for validation and comparison of these nine methods. Additionally, a parametric study was conducted to reveal the effect of different parameters on arching factor as well as normal earth stress.

1.4. Thesis structure

This thesis is presented in the following order:

- Chapter 1 – Introduction: In this chapter, a brief background of trenchless technology, especially GBM, pipe jacking and microtunnelling, is provided. Research objectives, methodology and thesis structure are also discussed.
- Chapter 2 – Literature review: In this chapter, GBM, pipe jacking and microtunnelling are described. Selected indices for subsurface profiling are discussed. An overview of different standards and models for calculating normal stress during pipe jacking or microtunnelling are also given.
- Chapter 3 – Subsurface profiling using horizontal drilling indices for GBM: In this chapter, five distinct indices are introduced for subsurface profiling during jacking. Recorded data from a GBM project are used to investigate the applicability of economically investigating the subsurface using drilling parameter-based indices. The potential of drilling indices as an alternative geotechnical investigation tool over customary methods is also analyzed.

- Chapter 4 – Comparison of different methods for normal stress calculation during pipe jacking/microtunnelling: In this chapter, some popular standards and models for measuring normal stress during jacking are highlighted. Later based on the field-measured value of normal stress during pipe jacking by Norris (1992) and Marshall (1998), the standards and models in question arevalidated for non-cohesive and cohesive soil. A parametric study is also presented to show the effect of different parameters on arching.
- Chapter 5 – Conclusion: In this chapter, research approaches, results and finding are summarized. Scope of future research is also suggested.

2. Chapter 2: Literature review

2.1 Trenchless technology

Trenchless technology has undergone rapid development in the last 40 years over the traditional open cut method for pipeline installation and underground asset management (Sterling 2010). Trenchless technology is defined as methods for utility or other line installation, rehabilitation, replacement, renovation, repair, inspection, location and leak detection with minimum excavation from ground surface (ISTT 2015). The North American Society for Trenchless Technology (NASTT) defines trenchless construction as, “a family of methods, materials, and equipment capable of being used for the installation of new or replacement or rehabilitation of existing underground infrastructure with minimal disruption to surface traffic, business, and other activities” (Ariaratnam et al. 1999). The birthplace of trenchless technologies is Europe. In North America, the trenchless technology industry was established in 1990 by NASTT (Gottipati 2011). However, the use of trenchless technology started in the 1860s by the Northern Railroad Pacific Company (Ariaratnam et al. 1999). They mainly used pipe jacking techniques, but later, construction personnel started utilizing other trenchless methods (i.e. auger boring [1940], impact moling [1962], directional drilling [1971], microtunnelling [1973], and pipe bursting [1980]) (Ariaratnam et al. 1999). Until 1980, unmanned trenchless installations were non-steerable in Germany, which prevented installing sewer lines and drainage systems using trenchless techniques (Stein 2003). The situation has changed in the last few years as trenchless techniques have been developed to include steerable techniques using pilot pipe jacking, microtunnelling with auger spoil removal, etc. A survey shows that around 10% of the respondents used trenchless technologies for 10% of new installations five years ago, and that quantity has now increased to 28% (Ariaratnam et al. 1999). Trenchless techniques have several advantages over open cut methods, such as less disturbance to the environment, less disturbance to traffic, less chance of interference with underground existing utilities, less exposed work area, and it is steerable, safe for workers, and provides opportunity to upsize pipes (Najafi et al. 2001). For open cut methods, the cost of excavation increases rapidly below the water table; this can be minimized using trenchless technology. In congested urban areas or busy traffic areas, trenchless technology is a highly preferable technique for new installations, rehabilitations or renewals.

Trenchless technologies can be divided into two categories. One is new installation methods and the second is rehabilitation methods. New installation methods are mainly concerned with new pipeline installations. On the other hand, rehabilitation methods are associated with replacing or repairing existing pipe. Pipe bursting is the only exceptional technique which involves installing new pipe and breaking old pipe simultaneously. Figure 2.1 shows the classifications of trenchless technologies.

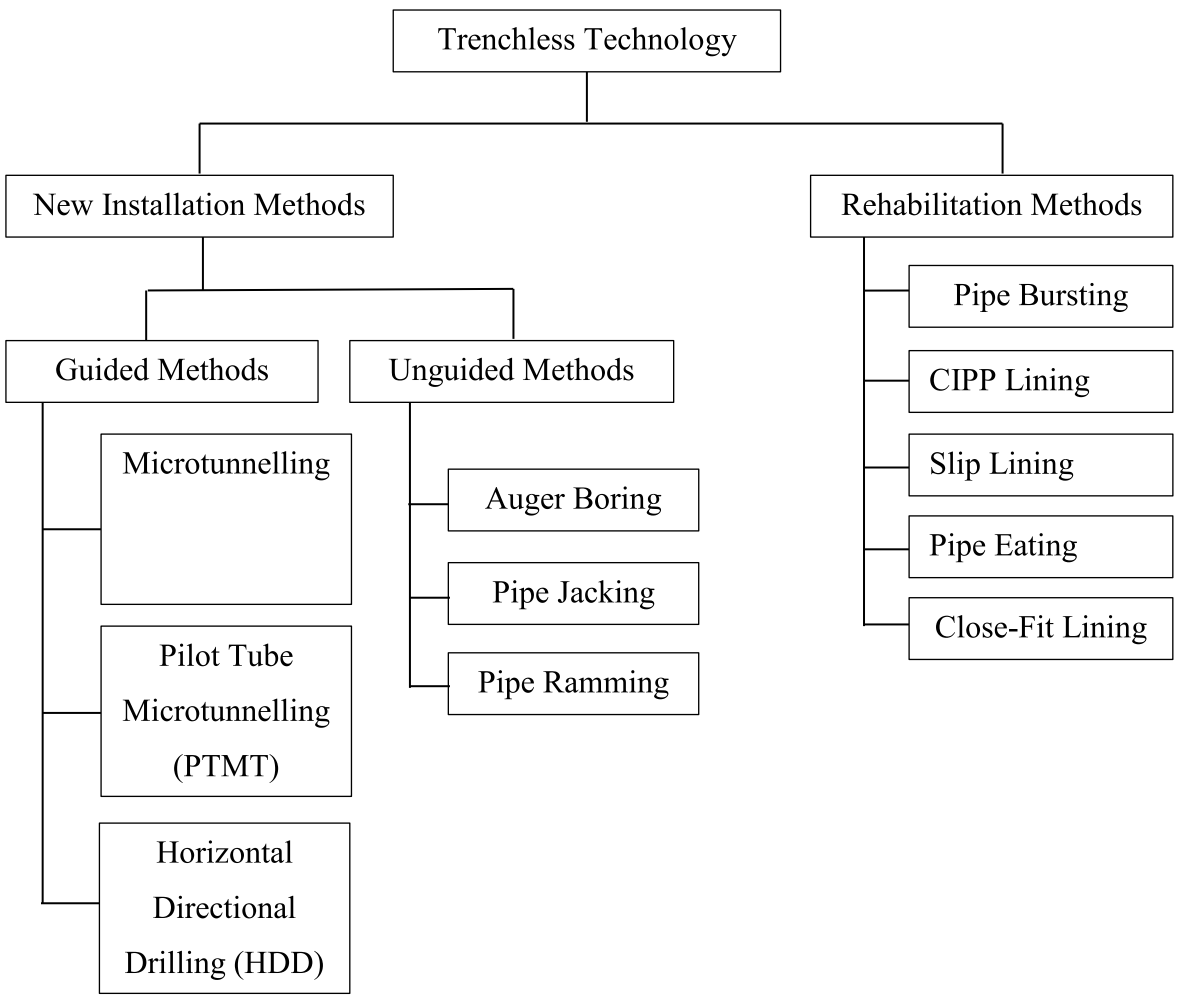


Figure 2.1 Family of trenchless technologies adapted from ISTT 2011 (Gottipati 2011)

This thesis focuses on three trenchless techniques: Guided Boring Method (GBM), pipe jacking, and microtunnelling. GBM is also known as Pilot Tube Microtunnelling (PTMT). All these techniques in question are trenchless techniques for new installation, and the common factor among these techniques is that they are all performed by jacking. Among these three, GBM and microtunnelling are steerable techniques, while pipe jacking is non-steerable.

GBM is a three-step installation process consisting of jacking pilot tubes through soil using a hydraulic jacking frame, jacking auger borings and jacking product pipes. Having the site's geotechnical information is important so decisions like selection of cutting head, proper jacking frame, etc. can be made before commencing GBM. Research has been carried out to provide trenchless personnel a geotechnical investigation tool that can provide subsurface information without performing costly soil investigations (e.g. Standard Penetration Test [SPT], Cone Penetration Test [CPT], etc.) Several indices consisting of drilling parameters have been implemented in vertical drilling for ground exploration, but they have not been introduced in the horizontal drilling sector. These indices may be effective for staged construction like GBM. Recorded drilling parameters during the pilot tube installation phase may prove to be effective in auger casings and product pipes installation phase since drilling parameters or a combination of drilling parameters have the potential to explore the subsurface.

Precise determination of normal stress is vital as it affects the jacking force and selection of jacking pipes directly. Several standards and models can predict normal soil stress values during pipe jacking or microtunnelling. Variation of normal stress values produced by those standards and models often confuse trenchless personnel about which method is the best to choose. No study shows a comparison of those methods with respect to field-measured values. A validation among those methods can help on deciding proper normal stress for design in future pipe jacking and microtunnelling constructions.

2.2 Guided Boring Method (GBM)

2.2.1. Description of GBM installation

The practice of GBM installations started two decades ago in Japan and Europe as part of a project installing 100 to 150-mm house connections (Boschert 2007). This method was introduced to the USA in 1995. GBM is a popular method to install small diameter pipelines with grade precision. This is a three-stage installation process starting with pilot tube installation and followed by auger casings and product pipes installation. This method is mostly applicable for sand with SPT value less than 50. GBM is also applicable for non-displaceable soil (SPT>50) with the help of eliminator tooling. The typical outside diameter range for product pipes of GBM is 0.10 m (4 in.) to 1.22 m (48 in.) (Akkerman 2015). The upper range of pipe diameter can be customized in terms of project needs. The common drive length for GBM is up to 122 m (400 ft). The first step is precise installation of pilot tubes as shown in Figure 2.2.

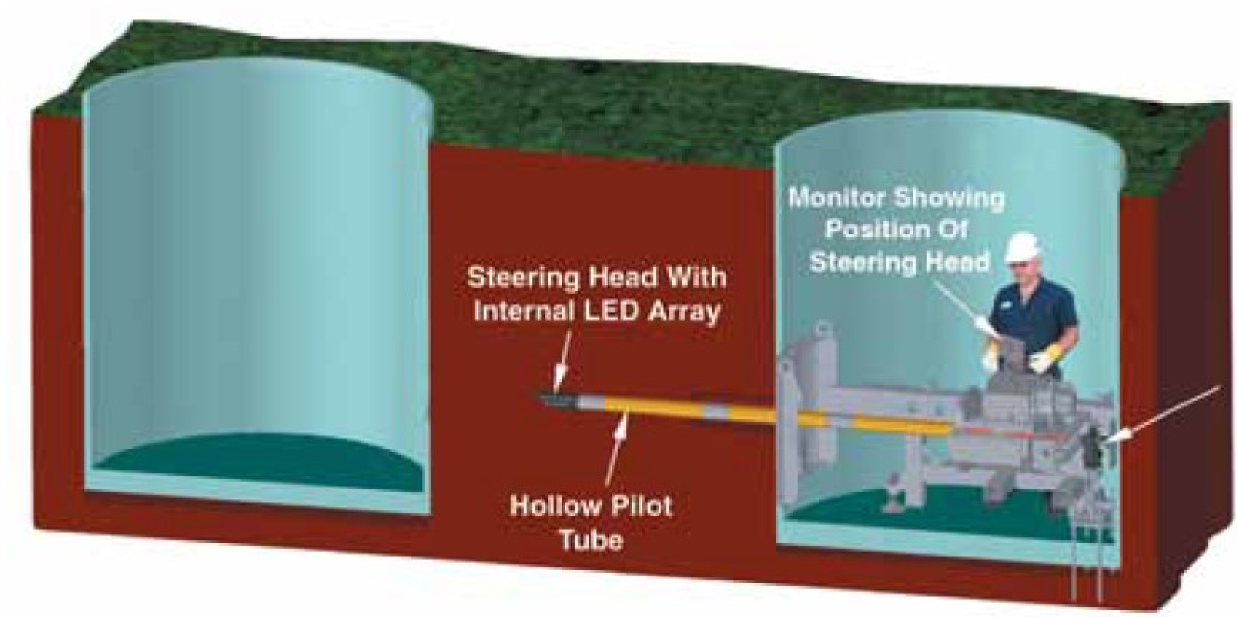


Figure 2.2 Precise installation of pilot tubes (Akkerman 2013)

Ordinarily, two shafts are prepared for installation. One is a launch shaft and the other is a reception shaft. Pilot tubes are inserted into the ground through launch shafts with proper alignment. A jacking frame is used to penetrate pilot tubes into the soil. A power pack provides hydraulic power to operate the jacking frame, and a steering head connects at the front side of the pilot tubes. Some steering heads are shown in Figure 2.3. A Light Emitting Diode (LED)-illuminated target is set at the steering head, which is visible through a theodolite, camera and monitor as shown in Figure 2.4. An operator controls the alignment using a crosshair appearing on the monitor. The position of the crosshair is maintained by a theodolite and LED-illuminated target through hollow pilot tubes. Step one ends when the pilot tubes reach the reception shaft. Sometimes lubrication is used to facilitate pilot tube insertion.

Figure 2.3 Various types of steering heads and supporting equipment adapted from Akkerman (2010) (Gottipati 2011)

Figure 2.4 Guidance system for GBM adapted from ISTT (2011) (Gottipati 2011)

The second step of GBM is to install auger casings along the pilot tube path as shown in Figure 2.5. Auger casings (Figure 2.6) are connected to a reaming head (Figure 2.7) as well as to the end of pilot tubes by special adapter. A jacking frame is used to push the auger casings, and the reaming head is used to reach the desired diameter of the borehole. The size of the reaming head is slightly more than the auger casings. The higher size of the reaming head reduces the friction between soil and auger casings' surface. Sometimes lubricants are used to reduce friction as well. As the auger casings advance, pilot tubes are removed one by one from the reception shaft. The spoil is carried through the augers to the launch shaft and removed manually or by special equipment. This step ends when the auger casings arrive at the reception shaft.

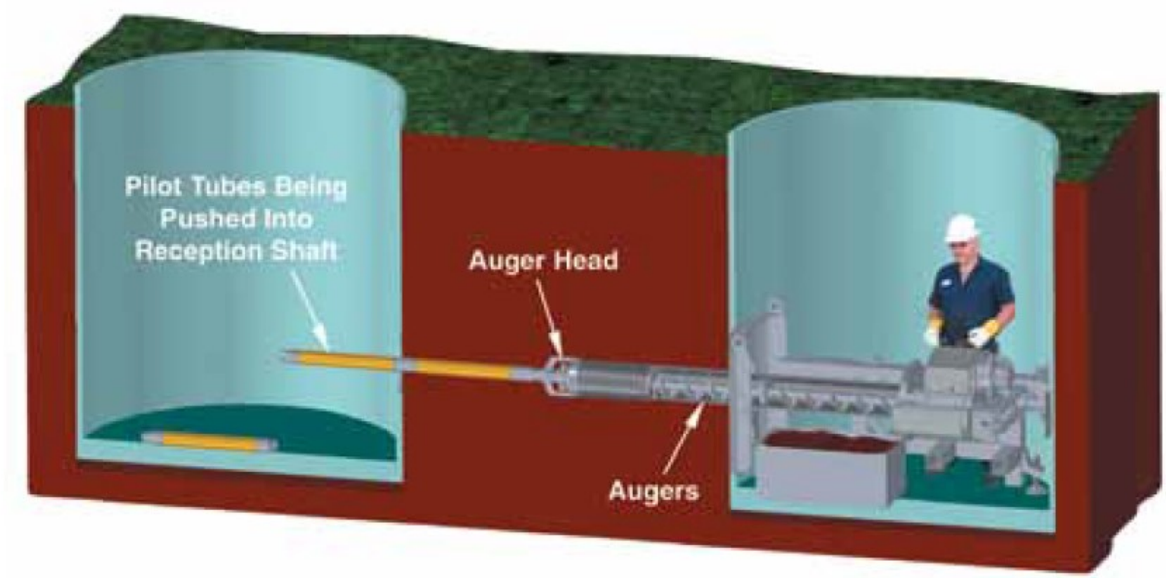


Figure 2.5 Auger casings' installation along the pilot tube path (Akkerman 2013)

Figure 2.6 Auger casings (Boschert 2007)

Figure 2.7 Reaming heads for GBM (Boschert 2007)

The third step is to install product pipes as shown in Figure 2.8. The product pipe is connected to the end of the auger casing through a pipe adapter, then the pipes are pushed through the borehole. On the other end, auger casings are removed one by one from the reception shaft. No spoils need to be removed in this stage. This step ends when the product pipes installation is completed.

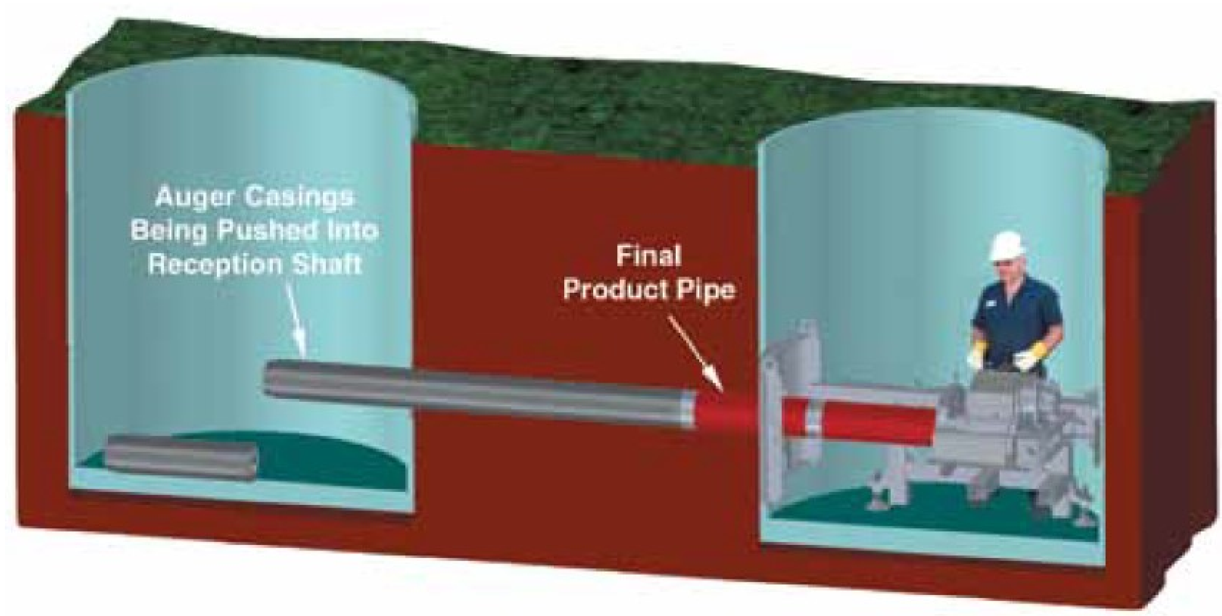


Figure 2.8 Final product pipes installation (Akkerman 2013)

GBM installation is not limited to a three-step installation process. GBM installations can be performed in two steps as well as three steps by using a hydraulic Powered Cutter Head (PCH) (Figure 2.10) or Powered Reaming Head (PRH) (Figure 2.11). In the two-step process, the first step remains same as the three-step method, but the second step is to install auger casings and product pipes simultaneously. Each product pipe contains auger casings and augers inside of it, as shown in Figure 2.9. The advantage of the two-step installation process is that contractors can use the same set of augers for multiple diameter pipe installations.

Figure 2.9 Product pipes ready for installation (Boschert 2007)

The modified three-step method is almost the same as the general three-step GBM installation. The only difference is a PCH or PRH is used after the auger casings' installation to increase the borehole size. PCH or PRH is connected to the end of auger casings, and as the PCH or PRH advances, it increases the borehole size and excavated material is transported to the reception shaft.

Figure 2.10 Powered cutter head from Akkaerman (Gottipati 2011)

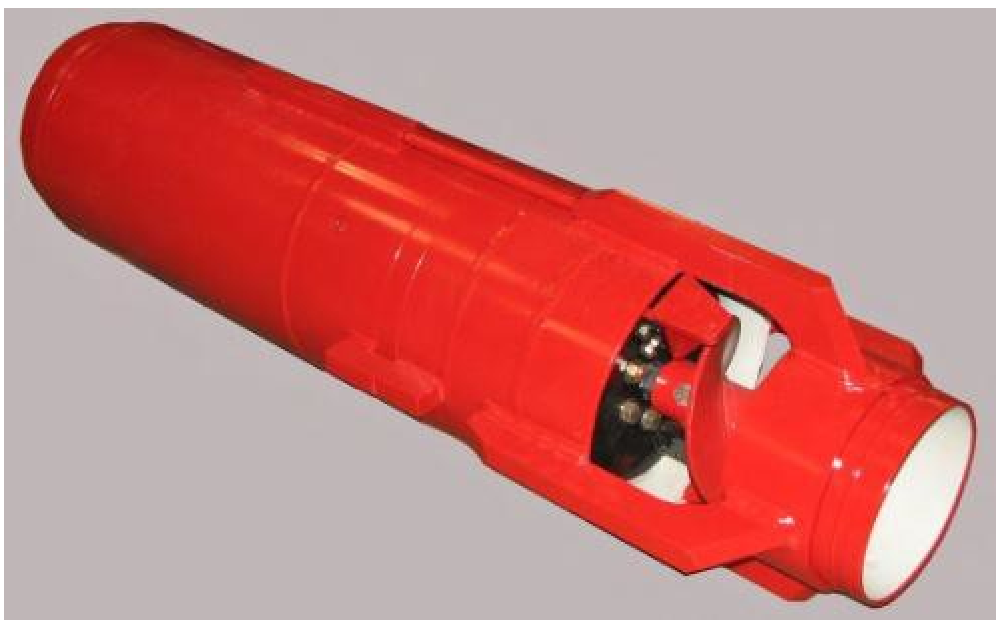


Figure 2.11 Powered reaming head from Akkerman (Gottipati 2011)

Advantages and limitations of GBM

Advantages:

- Accurate on-line and on-grade installation.
- Projects are often less costly than conventional open cut methods.
- Provides solutions to several engineering problems (e.g. utility obstacles, poor soils, deep installations and high ground water).

- Necessity of lift stations and associated maintenance costs are eliminated.
- Elimination of traffic delays, road closures, street repairs, citizen complaints, and reduction of contaminated soil disposal.
- Applicable for weak soils where sewer lines can be installed in zero blow count conditions.
- Possible to install pipes with close clearances to existing utilities.

Limitations:

- The guidance system is very critical. Any obstruction in pilot tube cavity may affect the guidance system.
- Removing any obstacles from the pilot tube cavity cost the project time and money.
- The technology is only applicable for selected soil conditions.
- Hard grounds (e.g. rocks) require additional special equipment for installation.
- Controlling flowing fluid may require extra effort during installation.

2.2.2. Problem statement for GBM

Contractors need to think about several issues, such as drill output, drill hazards and cost, before commencing drilling projects (Pfister 1985). These concerns have paved the way for a large number of emerging drilling techniques and tools. GBM is a current state-of-the-art technology to install pipelines with grade precision. The fundamental objective of GBM projects is economic profit. Several factors comprise the total cost of GBM projects, and optimizing some of these factors can reduce operational time and make the project cost-effective. When designing GBM pipe installations, designers require a detailed knowledge of the subsurface; a successful drilling is dependent on knowledge earned from preliminary investigations of the soil. Engineers have few methods to gain this knowledge without testing the medium. For ground exploration using traditional soil investigation methods (e.g. SPT, CPT, etc.), a large number of holes must be drilled to create a reliable image of the soil, but this process is costly. The necessity of ground information is counterbalanced by the necessity to control the cost during the early stages of the project. The length of GBM projects is usually shorter than other trenchless techniques (e.g. tunnel), which makes these projects restrained by their budgets. As the contractors are bound to finish the project with a fixed amount of money, they often avoid costly geotechnical investigations so as to prevent spending a large amount of money at the initial stage of the project. Several influencing factors (e.g. smaller project length, an idea about site soil type based on previous projects, etc.) push contractors to take the aforementioned risks. No such indicator specifies if these risks are worth taking or not. Being uncertain about taking risks makes GBM projects vulnerable in terms of expenses, even though the projects are of smaller lengths. Negligence of ground conditions in design may bring undesired consequences, and lack of subsurface knowledge may lead the installation process to an immovable condition, making the project even more expensive than conducting a geotechnical investigation.

GBM consists of an initial pilot tube installation with further borehole reaming. The installation of the pilot tube is similar to CPT, whereby a hydraulic pushing system penetrates a cone rig through the soil. Several indices consisted of drilling parameters (which will be presented in Section 2.4) have been implemented in vertical drilling for ground exploration, but they have not been introduced in the horizontal drilling sector. Hence, profiling the subsurface using drilling parameters during pilot tube installation may be an effective alternative tool to explore the subsurface for GBM projects, which is one of the major objectives of this study.

2.3 Pipe jacking and microtunnelling

2.3.1. Description of pipe jacking and microtunnelling

Pipe jacking originated in the United Kingdom in the early sixties to provide short crossings (e.g. rail roads, canals) (PJA 1995). This method developed with time and became practical for long tunnel installations. The main applications of pipe jacking are new sewerage and drainage construction, sewer replacement and lining, gas and water mains installation, electricity and telecom cable ducts installation, and subway installation. Pipe jacking is a process of driving prefabricated pipes through soil using hydraulic jacks from launch shaft to reception shaft. Thomson (1993) defined pipe jacking as a trenchless technique to push up pipe sections to line the hole formed by a cutting head or shield using hydraulic rams. A range of 800 mm to 5,000 mm diameter pipes can be installed using this method with an accuracy of ±1.25 in. (Clarkson and Thomson 1983). Using this method has no specific limit for drive length since intermediate jacking stations can be used. For jacking purpose, a thrust shaft (Figure 2.12) and reception shaft are constructed, usually at manhole level. Specific requirements and cost play key roles in constructing those shafts. Sometimes intermediate jacking stations are constructed for long drive lengths. This technique helps avoid too much jacking pressure on product pipe as well as curve drive. A thrust wall is prepared in the opposite direction of drive in the jacking shaft to prevent the reaction force of the jacking frame. In the case of low strength soil, special arrangements like piling are done to make the wall thrust restraint enough. For insufficient depths, sometimes structural frameworks are provided. A thrust rig (Figure 2.13) is used to distribute the jacking force evenly from jacking frame to product pipe. A guide rail is used at the thrust shaft to maintain the proper alignment of pipe initially. Each pipe section is lowered in the launch shaft, connected to the end of the previous pipe, and jacked through the ground. The product pipes are made in such a way so as to withstand the jacking force. During jacking, the excavated soil is removed through the hollow portion of jacking pipe either manually or mechanically. In both cases, the crew needs to stay inside the jacking pipe. The reception shaft is used to remove the jacking shield. In the case of driving pipes below ground water level, special arrangements are made to seal the thrust and reception assembly to prevent ingress of water.

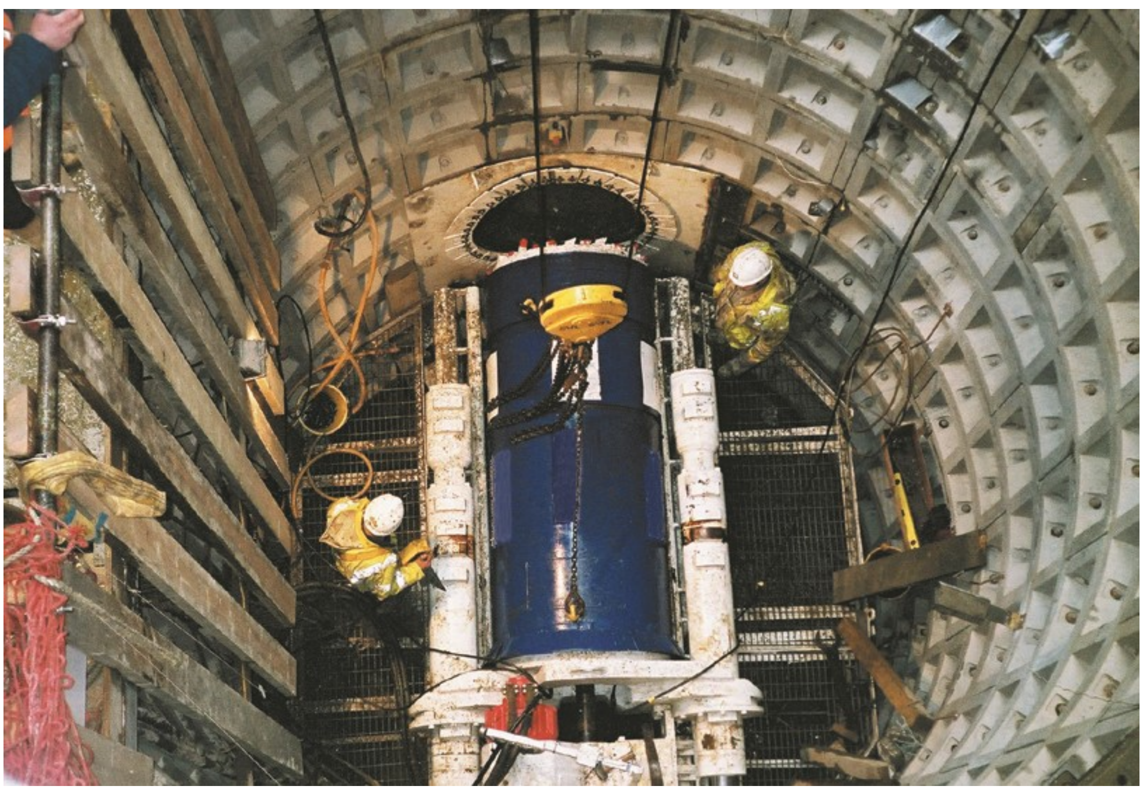
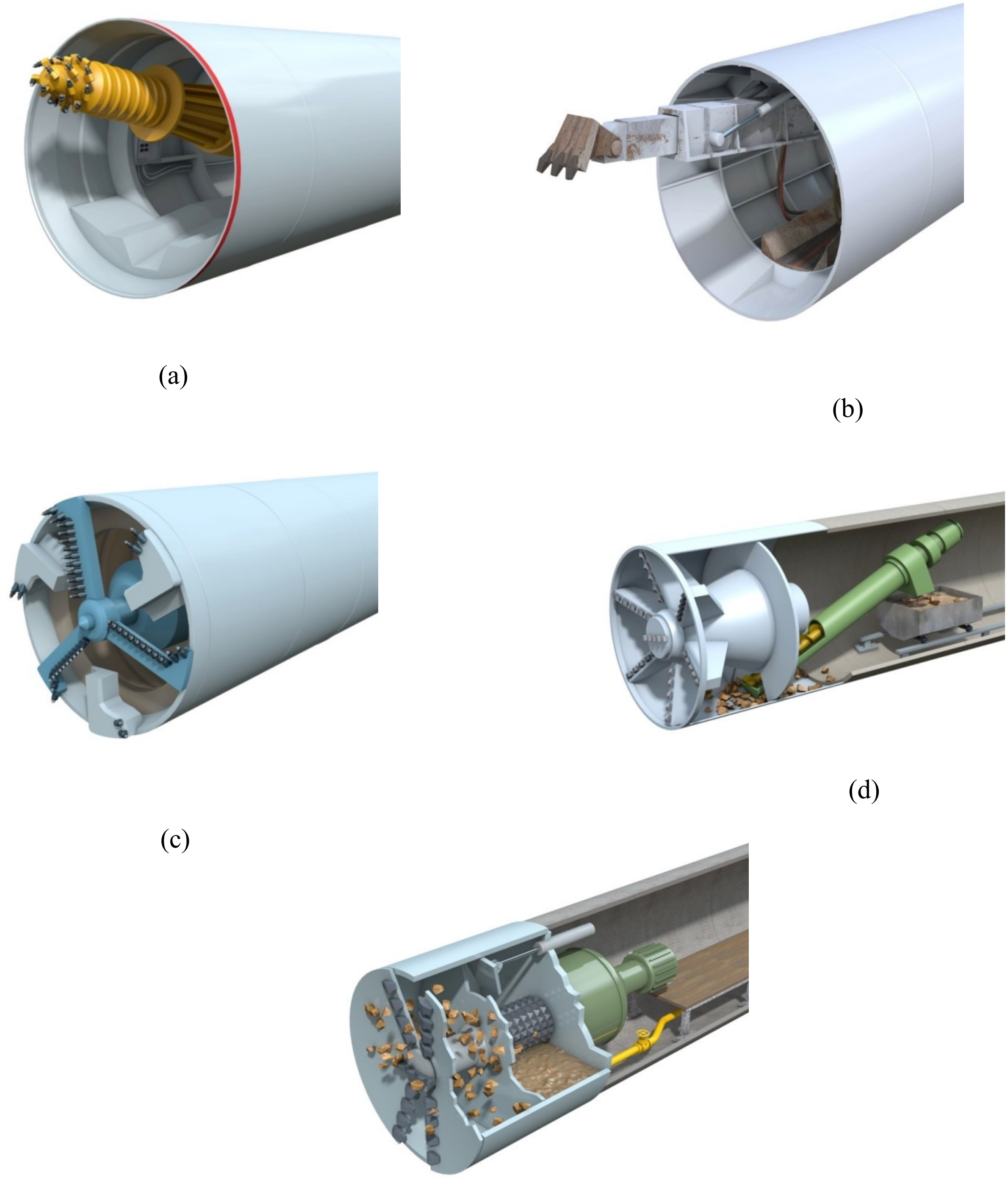


Figure 2.12 Thrust shaft for pipe jacking process (PJA 2014)

Figure 2.13 Jacking rigs (PJA 2014)

Several machines are available for excavation at face during pipe jacking. Backacters with mechanical hands, cutter boom shields, tunnel boring machine (TBM), TBM with earth pressure balance, and TBM with pressurized slurry are common examples (Figure 2.14).



(a)

(b)

(c)

(d)

(e)

Figure 2.14 (a) Backacters (b) Backacters with open face cutter booms (c) TBM with open face cutter booms (d) TBM earth pressure balance (e) TBM with pressurized slurry (PJA 2014)

TBMs are used in many forms during pipe jacking, and excavated face is supported using many techniques. Machines are divided into two main categories: slurry pressurized and air pressurized. Slurry pressurized machines use the pressure of slurry against ground water pressure for supporting face excavation; excavated materials are removed using slurry. Air pressurized machines use compressed air pressure against ground water pressure.

Microtunnelling can be defined as a steerable, unmanned technique for installing new pipes. PJA (1995) suggests pipe diameter less than 900 mm is known as unmanned size and applicable for microtunnelling. The principle for pipe jacking or microtunnelling is the same as they are both related in regards to pushing up pipes through soil. The difference is pipe size. Microtunnelling is controlled remotely by an operator from the surface, whereas pipe jacking is controlled by an operator at the face inside the pipe (Bennett 1998). The International Society for Trenchless Technology (ISTT) defines microtunnelling 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" (ISTT 2015). Support at the excavation face is an important aspect of microtunnelling that separates this method from pipe jacking. A jacking shaft and a reception shaft are prepared for pushing up pipes, then a microtunnel boring machine (MTBM) is used to excavate soil. Product pipes are lowered at the jacking shaft using a crane and connected to the MTBM or the end of the product pipe already driven through the soil. The other end of the pipe is connected to the jacking frame. Slurry lines and power cable connections are made. The slurry system helps remove excavated soil, and a slurry cleaning system is also used to remove soil from slurry. Additionally, a lubrication system is used to lubricate the exterior part of product pipes during installation. The pipes are pushed through the soil until they reach the reception shaft. At the completion of driving, MTBM is removed via reception shaft while a guidance system and steering jack are used to maintain proper allignment of pipes remotely. The process is shown in Figure 2.15. TBM auger or TBM slurry type machines adapted for smaller diameter are used for microtunnelling. These machines are equipped with an active target system or closed-circuit camera system for controlling line and grade remotely. Slurry machines transport the soil using slurry, and TBM auger machines transport the soil using augers.

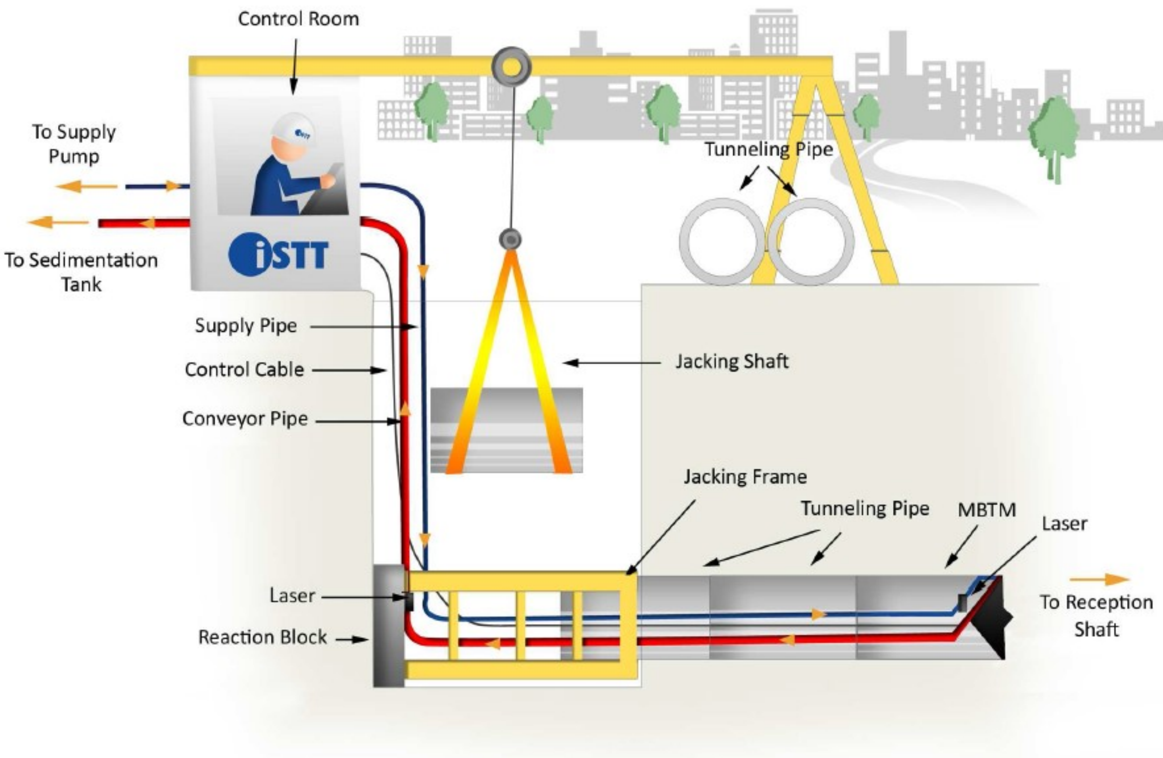


Figure 2.15 Pipe installation using microtunnelling (ISTT 2015).

(a)

(b)

Figure 2.16 (a) Microtunnelling machine (b) Installation by microtunnelling (PJA 2014)

Advantages and limitations of pipe jacking and microtunnelling

Advantages:

- Internal finish smoother than general tunnel.

- No secondary lining is required.
- Installed pipelines are watertight.
- There is no impact or minimal impact on surface.
- Fewer utility diversions required.
- Installation process is quick than open cut.
- Less space and man-hours required.
- Reduces disruption.
- Reduces damage to services.
- Less amount of spoil than open cut.
- Economic alternative to deep open cut.

Limitations:

- Technologies rely on a combination of planning, investigation and experienced applications. Absence of any can lead the project to failure.
- Recovery operations are difficult and costly.
- The control system becomes complicated in a long drive.
- Recommended for specific types of soil.

Commonly used pipes for pipe jacking and microtunnelling are concrete pipes, clay pipes, and steel pipes. Jacking pipes must be designed in a way to withstand jacking forces and constructed to specific standards.

2.3.2. Problem statement for pipe jacking and microtunnelling

Several aspects must be considered for a successful pipe jacking or microtunnelling installation. Jacking force plays a vital role throughout a pipe jacking or microtunnelling project since pipe material, joint limitation, stress requirement, lubrications, etc. are connected to it. One of the factors that influences jacking force most is the normal earth pressure on pipe, so determining precise normal stress over pipe crown is a matter of great importance for pipe jacking or microtunnelling. Trenchless personnel in different places of the world use their own standards or models to calculate this normal stress; however, these standards and models are different from each other and provide different stress values for the same soil conditions. Variations of assumptions considered by different standards and models may be the primary reason for the discrepancies of produced normal stress values by those methods. For example, ATV (1990) developed directives for stress calculation based on Terzaghi's (1943) trap door experiment. Since trenchless pipe installation is quite different from a trap door experiment, ATV (1990) modified some parameters (e.g. lateral earth pressure coefficient) in Terzaghi's (1943) original equations for stress calculation. Alternatively, O'Rourke et al. (1991) considered an elliptical soil failure over pipe crown; the loosened soil that occurs up to three to four times of pipe diameter over crown develops normal pressure. Staheli (2006) recommended a modified Terzaghis's (1943) equation to calculate normal stress for non-cohesive soil; the author assumed failure of soil over pipe crown. Staheli (2006) mainly modified silo width value from Terzaghi's (1943) equation to simulate pipe condition rather than trap door. Bennett (1998) proposed that the normal stress on pipe is a function of unit weight of soil and pipe diameter; the author provided specific arching factor values for specific types of soil based on field experiments. PJA (1995) assumed collapse of soil on pipe crown in developing a normal stress calculating equation for non-cohesive soil, but for cohesive soil, PJA (1995) assumed that the borehole would remain stable. Comparison among those methods has not been done based on field-measured normal earth stress value to give trenchless personnel a general idea about which standard or model provides the best result for a specific soil condition. It is also not clear why different standards and models provide different normal stress results for a certain soil condition. Hence, this is another focus of this study.

2.4 Subsurface profiling indices

Five popular indices have been discussed in this study. These indices have proven effective in subsurface exploration during vertical drilling. Several researchers carried out laboratory tests to correlate soil properties with drilling indices, and these indices may turn out to be an effective ground exploration tool in horizontal drilling too.

2.4.1. Somerton index (Somerton 1959)

Somerton (1959) introduced a term resistance to drilling, also known as Somerton index. A laboratory test was carried out to investigate the controlling factors for bit penetration in rotary drilling, then Somerton (1959) studied the influencing factors during rotary drilling to minimize the applied energy for rock breakage. Somerton index is a relative measurement of soil strength without using torque parameter. This index contains jacking force, revolution per minute (RPM) and rate of penetration parameters, and the index value has no specific range. A comparatively smaller value indicates softer soil, and a higher value indicates harder soil. The index is calculated by equation 2.1.

$$S_d = W \times \left({}^{N}\!/_{V} \right)^{1/2} \qquad \textbf{2.1}$$

S_d = Somerton index

W = weight on bit (thrust – retention force + weight of rods and bit) (kN)

N = rotation speed (rps)

V = instantaneous penetration rate (m/s)

2.4.2. Γ-hardness parameter (Bingham 1965)

Γ-hardness parameter was developed by Bingham (1965) and gives a relative measurement of the hardness of soil. Higher hardness value means hard to drill, which does not represent only hard soil. Even clay may clog the drilling bits and increase the hardness value. Easily removed drilled materials largely affect the measurement of hardness of soil. Γ-hardness parameter is calculated by equation 2.2.

$$\Gamma\text{-hardness} = NFD^2/VT \qquad \textbf{2.2}$$

N = rotation speed (rps)

F = thrust applied on the drilling bit (kN)

D = bit diameter (m)

V = instantaneous penetration rate (m/s)

T = rotation torque (kN·m)

2.4.3. Mechanical specific energy (MSE) (Teale 1965)

Mechanical Specific Energy (MSE) was derived mainly for rock breakage and is defined as the energy required to excavate one unit volume of rock. Teale (1965) suggested that specific energy and compressive strength are both functions of rock strength, so there must be a relation between compressive strength and specific energy. Studies showed that the energy required to crush rocks and the compressive strength of rocks are very close to each other; therefore, MSE can be used as an index to monitor drilling efficiency during rock drilling. One other way a specific MSE value can be used is as an indication for a specific type of drilled material or rock mass classification. MSE can also be correlated to other functions where compressive strength is not the best measurement of strength for drilled material (e.g. shear strength of soil). MSE is calculated using equation 2.3.

$$MSE = {}^{F}\!/_{A} + 2\pi NT/AV \qquad \textbf{2.3}$$

MSE = Mechanical specific energy (kPa)

F = thrust on bit (kN)

A = area removed by drill bit (m^2)

N = rotation speed (rps)

T = rotation torque (kN·m)

V = drilling speed (m/s)

2.4.4. Alteration index (Pfister 1985)

Alteration index was developed by Pfister (1985). This index gives a comparative strength measurement of soil and is very sensitive for low to medium strength soil. The value of Alteration index varies from 0 to 2. 0 is an indication of softer soil while 2 indicates harder soil. This index only gives a comparative strength profile throughout the drive length, but it cannot specify the exact type of soil based on a standard scale. Still, this index can play an important role by giving an idea of comparative ground information in the absence of a geotechnical investigation. Alteration index is calculated using equation 2.4.

$$AI = 1 + \left({}^{W}\!/_{W_{max}}\right) - \left({}^{V}\!/_{V_{max}}\right) \qquad \textbf{2.4}$$

AI = Alteration index

W = weight on the bit (thrust – retention force + weights of rods and bit) (kN)

W_{max} = theoretical maximum value of W (kN)

V = instantaneous penetration rate (with maximum value V_{max}) (m/s)

2.4.5. Drilling energy (Pfister 1985)

Energy used for drilling was developed by Pfister (1985). This energy concept is different from MSE developed by Teale (1965) as this theory has only three parameters (torque, rate of penetration and revolution per minute). This concept is very effective for hard soil and soft rocks. However, this theory is not very useful where variation of jacking force is much higher than the variation of other drilling parameters as the equation does not contain jacking force parameter. Drilling energy defined by Pfister (1985) can be calculated by equation 2.5.

$$E = {}^{TN}\!/_{V} \qquad \textbf{2.5}$$

E = Energy used for drilling (kJ/m)

T = value of the rotation torque (kN·m)

N = rotation speed (rps)

V = instantaneous penetration rate (m/s)

Rotary drilling through the rock for geotechnical investigations can be improved in terms of quality and quantity of data by recording additional operating variables (e.g. thrust, torque, etc.) (Brown and Barr 1978). Brown and Barr (1978) proposed that it is possible to develop a relation between these variables and compressive strength of rock. A continuous record of these variables may provide information about rock properties, and tt is easier to determine rock properties by relating drilling variables than by doing laboratory experiments on drilled materials.

Pfister (1985) discussed three indices to interpret drilling parameters. During drilling, the appearance of a new soil layer brings change in the intensity of one or more drilling parameter. For a specific type of soil, one specific drilling parameter becomes predominant over others. Pfister (1985) used drilling energy, Alteration index and Somerton index for investigating soil properties. The probability of quantifying traditional SPT and CPT test results using drilling parameters was also discussed.

Zacas et al. (1995) used Stiffness index for limestone quality classification. This stiffness is actually the Alteration index developed by Pfister (1985). This index is very useful to classify medium to low strength soil and can also be used to indirectly identify hardness and weathering of rocks. Zacas et al. (1995) suggested that a significant amount of money and time can be saved by analyzing recorded drilling parameters for large projects (e.g. investigation program of foundation conditions).

LaBelle (2001) carried out laboratory experiments to develop a lithological classifier using drilling parameters. The experimental apparatus consisted of an instrumented coal mine drill and computerized system for collecting data as shown in Figure 2.17.

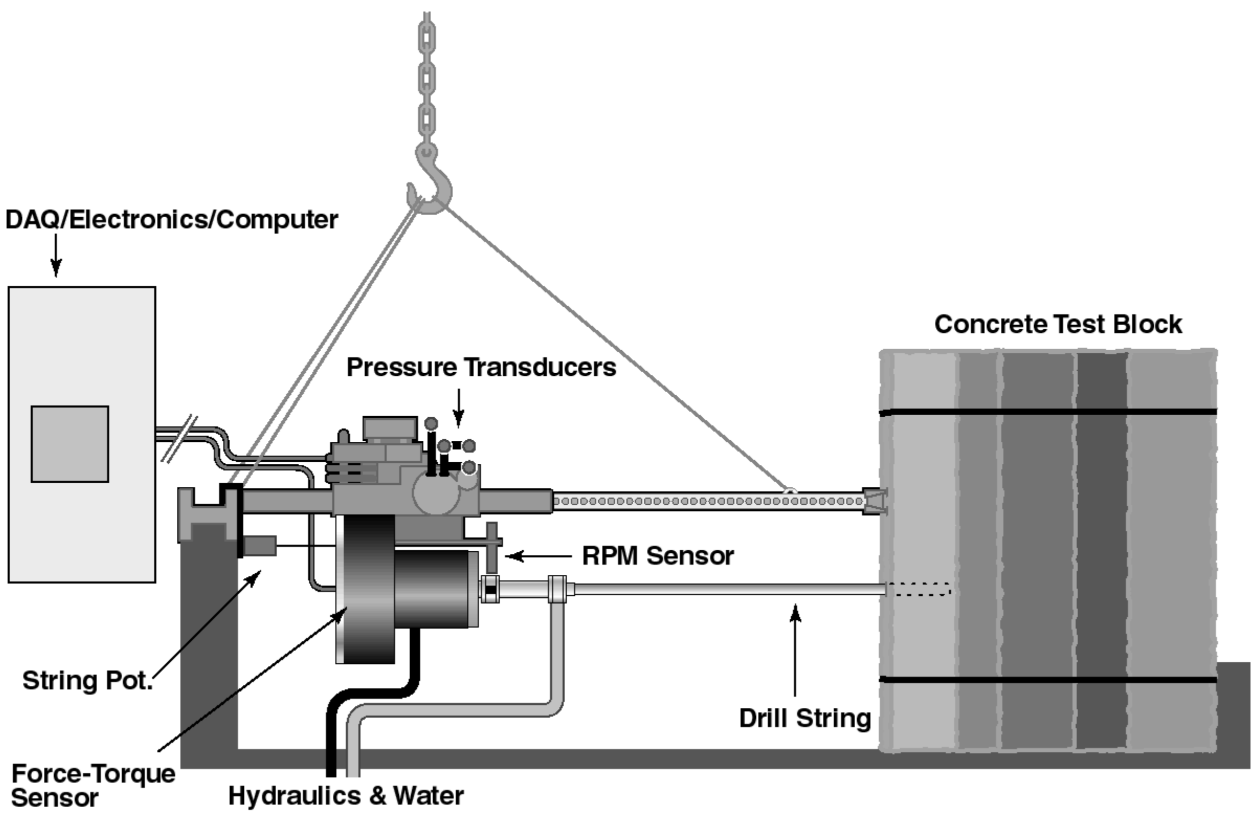


Figure 2.17 Laboratory drill apparatus and set-up (LaBelle 2001)

Several sensors were placed on drilling apparatus to record torque, RPM, and thrust. The test was conducted on a 0.91 m × 0.91 m × 1.52 m (3 ft × 3 ft × 5 ft) block of five different rock layers, all of which had different strengths. 40 holes were drilled on that test block, and drilling parameters were recorded during drilling using sensors. The objective was to investigate if the drilling parameters can identify the lithological changes. The author used an artificial neural network system and trained it with known values. The author suggested that drilling parameters can be used to classify rock layers and that additional features derived from drilling parameters can improve the accuracy of classification.

Gui et al. (2002) tried to interpret drilling parameters qualitatively and quantitatively. Drilling parameters were recorded in a field test conducted at Kennington Park, London. A hydraulically operated rotary type drilling rig was used for drilling. The drilling rig (Figure 2.18) was equipped with real-time datalogger ENPASOL-3 to monitor and measure drilling parameters. Seven distinct drilling parameters, which are fluid pressure, torque, thrust on bit, hold-back, time, drilling speed and rotational speed, were measured.

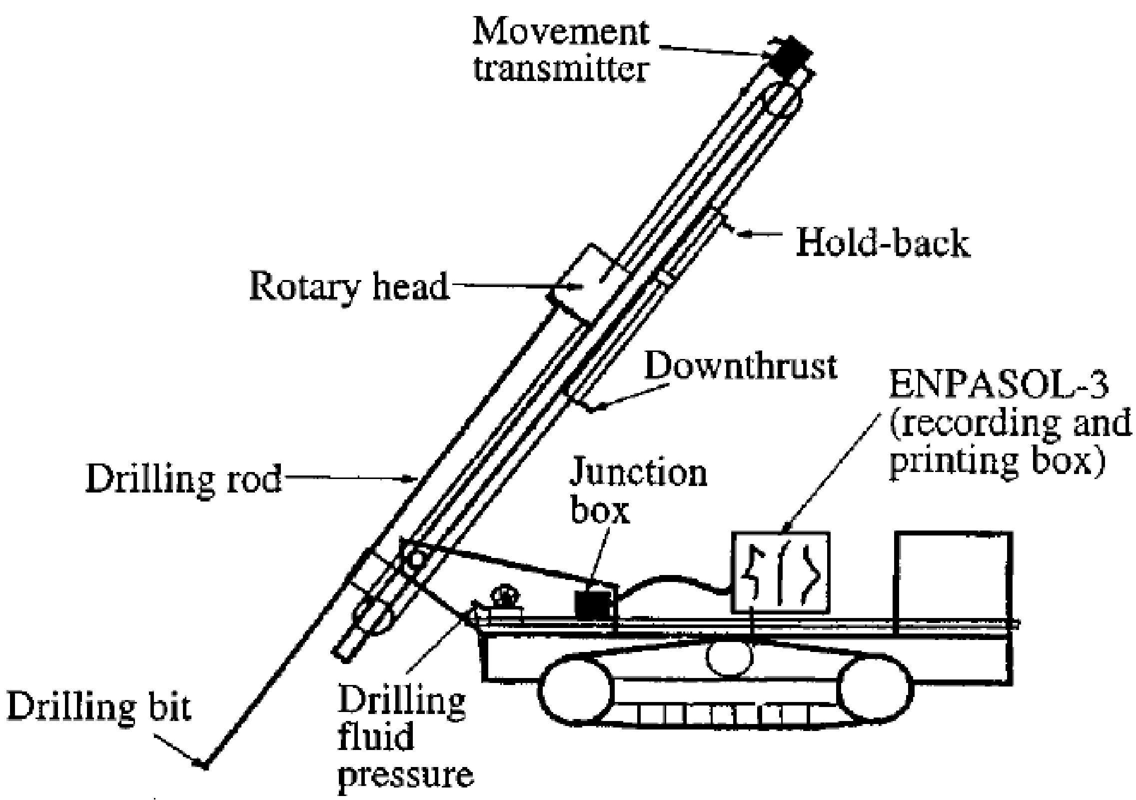


Figure 2.18 Typical instrumented rotary drilling rig (Gui et al. 2002)

Gui et al. (2002) attempted to interpret soil formation change using indices combined of drilling parameters. Alteration index profile was plotted for the test site. An almost-constant index value was found for London clay, but other materials (limestone, terrace gravel) showed fluctuations. The authors tried to develop a relation between drilling parameters and undrained shear strength of soil based on a series of instrumented borehole drilling. However, no confidence could be placed in determining shear strength of soil from drilling parameters analysis as the degree or correlation between them was very poor.

The nature of weathered rock is highly variable, and that poses significant challenges in geotechnical design of underground installations (e.g. tunnel). Determining a geotechnical profile in rock by borehole investigation is time consuming and costly. Fonseca and Coelho (2006) suggested recording drilling parameters may be a complementary alternative to this site investigation. Fonseca and Coelho (2006) studied the contributions of Somerton index, drilling energy, specific energy and Γ-hardness parameter in evaluating weathered zones of rock; they proposed that the zonation of rock can be done effectively using drilling parameters analysis, but a limit for each parameter needs to be defined for each specific rock type.

Celada et al. (2009) used MSE as an index for geotechnical site characterization. A set of lab experiments were carried out on five distinct types of materials (shales, sandstones, schists, coal and massive sulphide) to correlate specific energy with three geotechnical properties of drilled materials: Rock Mass Rating (RMR), number of joints per meter, and rock mass uniaxial compressive strength. The test result showed clear evidence that specific drilled material has specific correlation with MSE. Later, drilling parameters were recorded during a San Pedro tunnel construction using TBM to develop a relationship between MSE and geotechnical properties of drilled materials. Celada et al. (2009) suggested that MSE can be an interesting tool for site depiction.

By collecting some additional information during drilling, unfortunate drilling incidents that increase operational time can be eliminated (Solberg 2012). Problems during drilling are mainly created by the appearance of hard soil layers. Solberg (2012) proposed that these hard or soft layers can be detected by recording and comparing drilling parameters (e.g. torque, rate of penetration, etc.). A decrease in RPM and an increase in weight on bit represent the appearance of a hard soil layer. Analyzing and comparing these parameters manually is time consuming;

therefore, Solberg (2012) introduced a hardness detection program based on the concept of rate of penetration for identifying different soil layers.

Bevilacqua et al. (2013) asserted that a successful oil drilling is only possible by reducing drilling time and increasing drilling performance. Geology of the drilled formation has a great influence on drilling performance. For rocks, the dominating geological properties are compressive strength and abrasiveness. Several techniques can identify the resistance and abrasiveness of the rocks (e.g. gamma ray emission of soil to identify presence of clay, compressive acoustic transit time measurement to find out porosity of rock and type of materials, determination of mechanical resistance of rocks) (Bevilacqua et al. 2013). Mechanical resistance of rocks can be determined by two ways: 1.) by measuring unconfined compressive strength, and 2.) by measuring confined compressive strength. Bevilacqua et al. (2013) suggested that monitoring of MSE is a key element in maintaining drilling performance. Unexpected change in MSE represents either formation change, drilling inefficiency, or both (Bevilacqua et al. 2013).

Single or compound measurements of drilling parameters can clearly provide a qualitative evaluation of soil types (Laudanski et al. 2014). Laudanski et al. (2014) advised that the statistical analysis of results obtained from customary geotechnical investigations can help to develop an empirical relationship between soil properties and drilling parameters. The authors conducted a massive study of drilling parameters on a specially constructed test embankment. The study showed how soil can be evaluated qualitatively using drilling parameters. Seven distinct indices, including Alteration index, Somerton index, and specific energy, were used to interpret recorded drilling parameters. Five different drill bits (Figure 2.19) were used to see the effect of bit variation on drilling.

Figure 2.19 Drilling bits used: 1: button bit, 2: cross-type bit, 3: drag bit, 4: bicone roller bit 5: continuous flight auger (Laudanski et al. 2014)

The test embankment (Figure 2.20) was 3 m high with eight separated zones on the surface. Six of the zones had a single layer of soil, and the other two zones had multiple layers. The single-layer zones were constructed of clay, sandy clay, sand, chalk, gravel and silt.

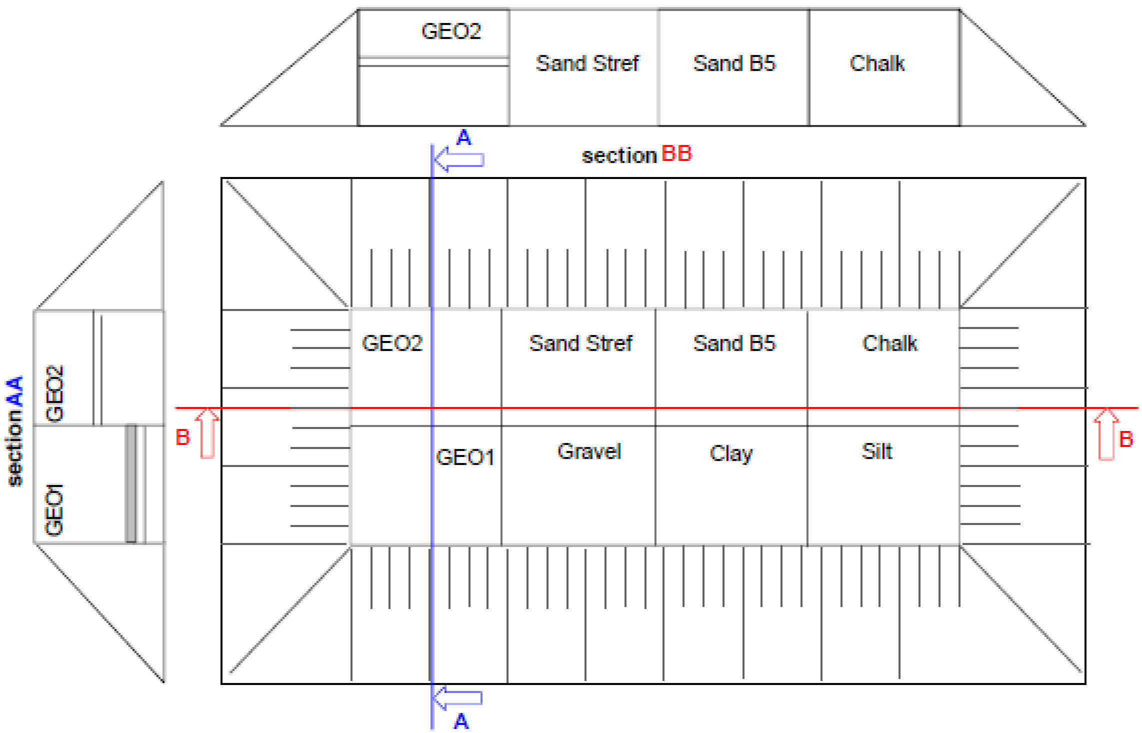


Figure 2.20 Schematic plan of the embankment built by Rouen Experimental Station (Laudanski et al. 2014)

Drilling parameters were recorded during drilling for different bits and soil layers, and later, indices were calculated and plotted to see the correlation of indices with soil properties. The result for each tool was clearly dependent on soil type, and results for various tools and various soil types were different from each other. The research revealed the potential of drilling parameters to locate the transition of soil layers. Laudanski et al. (2014) correlated the combined drilling parameters analysis results with traditional field test results (i.e. SPT, CPT). The authors suggested that the combined parameters undoubtedly have physical significance that can provide reliable quantitative value of soil properties.

Ngerebara and Youdeowei (2014) conducted laboratory tests on five different types of rocks (lateritic soil, light grayish shale, dark grayish shale, carbonate rock, dolerite) to establish several rock properties. The investigated rock properties were compressive strength, point load strength, mean impact strength and density. Sample rocks were collected from eastern Nigeria during rotary drilling. Time during drilling was recorded and rate of penetration for each rock type was calculated later. The authors tried to correlate different rock properties with net rate of penetration; they found the strongest correlation between rock compressive strength and rate of penetration. The authors also concluded that rate of penetration drill bits is inversely proportional to specific energy value.

2.5 Standards and models for calculating normal stress on jacking pipes

2.5.1. ACPA (1987) and CPAA (1990)

The American Concrete Pipe Association (ACPA) and Concrete Pipe Association Australasia (CPAA) suggested the same formula to calculate earth pressure on jacked pipes (ACPA 1987; CPAA 1990). This is an indirect design method similar to the Marston-Spangler trench installation method (Spangler 1960). The load on jacked or tunnelled pipe is calculated by equation 2.6 and 2.7.

$$W_t = C_t \times \omega \times B_t^2 - 2c \times C_t \times B_t \quad \textbf{2.6}$$

$$C_t = \frac{1 - e^{-2K_\mu' \times \frac{H}{B_t}}}{2K_\mu'} \quad \textbf{2.7}$$

W_t = earth load (kN/m)

C_t = load coefficient for jacked pipe

ω = unit weight of soil (kN/m^3)

B_t = maximum width of tunnel bore excavation (m)

c = cohesion of the soil above the excavation (kPa)

K = ratio of active lateral unit pressure to vertical unit pressure

μ' = coefficient of friction between fill material and sides of trench

H = overburden depth (m)

The suggested $K\mu'$ values are 0.165, 0.150, 0.130 and 0.110 for sand and gravel, saturated top soil, ordinary clay and saturated clay, respectively (ACPA 1987). Here, $C_t \times \omega \times B_t^2$ is the same as calculating backfill load on a pipe in a trench. On the other hand, $2c \times C_t \times B_t$ incorporates the cohesion of undisturbed soil, which reduces the total earth load on pipe (CPAA 1990).

2.5.2. ATV (1990)

Abwasser Technische Vereinigung (ATV) specifications were developed to form directives for stress and strain analysis of jacking pipes since a customary calculation does not cover the stress calculation for jacking pipes sufficiently (Stein et al. 1990). The specifications are applicable for circular-type jacking pipes made of reinforced concrete, asbestos cement and steel. These are suitable for pipes installed in both non-cohesive and cohesive loose soil along straight and curve routes (Stein et al. 1990).

The equation was developed on the basis of Terzaghi's trap door experiment (Terzaghi 1943). Terzaghi's trap door experiment provides equation 2.8 for a normal soil stress (P_{EV}) calculation.

$$P_{EV} = \gamma \times h \times k \qquad \textbf{2.8}$$

Here, k is the arching factor.

P_{EV} is the normal soil stress (kN/m^2)

For cohesionless soil, the arching factor becomes

$$k = \frac{1 - e^{-2 \times K \times tan\delta \times \frac{h}{b}}}{2 \times K \times tan\delta \times \frac{h}{b}} \qquad \textbf{2.9}$$

Arching factor is a function of overburden depth (h), silo width (b), soil density (γ), coefficient of lateral earth pressure (K) and angle of wall friction (δ) in the plane of shear. Overburden depth and soil density can be calculated precisely in the site, but the other parameters must be assumed based on soil behaviour.

According to ATV (1990), the boundary line of the wedge of failure starts from the side of the pipe below the angle of $v = 45^\circ + \phi/2$ (corresponding to calculation 60° with all loose rock), propagates diagonally, and passes the vertical line of shear plane at the level of pipe crown. The angle of wall friction is dependent on shear shift of soil. From soil mechanics, it is known that one half of the angle of wall friction δ is activated with 10% of maximum shear shift. Higher overcut value may activate δ close to ϕ, which will certainly reduce normal stress on pipe, but a higher overcut may cause settlement on the surface. So to remain conservative, ATV (1990) assumes the angle of wall friction is equal to half of the angle of soil internal friction.

The vertical load on pipe according to ATV (1990) becomes

$$P_{EV} = k \times \gamma_B \times h \quad (\mathrm{kN/m^2}) \qquad \textbf{2.10}$$

where

$$k = \frac{1\text{-}e^{-2K_1 \times \tan(\phi/2) \times \frac{h}{b}}}{2K_1 \times \tan(\phi/2) \times \frac{h}{b}} \qquad \textbf{2.11}$$

K_1 = coefficient of horizontal soil pressure above silo wall

b = ideal silo width ($\boldsymbol{b} = \sqrt{\mathbf{3}} \times$ d) (m)

h = height of cover (m)

ϕ =Angle of internal friction of soil (˚)

γ_B = soil density (kN/m³)

Modern pipe jacking equipment causes minimal disturbance of overlaying soil. ATV (1990) considers the lateral earth pressure coefficient value K=0.5, which represents static soil condition (Stein et al. 1989). ATV (1990) disregards cohesion of soil, although presence of cohesion certainly reduces the vertical stress on pipe. For jacking below the ground water table, ATV (1990) recommends considering full soil load (K=1.0) to remain safe.

2.5.3. O'Rourke et al. (1991)

O'Rourke et al. (1991) described soil load transfer mechanism on auger bored pipes based on the assumption that load is a function of soil height up to which collapse occurs above the pipe crown (Castronovo 1991). He introduced the term *bulking factor* for calculating normal stress of loosened soil.

Bulking factor is defined as

$$\beta = \frac{V_L - V_O}{V_O} \qquad \textbf{2.12}$$

Where V_L (m^3) is the volume of the loosened soil extended to a depth of b (m) and V_0 (m^3) is the initial soil volume. V_0 is further defined as:

$$V_O = \frac{\pi \times B_d}{8}[2b - B_d \times (1{+}2\alpha)]\ (\mathrm{m}^3) \qquad \textbf{2.13}$$

And V_L (m^3) is expressed as

$$V_L = \frac{\pi \times B_d}{8}[2b{+}B_d - 2D \times \left(\frac{D}{B_d}\right)]\ (\mathrm{m}^3) \qquad \textbf{2.14}$$

In which B_d is bore diameter (m), D is external pipe diameter (m), α is percentage volume of the lost ground attributed to difficulties in excavation and $b = H_e{+}D - {B_d}/{2}$ (m). Equations are developed based on elliptical collapse or loosening of soil over pipe crown. For smaller values of bulking factor, the loosening soil depth may extend up to three and four times that of bore diameter (Castronovo 1991).

Vertical soil load on pipe is expressed as

$$W_e = \gamma \times {B_d}^2 \times C_d\ (\mathrm{kN/m}) \qquad \textbf{2.15}$$

For elliptical collapse of soil, the factor C_d is expressed as

$$C_{de} = \frac{\pi}{4\beta}[1{+}\alpha - \left(\frac{D}{B_d}\right)^2] \qquad \textbf{2.16}$$

Although it is a good approach to calculate normal stress, this model predicts more stress than normal as it does not consider the shear transfer between the loosening soil and elliptical periphery. In the presence of ground water, the model may not provide accurate results as soil loosening is greatly affected by water. Another important consideration is soil density, and over consolidation ratio is overlooked in this model.

2.5.4. Thomson (1993)

Thomson (1993) suggested different equations for cohesionless and cohesive soil based on elliptical collapse theory of soil. For dense sand, the equation is

$$W_s = 0.75\pi \times \gamma_b \times D_s^2 \qquad \textbf{2.17}$$

Where

W_s = weight of soil acting on pipe (kN)

γ_b = bulk unit weight of soil (kN/m³)

D_s = diameter of shield (m)

For loose sand and over-consolidated clay, the equation is

$$W_s = \gamma_b \times H \times D_p \qquad \textbf{2.18}$$

Where

H = cover depth (m)

D_p = diameter of pipe (m)

Once full overburden depth for load calculation is considered, this theory becomes much too conservative for design

2.5.5. PJA (1995)

The Pipe Jacking Association (PJA) suggests that soil would collapse onto the pipe and exert circumferential stress. The normal stress on pipe is defined by equation 2.19.

$$\sigma_V = \frac{\gamma \times B}{K \times tan\phi}\left(1 - e^{-K \times tan\phi \times \frac{H}{B}}\right) \qquad \textbf{2.19}$$

σ_V = normal stress on pipe crown (kN/m^2)

γ = bulk unit weight of the soil (kN/m^3)

D = pipe diameter (m)

ϕ = soil internal friction angle (˚)

K = earth pressure coefficient (K= $\frac{(1-sin \varnothing)}{(1+sin \varnothing)}$)

B = silo width (B= $\frac{D}{2} tan\left(45°-\frac{\phi}{2}\right)+\frac{D}{2\,sin\left(45°+\frac{\phi}{2}\right)}$) (m)

PJA (1995) considers the angle of wall friction to be the same as the angle of internal soil friction by assuming perfect rough shear plane (Beaucour and Kastner 2002). They use Rankine's active earth pressure condition to calculate lateral earth pressure coefficient.

When a water table is present at depth H_1 from the surface, the equation for calculating normal stress becomes

$$\sigma_v{'}=\sigma_{v1}{'}e^{-K\times tan\phi\times {(H-H_1)}/{B}}+\frac{\gamma' B}{K\times tan\phi}(1-e^{-K\times tan\phi\times {(H-H_1)}/{B}}) \qquad \textbf{2.20}$$

Where

$$\sigma_{v1}{'}=\sigma_{v1}=\frac{\gamma\times B}{K\times tan\phi}(1-e^{-K\times tan\phi\times\frac{H_1}{B}}) \qquad \textbf{2.21}$$

$\sigma_v{'}$ = normal stress on pipe crown at presence of ground water (kN/m^2)

H = overburden depth (m)

H_1 = depth of water table from surface (m)

γ = bulk unit weight (above water table) (kN/m^3)

γ' = submerged unit weight (below water table) (kN/m^3)

In the case of cohesive soil, PJA suggests that the borehole will remain stable, which represents no normal stress on the crown (PJA 1995).

2.5.6. Bennett (1998)

Bennett (1998) proposed the normal stress (σ_n) on a pipe is a function of unit weight of soil and pipe diameter. The equation is

$$\sigma_n = C_a \times \gamma \times d_p \qquad \textbf{2.21}$$

Here, C_a is the arching factor. From case histories, it was ensured that C_a may vary in the range of 1/3 to 3. For most of the cases, the range was 1/2 to 1-1/2 (Bennett 1998). d_p is the pipe diameter (m).

Bennett recommended different values of C_a for different soil types and conditions (e.g. lubricated and non-lubricated). He suggested three types of values for arching factor (e.g. upper bound, best fit, lower bound), which are presented in Table 2.1.

Table 2.1 Values of arching factor recommended by Bennett (1998)

Soil type		Non-lubricated interval	Lubricated interval
Sands	Upper bound	1.5	1
	Best fit	1	0.66
	Lower bound	0.75	0.5
Stiff to hard clay	Upper bound	1	0.66
	Best fit	0.66	0.5
	Lower bound	0.33	0.5
Soft to medium clay	Upper bound	1	3
	Best fit	0.66	1.5
	Lower bound	0.5	1

2.5.7. ASCE (2000)

The American Society of Civil Engineers (ASCE) proposes a direct design method that uses factored load and helps calculate reinforcement of concrete pipes based on the surrounding soil envelope's reaction. The earth load on pipe is calculated by equation 2.21.

$$W_e=[VAF]\times w \times B_{Th} \times H \qquad \textbf{2.21}$$

$$[VAF]=\frac{1\text{-}2.718^{-\alpha}}{\alpha} \qquad \textbf{2.22}$$

$$\alpha=24K_{\mu}'\times\frac{H}{B_{Th}} \qquad \textbf{2.23}$$

W_e = earth load on pipe (kN/m)

VAF = vertical arching factor

B_{Th} = maximum span of tunnel bore for jacked pipe (m)

H = design height of earth above top of pipe (m)

w = unit weight of soil (kN/m^3)

The value of K_{μ}' is dependent on soil. It varies in the range of 0.192 for granular soil to 0.11 for saturated clay.

Equation 2.21 neglects the effect of cohesion, and presence of cohesion certainly reduces the amount of effective vertical load. In the case of reliable determination of c with proper site investigation, ASCE (2000) recommends equation 2.24 for reduced arching factor, which accounts the effect of cohesion.

$$[VAF_R]=(1\text{-}\frac{24c}{w\times B_{Th}})\times[VAF] \qquad \textbf{2.24}$$

c = cohesion (kPa)

VAF_R = reduced vertical arching factor

2.5.8. Staheli (2006)

Staheli (2006) developed an equation based on Terzaghi's trapdoor theory for cohesionless soil, but she replaced Terzaghi's (1943) silo width *B* with a modified *B** (Staheli 2006).

$$B^{*}=r\times cos(45°+\frac{\phi}{2})\ (\mathrm{m}) \qquad \textbf{2.25}$$

Replacing Terzaghi's (1943) *B* with *B** provides the following equation for calculating normal stress on buried pipe in cohesionless soil

$$\sigma_v=\sigma_{v\infty}=\frac{\gamma\times r\times cos\left(45°+\frac{\phi}{2}\right)}{tan\phi} \qquad \textbf{2.26}$$

Where

γ = the total unit weight of the soil (kN/m^3)

r = the radius of the pipe (m)

ϕ = soil internal friction angle (˚)

2.6 Research motivations

Application of drilling parameters-based indices for ground exploration has been used in vertical drilling. Several researchers have investigated the effectiveness of those indices using instrumented drilling in the laboratory as well as in the field, and they proposed that the indices have the potential to identify differences in soil layers. Drilling parameters-based indices can specify changes in soil strength by providing predefined numerical values or by changing its pattern. These indices have the potential to be cost effective soil investigation tools. However, this idea has not been implemented in the field for subsurface exploration during horizontal drilling. The motive of this study was to inquire about the practicability of the indices locating transitions of soil layers based on field data collected from a GBM project. The idea was to develop a feasible approach to describe soil strength in terms of indices' values.

The aforementioned models and standards for pipe jacking or microtunnelling can predict normal stress on pipe crown, but there are differences among those values that propel the necessity of a validation among them. Although different countries and regions use their own standards and models for normal stress calculation, there is no such study to show practitioners which method produces the best result in comparison with field-measured stress values. A comparison among the mentioned standards and models against field data can provide the best method of calculating vertical earth pressure. The motive of this study was to show such a comparison to validate the mentioned methods against field values. A parametric study was also conducted to find out the differences of a specific parameter (e.g. silo width) in different methods. It is the variation of assumed parameters by different models and standards that brings the differences in calculated normal stress values.

3. Chapter 3: Subsurface Profiling Using Horizontal Drilling Indices for Guided Boring Method

3.1. Introduction

Trenchless technique has recently become a popular method to install or rehabilitate pipelines with minimal surface disturbance (Najafi 2005). Guided Boring Method (GBM) is a widely used trenchless technique to install pipelines with grade precision. Guided pilot tubes are initially installed to maintain line and grade precisely in GBM, followed by upsizing and product pipes installation. This technology has grown from 0.10 m diameter to 1.22 m diameter pipeline installation with a drive length of 121.92 m in range (Boschert 2007). GBM has several advantages over traditional pipe jacking/microtunnelling, e.g., the initial phase of pipe jacking/microtunnelling is auger boring without pilot tube installation, which may lead to deviation from the original line and grade. GBM is a three-step installation procedure, as shown in Figure 3.1. The first step is to install pilot tubes on line and grade. The second step is to install auger casings, followed by removal of pilot tubes one by one. The third step is to install product pipes and remove auger casings (Boschert 2007).

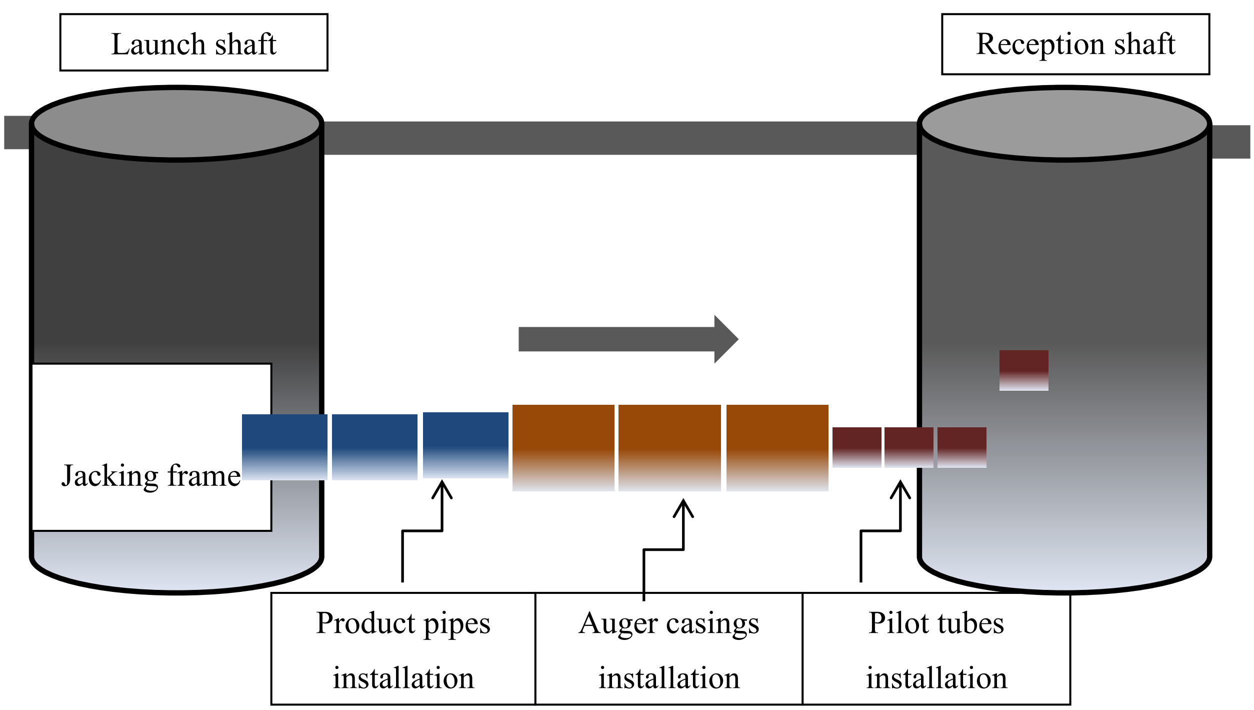


Figure 3.1 Three-step installation procedure of GBM

For the past few years, contractors have been optimizing drilling in addition to reducing drilling hazards and increasing drilling output in terms of time and cost. Consequently, a number of drilling techniques, tools and machines have been introduced in the drilling sector to achieve optimized drilling. A better knowledge of soil subsurface is essential to implement improved drilling methods. The construction of large projects (e.g. tunnels) requires an extensive investigation of ground to gain sufficient knowledge about soil. This investigation leads to drilling a large number of holes for different in-situ tests, e.g. Cone Penetration Test (CPT). However, as the length of these projects is much smaller than other underground projects, budget constrains can make geotechnical investigations for GBM unjustified at times. Therefore, trenchless contractors frequently take risks by performing GBM installations without proper subsurface information, which often leads to unusual circumstances. Sometimes the installation becomes immovable upon encountering an unusual soil layer; this costs the project even more than the cost of a geotechnical investigation.

The use of drilling parameters can be an effective alternative tool for subsurface exploration in drilling. Recording and analyzing drilling parameters for drilling optimization has been used for a long time in the oil and gas industry (Gui et al. 2002). Evidence based on theory as well as laboratory experiment shows that the compressive strength of drilled strata can be determined using the relationship between the observed values of drilling variables (Brown and Barr 1978). Laudanski et al. (2012) conducted an extensive experiment on a specially constructed test embankment to develop an empirical relationship between drilling parameters-based indices and test results from the Standard Penetration Test (SPT), as well as CPT. Real-time drilling surveillance software based on drilling parameters (e.g. ENPASOL) is being utilized to discover lithological transitions for vertical drilling. Trenchless technologies such as GBM or Pilot Tube Microtunnelling (PTMT) consist of an initial pilot tube installation with further borehole reaming. Inspecting pilot tube insertions can therefore gather ground information with CPT, whereby a hydraulic pushing system penetrates a cone rig through the soil. Hence, profiling the subsurface using drilling parameters can be an effective alternative tool of geotechnical investigations for GBM or PTMT projects. In these types of projects, profiling the subsurface during the pilot tube installation phase can help operators select proper drilling tools for the next reaming stages.

Using drilling data, different researchers have developed different indexes to create a relative soil profile, as shown in Table 3.1. The Somerton index (Somerton 1959) is a measurement of resistance to drilling, which is an effective tool for rock mass classification. The Somerton index was initially proposed as a strength parameter for rock to correlate rock properties with drilling parameters, and this tool proved to be very useful in differentiating weathered rock layers (da Fonseca and Coelho 2006). This index has no specific value range: a comparatively smaller value indicates softer soil, and a higher value indicates harder soil. Bingham (1965) proposed a Γ-hardness parameter that represents how hard it is to drill soil. A higher Γ-hardness value signifies the soil is hard to drill, which does not necessarily mean hard soil. Even clay may clog the drilling bits and increase the Γ-hardness value. The ease of removing drilled materials largely affects the measurement of soil hardness.

Mechanical Specific Energy (MSE) proposed by Teale (1965) is a widely used index for monitoring drilling efficiency. MSE is defined as the energy required for removing one unit volume of rock or soil. This index can be used as a tool for indirect assessment of soil strength. As the drilling progresses, abrupt variation of MSE represents inefficient drilling. This index has been used to increase performance and reduce time by adjusting drilling parameters in real time. Several factors may cause changes in MSE, e.g., bit balling, bottom hole balling, and vibrations (Bevilacqua et al. 2013). In most of the cases, change in soil formation causes change in MSE; MSE can thus be used as a diagnostic tool to identify different geological formations (Bevilacqua et al. 2013).

Alteration index was developed by Pfister (1985). This index provides a comparative strength measurement of soil using Weight On Bit (WOB) and penetration rate, but cannot specify the exact soil type based on a standard scale. This index, with values varying from 0 to 2, is very sensitive for low to medium strength soil; 0 is an indication of soft soil, while 2 is an indication of hard soil. Pfister (1985) also introduced the energy parameter used for drilling, which is different from MSE developed by Teale (1965) since this index excludes jacking force parameter

(Table 3.1). This energy parameter is very useful for hard soil and rock analysis. However, the omission of jacking force makes this index less effective than MSE to assess soil layers through a trenchless technique where the jacking force parameter is dominant. Celada et al. (2009) used MSE for rock mass characterization during a tunnel construction by measuring the drilling parameters of a Tunnel Boring Machine (TBM). A summary of different indices is presented in Table 3.1.

Table 3.1 Summary of indices

Index	Equation	Description of parameters
Somerton index (Somerton 1959)	$S_d = W \times (N/V)^{1/2}$	S_d = Somerton index W = weight on bit (thrust – retention force + weight of rods and bit) (kN) N = rotation speed (rps) V = instantaneous penetration rate (m/s)
Hardness parameter (Bingham 1965)	$\Gamma\text{-hardness} = NFD^2/VT$	N = rotation speed (rps) F = thrust applied on the drilling bit (kN) D = bit diameter (m) V = penetration rate (m/s) T = rotation torque (kN·m)
MSE (Teale 1965)	$MSE = F/A + 2\pi NT/AV$	MSE = Mechanical specific energy (kPa) F = thrust on bit (kN) A = area removed by drill bit (m^2) N = rotation speed (rps) T = rotation torque (kN·m) V = drilling speed (m/s)

Alteration index (Pfister 1985)	$AI = 1 + \left(W/W_{max} \right) - \left(V/V_{max} \right)$	AI = Alteration index W = weight on the bit (thrust – retention force + weights of rods and bit) (kN) W_{max} = theoretical maximum value of W (kN) V = instantaneous penetration rate (with maximum value V_{max}) (m/s)
Energy used for drilling (Pfister 1985)	$E = TN/V$	E = Energy used for drilling (kJ/m) T = value of the rotation torque (kN·m) N = rotation speed (rps) V = instantaneous penetration rate (m/s)

This paper examines a GBM project to install a sewer line and storm line with grade precision. Drilling parameters for this project were recorded using sensors during the pilot tube installation. The strength of soil throughout the drive length is investigated using the five indices in Table 3.1, incorporating recorded drilling parameters. The intent is to use drilling parameter analysis to identify lithological changes of the soil being drilled. The objective is (1) to form a geotechnical investigation tool that can provide a reliable pattern of comparative soil strength, and (2) to determine the most effective indices for subsurface profiling. This is a primary work which only explores relative strength measurement of soil, but provides opportunity for future researchers to define exact values of indices for particular strengths of soil.

3.2. Field Instrumentation

3.3. Project Overview

The GBM project was located at the intersection of 90th Avenue and 110th Street, Edmonton, Alberta, Canada. The project consisted of installing two HDPE pipelines, one for a sanitary line and the other one for a storm line, as shown in Figures 3.2 and 3.3. The pipe diameter for the sanitary line was 0.41 m with a casing diameter of 0.71 m, and the drive length was 39.93 m. The pipe diameter for the storm line was 0.61 m with a casing diameter of 0.91 m, and the drive length was 38.71 m (Figure 3.3). The distance between the two lines was 0.76 m (Figure 3.3), but there was no elevation difference.

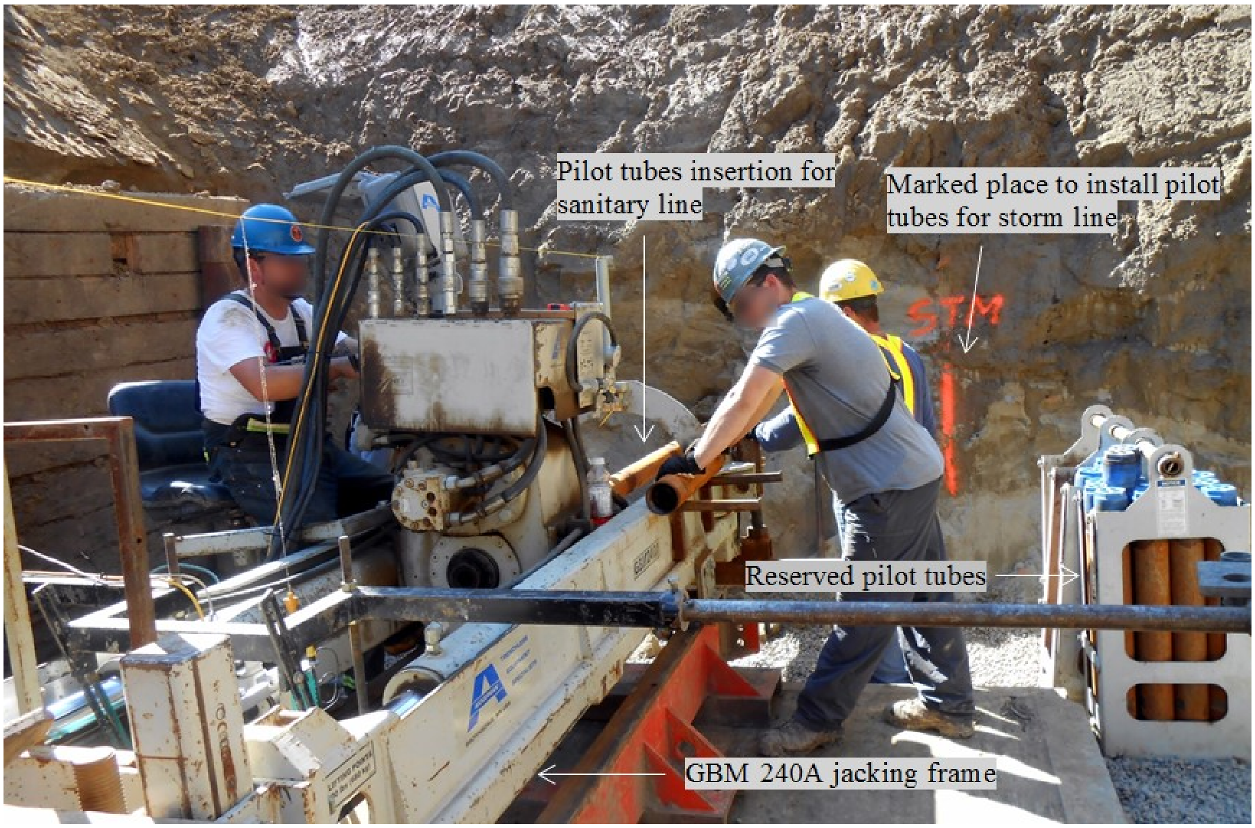


Figure 3.2 Pilot tubes installation for sanitary line

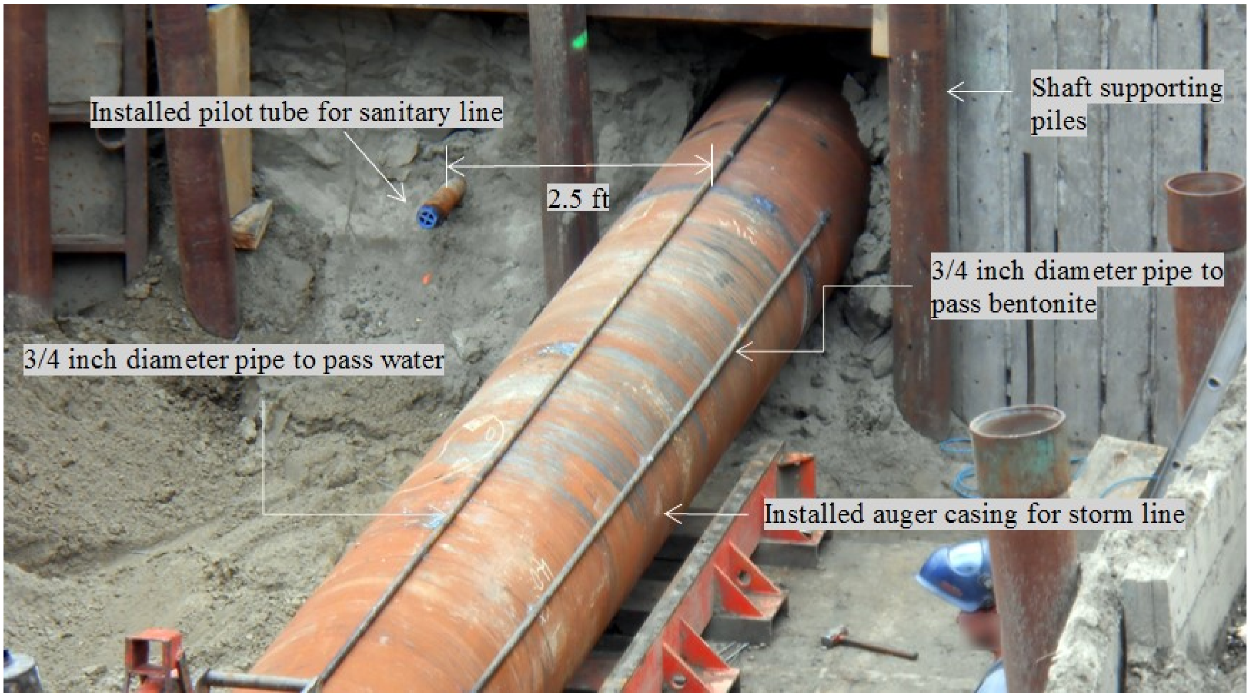
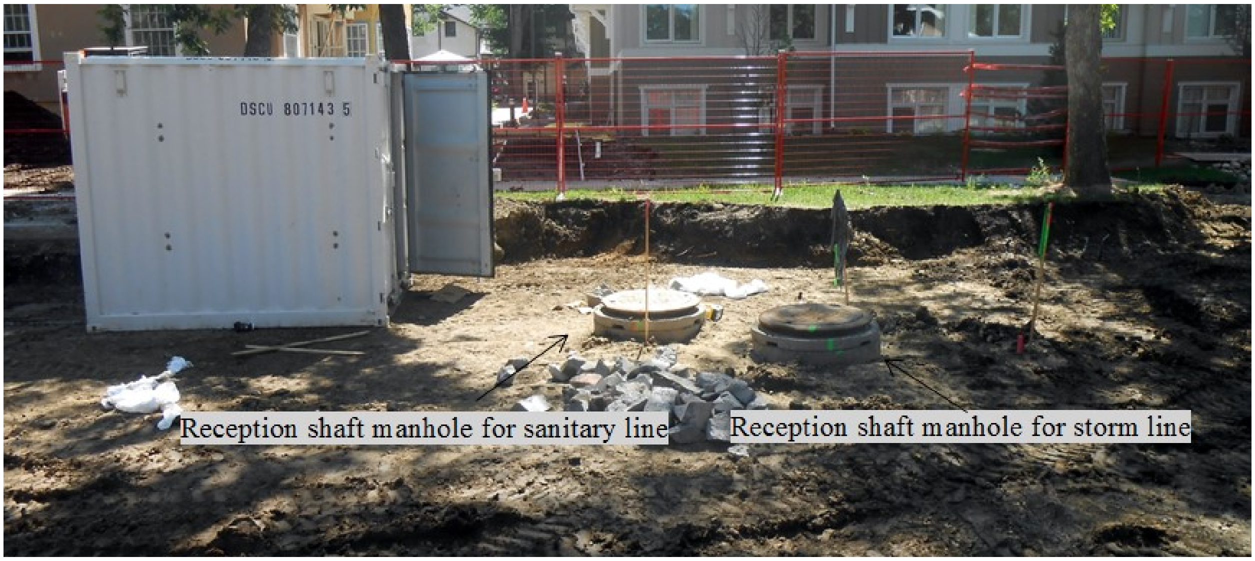


Figure 3.3 Auger casing installation for storm line

A 12.80 m × 3.66 m × 6.10 m sized launch shaft was prepared to drive the pilot tubes through the ground (Figure 3.3). A total of 14 piles were used to support the excavation of the launch shaft. The piles were 3.96 m long and penetrated 1.50 m into the soil (Figure 3.3). The excavated soil was sandy, but no geotechnical investigation was conducted to determine exact ground conditions. There was no presence of ground water. No disruption of traffic and no sound restriction had been implemented on the site. An Akkerman 240A jacking frame (Figure 3.2) was used to insert the pilot tubes into the ground and a P100Q power pack was used to provide the pressure. Later, a Detroit diesel auger machine was used to drive augers through the soil. The length of one auger casing was 6.10 m. Auger casings were welded to each other during installation. Two 0.02 m diameter small pipes were installed on top of the casings (Figure 3.3). One pipe was used to convey water and ease the drilling process, and the other pipe was used to convey bentonite and support the borehole. Two distinct manholes were used as reception shafts for the installations, as shown in Figure 3.4(a). In Figure 3.4(a), the left manhole was used for the sanitary line and the right manhole was used for the storm line. Only one person was able to enter the manhole to facilitate the pilot tube removal (Figure 3.4(b)).



(a)

(b)

Figure 3.4 (a) Manholes as reception shafts and (b) pilot tubes removal from manhole

The product pipes installation in this project differed from the conventional GBM principle. According to GBM principle, installation of product pipes and removal of auger casings occur simultaneously. However, at the mid-length of the storm line project, the soil to the left side of the auger casing appeared softer than the rest of the length, thus pulling the auger casings slightly to the left. The alignment was later maintained by removing and reinstalling the auger casings. To prevent product pipes deviating from their courses in the future, the auger casings were kept inside the soil, as shown in Figure 3.5.

Figure 3.5 Product pipes installation

3.4. Instrumentation and Data Collection

Two hydraulic pressure transducers were used to record the jacking pressure and rotational pressure data of the jacking frame. Both of the transducers were installed on a P100Q power pack. A CR 800 datalogger was used to measure the data (Figure 3.6). The datalogger scanned data at 80-ms intervals and recorded data at 2-s intervals. Within a 2-s interval, the datalogger recorded the maximum, minimum and average values of jacking and rotational pressures. A laptop was initially connected to the datalogger to verify that the datalogger was recording data properly. Later, a SC 115 CS I/O 2G flash memory was connected to the datalogger to store data. The CRBASIC editor software was later used to extract the data from the flash drive and input it into an Excel sheet. Data was converted from rotational pressure to torque and from jacking pressure to jacking force for analysis. The data showed a clear pattern for each pilot tube installation, including preparing time and jacking time.

Figure 3.6 Continuous measurement of data using Datalogger

3.5. Result Analysis

The five commonly used indices, as shown in Table 3.1, interpreted the field data for this GBM project. Pressure transducers recorded the jacking force and torque during drilling. The length of each pilot tube was 0.76 m, and recorded data provided the length of time needed to penetrate each pilot tube. The average rate of penetration for each pilot tube was calculated from previous pilot tube length and time information. Revolution Per Minute (RPM) was not recorded during drilling. An average value of 37.5 RPM was used to calculate indices, since 25–50 was the range of RPM value used for the GBM 240A jacking frame. The recorded data is plotted in Figure 3.7 to show the variation of each drilling parameter except RPM throughout the drive length. Jacking pressure remains almost constant, but the rotational pressure varies because the diameter of the pilot tube was 0.10 m, which was easier to penetrate through the soil. Since maintaining proper alignment was more crucial than jacking during the drilling process, the rotational pressure became dominant over jacking pressure. Figure 3.7 shows that where the rotational pressure was higher, the rate of penetration was lower and vice versa. Soil was not the same strength throughout the length, and comparatively hard soil increased the rotational pressure value and decreased the rate of penetration.

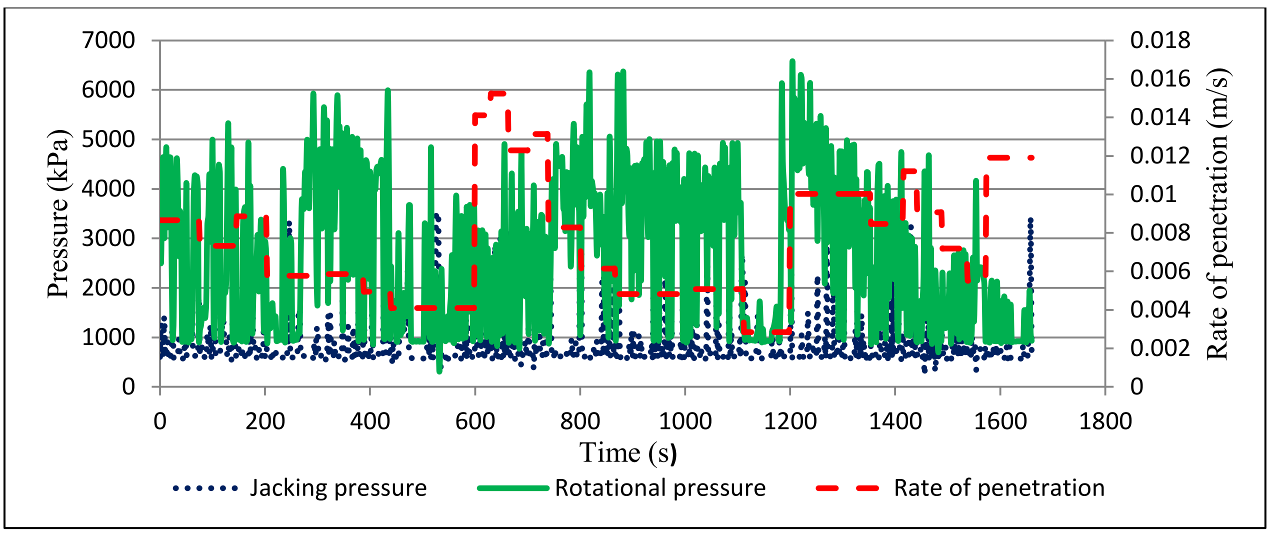


Figure 3.7 Recorded jacking pressure, rotational pressure, and rate of penetration

Drilling parameters based on Alteration index are plotted in Figure 3.8. The value of Alteration index is controlled by jacking force and rate of penetration (Table 3.1). The jacking force remained almost constant in the recorded data, so the rate of penetration became the only controlling parameter for the Alteration index in this project. This may account for the similar pattern between Alteration index and rate of penetration. The highest average rate of penetration of a pilot tube was considered the maximum rate of penetration in calculating Alteration index. From Figure 3.8, the drive length can be divided into three sections of approximately equal time slots. The first two slots show a nearly similar increasing pattern in Alteration index, which indicates comparatively lower strength soil to higher strength soil. But in the last time slot, the index value remains almost constant, which indicates the strength of soil is almost constant.

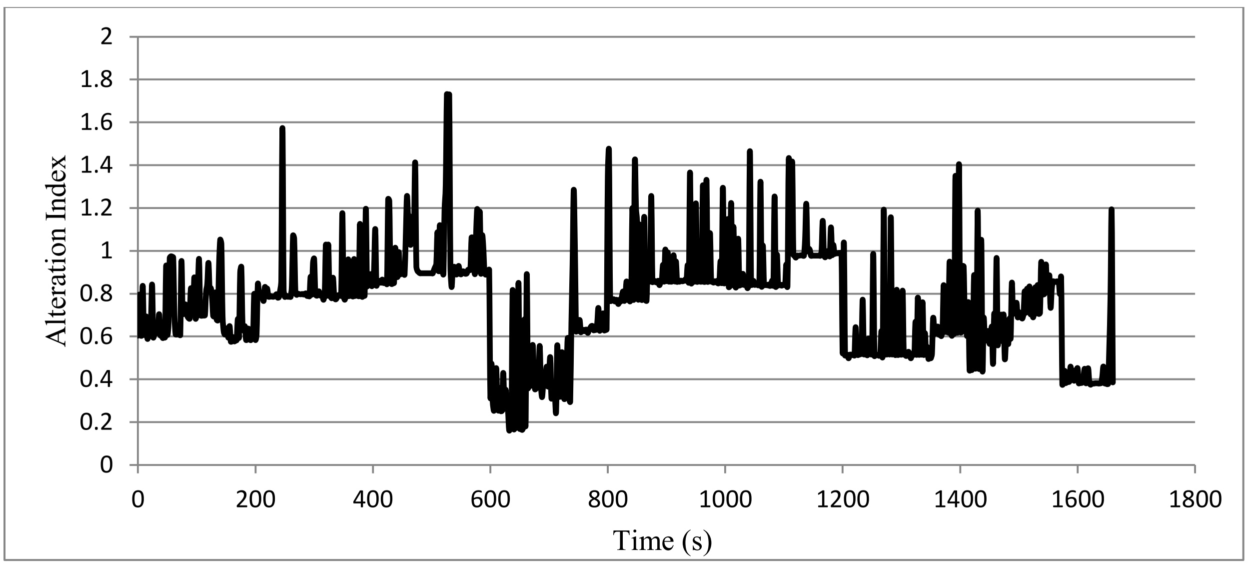


Figure 3.8 Change of Alteration index vs. time

The lower boundary of the Alteration index, 0, represents soft soil, and the higher boundary, 2, represents hard soil. If the penetration reaches an immovable condition, then the index value becomes 2 and the WOB reaches its maximum value. Conversely, if a jacking force of almost 0 creates maximum rate of penetration, then the index value becomes 0. For soil index, a value of 0 is almost impossible. An index value close to 0 represents substantially soft soil. In Figure 3.8, the index value reaches approximately 0.20 at one stage, indicating the existence of soft soil layers. On average, the index value remains close to 1 throughout the drive length, exhibiting soil of medium strength. The highest index value remains around 1.4. In conclusion, the soil was of comparatively low strength throughout the project length.

Γ-hardness parameter is controlled by four drilling parameters and bit diameter. Therefore, the result of Γ-hardness is more reliable than Alteration index as it includes rotational pressure, jacking pressure, and rate of penetration. Figure 3.9 presents the calculated Γ-hardness parameter for this project. Since the fluctuation of Γ-hardness parameter is not so intense, Figure 3.9 does not show as clear a pattern of soil formation change as the Alteration index in Figure 3.8. Regardless, the last one-third segment of Figure 3.9 shows no variable peaks compared with the rest of the drive length, which means the soil type for the last one-third segment was of consistent strength. As discussed earlier in the Result Analysis, Alteration index also reveals similar results in which the soil type for the last one-third segment is consistent.

Drillability is the opposite of hardness (Solberg 2012). There is no specific range of maximum and minimum Γ-hardness values, but a higher Γ-hardness value tends to slow the drilling process. The only thing that can be detected from a Γ-hardness profile is the variation of index value, which indirectly implies a change in soil strength. Figure 3.9 undoubtedly shows two segments of a higher Γ-hardness value, suggesting the soil was comparatively hard.

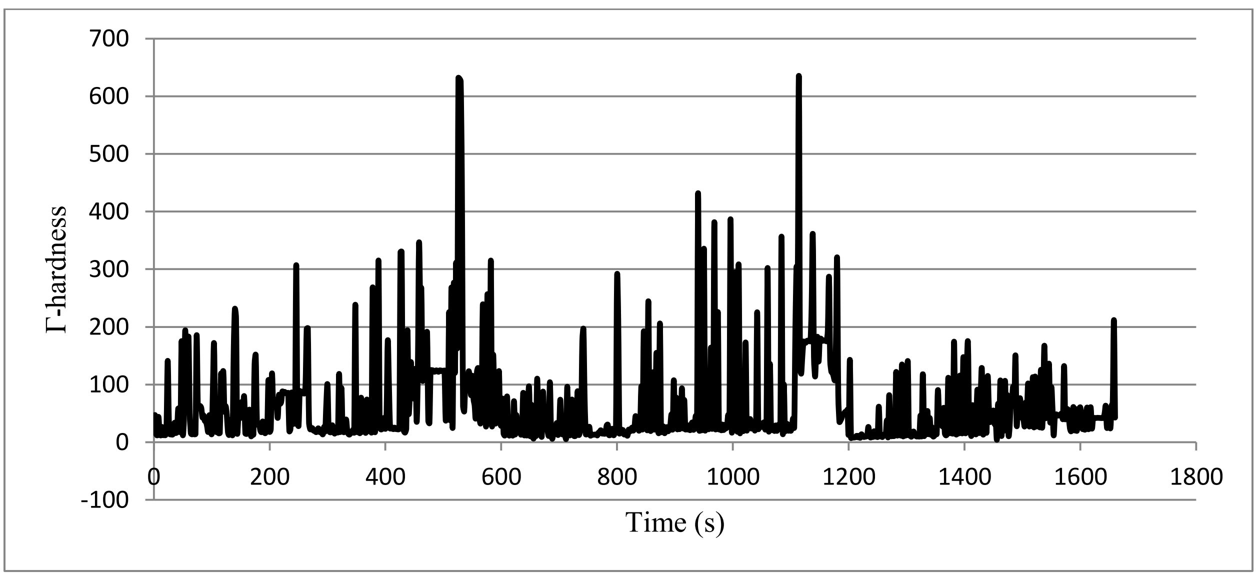


Figure 3.9 Change of Γ-Hardness parameter vs. time

To determine how effective this tool can be in differentiating soil layers, Figure 3.10 shows the calculated Somerton index for this GBM project. This index has three drilling parameters, excluding torque. In the recorded data, jacking force remained almost constant; the RPM value was also considered constant. As a result, the rate of penetration became the driving parameter in calculating the Somerton index in this GBM project.

The drive length of Figure 3.10 can also be divided into three equal time slots based on index value patterns. The first segment of the drive length shows several peaks, but the base line index value exhibits a slightly increasing trend in index value. The trend for the middle segment is similar to the first segment, and in the case of the last drive length segment, the base line index value remains almost constant in spite of having few peaks. Overall, the minimum index value line represents a clearly visible pattern showing transition in soil strength. This index value has no specific range to judge the exact strength of soil, so it is only helpful in determining the transition of soil layers.

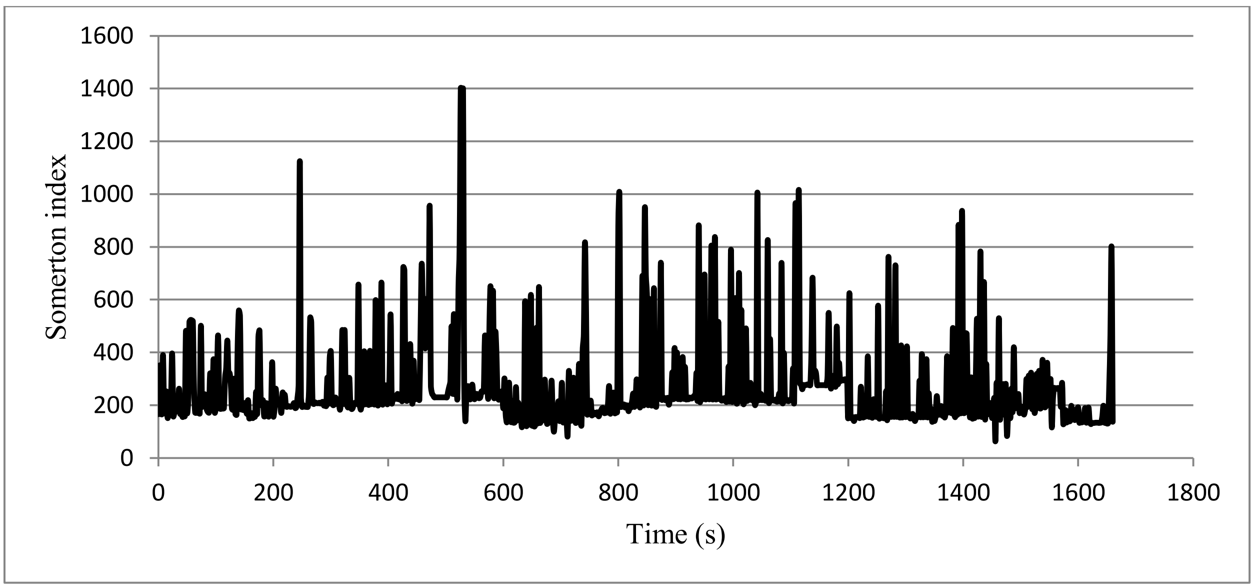


Figure 3.10 Change of Somerton index vs. time

MSE has been used as an indicator of efficiency in vertical drilling for a long time. MSE for this GBM project is plotted in Figure 3.11. Several factors can create a variation of MSE. For instance, the appearance of hard soil layers causes vibrations of the drill bit and leads to an increase in MSE (Teale 1965). Additionally, the appearance of clay layers creates bit balling so the bits become less effective at drilling, causing an increase in MSE (Teale 1965). In Figure 3.11, the MSE increased as the drilling advanced up to one third of the total drive length time. The index value then dropped, which was an explicit indication of new soil layer emergence. For the last one-third time segment in Figure 3.11, the MSE remained almost constant, representing a homogenous soil layer. This evaluation of soil strength by the MSE tool throughout the drive length matches the results of the Alteration index, Somerton index and Γ-hardness parameter. Since there is no specific MSE value to correlate with a soil of specific strength, an extensive laboratory test of drilling on various types of soil is required to correlate soil strength with MSE. Still, as an initial study, MSE can be used to identify drilling efficiency involved in different soil layers.

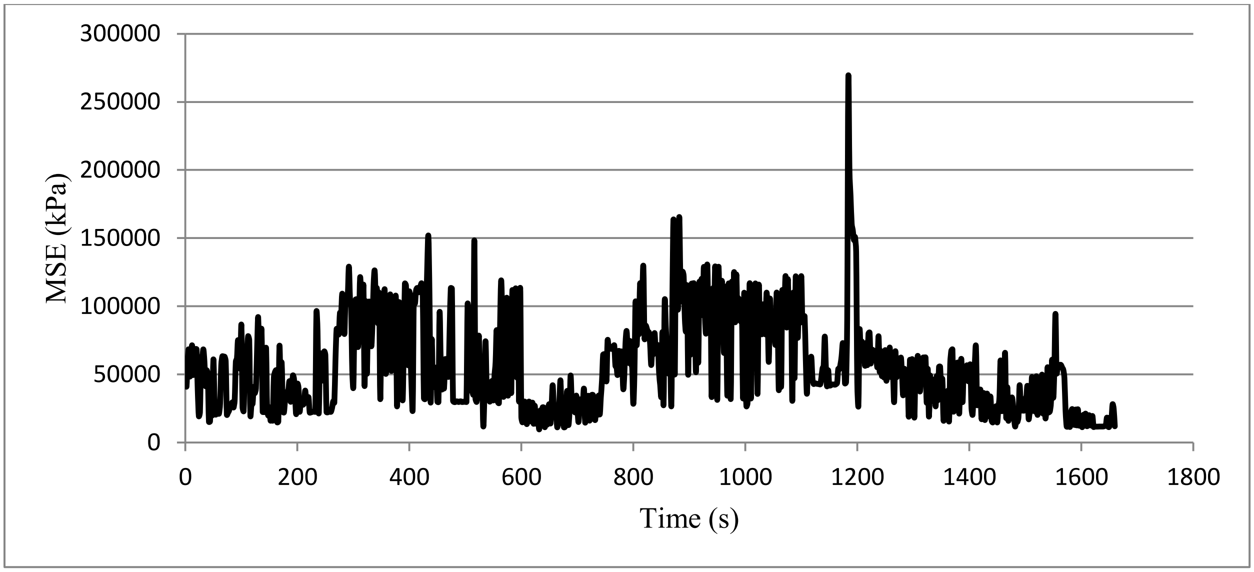


Figure 3.11 Change of MSE vs time

Energy used for drilling, proposed by Pfister (1985), is plotted in Figure 3.12 for this GBM project. Because of the nearly constant jacking force value of the project, the change of index value along the drive length is nearly identical to MSE. This is evident since the absence of jacking force is the only difference of energy from MSE used for drilling. The fluctuations of this drilling energy index in Figure 3.12 show the index value increased gradually, then dropped, but remained stable for the last part of the drive length. An unchanged drilling energy clearly refers to a consistent soil layer. A gradual increase in energy indicates drilling from soft soil to hard soil. Although further research is needed to correlate a specific energy value with a specific type of soil, this index profile gives a direct measurement of energy for drilling. The variations of energy value in Figure 3.12 signifies a heterogeneous soil profile. The drilling operator can therefore judge the soil strength onsite in real time by evaluating the amount of energy required for drilling.

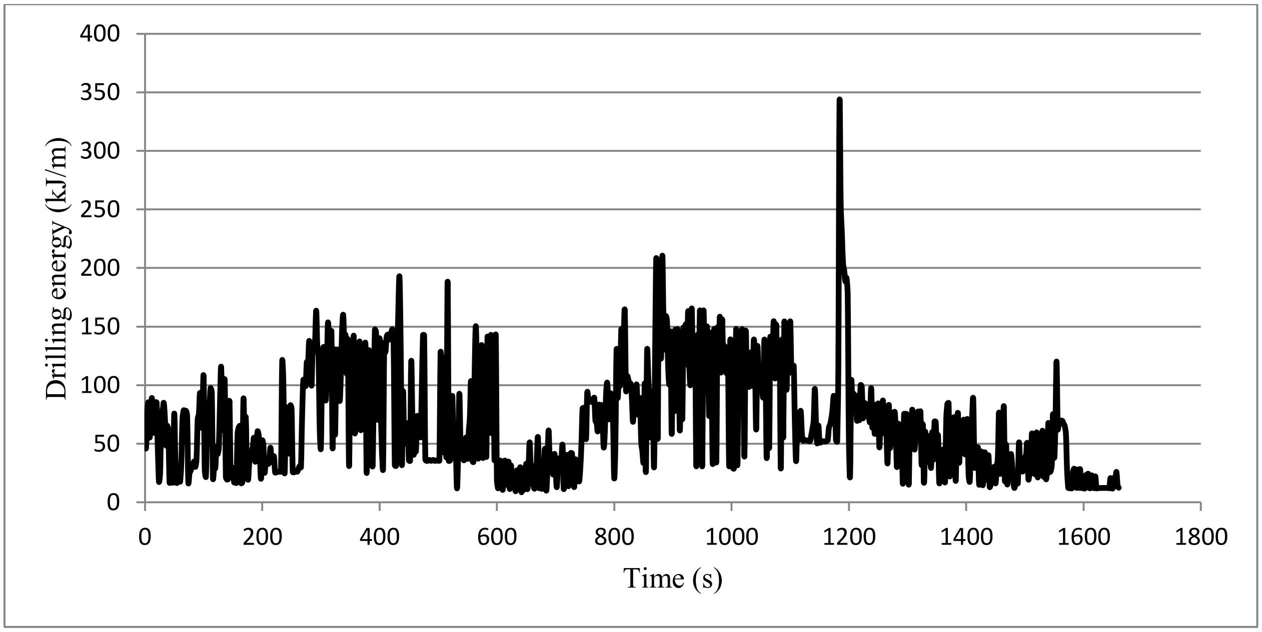


Figure 3.12 Energy required for drilling defined by Pfister (1985) vs time

Knowing geotechnical information plays an important role in selecting the proper drill bits or Powered Cutter Head (PCH) for the reaming stages of GBM projects. Hard soil layers resist penetration of drill bits, which results in the application of more force, and excessive weight on drill bits may cause buckling of the drill stem and wear of bits. Too little knowledge about the soil profile may lead to improper selection of drill bits, resulting in a low rate of penetration. The energy requirement for auger penetration is influenced by strength of soil. In this GBM project, the operators assumed the soil was hard because it was taking more time to penetrate one pilot tube through the ground relative to their experience. As a result, they selected a drill bit for auger casings installation which was compatible for hard soil, but this bit selection could have been more accurate with the knowledge of a soil profile. Regardless of experience, all trenchless personnel associated with drilling ahead of reaming can benefit from using indices for subsurface profiling. A proper drill bit selection accelerates the rate of penetration, but on the other hand, improper bit selection can lead the project to an immovable condition. Soil information also helps select the jacking frame and increase the overall project speed. This knowledge reduces the probability of a project stopping due to the appearance of unusual soil layers and increases the confidence level in the entire drilling process.

3.6. Conclusion

In this paper, subsurface profiling using horizontal drilling parameters during pilot tube installation has been implemented in a GBM project that had no geotechnical information. All the five indices could identify the soil transitions in the drive length. The soil strength profile was more transparent using the Alteration index, MSE and energy used for drilling than using the Somerton index and Γ-hardness parameter. Still, any of the discussed indices could provide practitioners an explicit knowledge of ground conditions without any traditional geotechnical investigation. The limitation of the study is that the indices only give a comparative strength measurement of soil throughout the drive length but cannot give the exact strength of soil. Further studies are required to define the limit of each index to identify a soil layer of specific strength. The preliminary study of indices for horizontal soil drilling in this paper can enhance the specific contributions of drilling parameters over conventional geotechnical investigations in the future.

4. Chapter 4: Comparison of different methods for normal stress calculation during pipe jacking/microtunnelling

4.1. Introduction

Serving as a foundation for a number of trenchless technologies, pipe jacking has added a new dimension to the pipe laying industry. The method entails pushing a pipe through the borehole formed by cutting head, shield or even hand. It is also used for microtunnelling that is usually distinguished by the size of pipe diameter (Thomson 1993). Pipe jacking is used to describe installations of pipes in diameter greater than 900 mm; microtunnelling refers to pipe installations up to 900 mm (PJA 1995). Pipe jacking as the minimal surface disruption pipe installation method was first developed in the USA in 1890 (Drennon 1979). Starting from the 1950s, this method was used for short length installations in densely populated urban areas of the UK (Yonan 1993). Within a short period of time; it became popular for pipeline installations not only in UK but also in Japan and West Germany. These countries performed major pipe jacking installations after 1960s and became one of the leading contributors to this research sector (Yonan 1993).

Determination of vertical soil load on jacking pipe is important for pipeline stability and friction estimation. Earth pressure calculation procedure on a pipe in trench was first published by Marston (1930). Later Terzaghi (1943) developed a theory for determining soil load on pipe crown based on the trap door experiment. However, these theories do not consider stress typical for jacking pipe to sufficient extent as the vertical earth load on jacked pipes is different from vertical earth load on buried underground structure from backfilling (Stein et al. 1989).

From 1980 to 2000, a number of pipe jacking research projects had been conducted to investigate the influence of different factors on jacking force, e.g. normal stress. An extensive research project regarding pipe jacking was conducted at the University of Oxford by several researchers. Laboratory experiment had been carried out to investigate the effectiveness of sensors during pipe jacking, which was funded by Pipe Jacking Association (PJA) and Science and Engineering Research Council (Milligan and Norris 1994). Norris (1992) extended the research to the site and examined the load transfer mechanism; he proposed that the normal stress on a pipe crown was directly proportional to the weight of pipe. Marshall (1998) conducted a similar research project as Norris (1992) placing more sensors on the instrumented pipe. Bennett (1998) considered the effect of arching and proposed an equation where the normal stress on pipe crown was a function of soil density and pipe diameter, and recommended taking into account different arching factor values for different soil types. American Society of Civil Engineers (ASCE 2000) published detailed specifications for designing precast concrete jacking pipes, suggesting two different equations for cohesive and non-cohesive soils, which both account for the arching effect of soil. Staheli (2006) proposed an effective soil depth of loading based on observations of shear plane failure and the Mohr-Coulomb failure criteria.

Although several procedures are currently in practice to determine normal stress during pipe jacking, they are not universally applicable (Stein et al. 1989). Thus, a comparison among different models/standards is necessary to shed further light into this research area. This paper presents a review of different methods for calculating vertical earth load on jacking pipes. After that, the comparison is shown among those based on field data from Norris (1992) and Marshall (1998). Lastly, a parametric study is also presented. This study reveals the crucial differences among different normal stress calculation methods which will assist in designing jacking pipes and further examining normal stress behaviour.

4.2. Summary of different standards and models

Nine commonly used models/standards have been selected for discussion as these are presumed as the most popular methods. Most of them are modified from the Terzaghi's (1943) trap door experiment. Figure 4.1 shows a typical cross section of a buried pipe introducing different parameters. A summary of different parameters for selected models/standards is presented in Table 4.1.

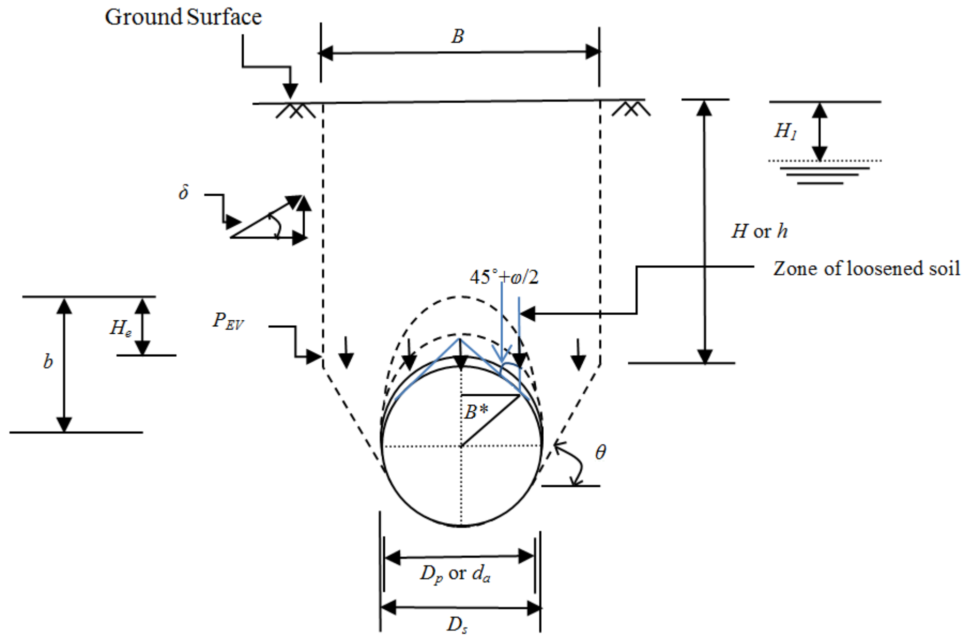


Figure 4.1 Typical cross section of pipe during pipe jacking

Both American Concrete Pipe Association (ACPA) and Concrete Pipe Association Australasia (CPAA) suggested the same formula to calculate earth pressure on jacked pipes using the same concept of the trench method where the silo width is same as borehole diameter (ACPA 1987; CPAA 1990). This is an indirect design method similar to the Marston-Spangler trench installation method (Spangler 1960).

Abwasser Technische Vereinigung (ATV 1990) specifications were developed to form directives for stress and strain analysis of jacking pipes. ATV (1990) assumes the angle of wall friction is equal to half of the angle of soil internal friction to remain conservative with the design perspective since higher angle of wall friction leads to higher shear displacement and less pressure on pipes due to arching. Modern pipe jacking equipment causes minimal disturbance of overlaying soil which is barely influenced during pipe jacking (Stein et al. 1989). ATV (1990) considers the lateral earth pressure coefficient value K=0.5 that represents static soil condition (Stein et al. 1989) and also disregards cohesion of soil, although cohesion certainly reduces the vertical stress on the pipe. For jacking below the ground water table, ATV (1990) recommends to consider full soil load (K=1.0) to remain on the safe side.

O'Rourke et al. (1991) described soil load transfer mechanism on auger-bored pipes based on the assumption that load was a function of soil height up to which collapse occurred above the pipe crown. He introduced the term "bulking factor" in loosened soil for calculating normal stress. Equations are developed based on elliptical collapse or loosening of soil over pipe crown. For a smaller value of bulking factor, the loosening soil depth may extend up to 3 and 4 times of bore diameter (O'Rourke et al. 1991).

Thomson (1993) suggested different equations for non-cohesive and cohesive soils based on the elliptical collapse theory of soil. For cohesive soil, this method considers full overburden depth of soil as effective vertical load contributing to soil height. Considering full overburden depth for load calculation makes this theory too conservative for design, since this assumption ignores arching phenomenon and considers full overburden pressure.

Pipe Jacking Association (PJA) proposes that the soil would collapse onto the pipe and exert circumferential stress for non-cohesive soil. PJA (1995) considers the angle of wall friction the same as the angle of internal soil friction by assuming perfect rough shear plane (Beaucour and Kastner 2002). The researchers used Rankine's active earth pressure condition to calculate lateral earth pressure coefficient. In the case of cohesive soil, PJA (1995) recommends stable bore theory which does not represent normal stress values on the crown.

Bennett (1998) proposed that the normal stress on a pipe was a function of soil unit weight and pipe diameter, and recommended different values of arching factor for different soil types and conditions (e.g. lubricated and non-lubricated) based on field observations, which is more practical.

ASCE (2000) is the direct design method that uses factored load and calculates reinforcement of concrete pipes based on surrounding soil envelope's reaction. This standard provides different equations for non-cohesive and cohesive soils. In case of reliable determination of c with proper site investigation, ASCE (2000) recommends a distinct equation for reduced arching factor which accounts for the effect of cohesion.

Staheli (2006) developed an equation based on Terzaghi's (1943) trapdoor theory for non-cohesive soil. However, Terzaghi's (1943) silo was replaced by width B with a modified B^* based on cavity collapse model and Mohr-Coloumb failure theory (Staheli 2006). The new B^* is much smaller than the silo width recommended by Terzaghi. However, Staheli (2006) did not provide a formula for determining normal stress values for cohesive soil.

The above listed methods are not applicable to all types of soils and conditions. ACPA (1987) and CPAA (1990) recommend separate equations for cohesive and non-cohesive soils. ATV (1990) suggests only one equation which is applicable to both cohesive and non-cohesive soils; however, it disregards the effect of cohesion on arching. This standard also considers full overburden depth in presence of ground water. O'Rourke et al. (1991) introduced a new concept of a bulking factor overlooking the importance of soil density or over consolidation ratio. Although Thomson (1993) presents separate equations for cohesive and non-cohesive soils, this theory assumes full effective overburden depth for cohesive soil. PJA (1995) is only applicable to non-cohesive soil. In presence of ground water, PJA (1995) provides a modified equation. Bennett (1998) introduced specific arching values for specific soil type based on field observation. ASCE (2000) gives distinct equations for distinct soil types. On the other hand, Staheli (2006) model is only applicable to non-cohesive soil. To provide a more detailed overview, a comparison of the nine standards and models based on experimental data is presented in the next section.

Table 4.1 Basic parameters of arching factor from nine models/standards

Models/standards	Equations for normal stress calculation	Coefficient of earth pressure (K)	Wall friction angle (δ)	Silo width (b)	Explanation of terms
ACPA (1987) & CPAA (1990)	Cohesive and non-cohesive soil: $W_t=C_t\times\omega\times B_t^2\text{-}2c\times C\times_t B_t$	N/A	N/A	$B=B_t$	B, $\boldsymbol{B^*}$ = silo width d_a, D, d_p = pipe diameter Φ = soil internal friction angle r = pipe radius D_s= shield diameter $\boldsymbol{B_t}$, B_d = bore diameter
ATV (1990)	Cohesive and non-cohesive soil: $P_{EV}=\dfrac{1\text{-}e^{-2\times K_1\times\tan(\phi/2)\times\frac{h}{b}}}{2\times K\times\tan(\phi/2)\times\frac{h}{b}}\times\gamma_B\times h$	0.5	$\dfrac{\phi}{2}$	$B=d_a\sqrt{3}$	
O'Rourke et al. (1991)	Cohesive and non-cohesive soil: $W_e=\gamma\times B_d^2\times C_d$	N/A	N/A	$B=B_d$	
Thomson (1993)	Cohesive soil: $W_s=\gamma_b\times H\times D_p$ Non-cohesive soil: $W_s=0.75\pi\times\gamma_b\times D_s^2$	N/A	N/A	$B=D_s$	
PJA (1995)	Non-cohesive soil: $\sigma_v=\dfrac{\gamma\times B}{K\times tan\phi}(1\text{-}e^{-K\times tan\phi\times\frac{H}{B}})$ In presence of ground water: $\sigma_v'=\sigma_{v1}'e^{-K\times tan\phi\times(H\text{-}H_1)/B}+\dfrac{\gamma' B}{K\times tan\phi}(1\text{-}e^{-K\times tan\phi\times(H\text{-}H_1)/B})$ where $\sigma_{v1}'=\sigma_v v_1=\dfrac{\gamma\times B}{K\times tan\varnothing}(1\text{-}e^{-K\times tan\varnothing\times\frac{H_1}{B}})$	$\dfrac{1\text{-}\sin\phi}{1+\sin\phi}$	ϕ	$B/2=\dfrac{D}{2}tan(\mathbf{45^{\circ}}-\dfrac{\boldsymbol{\phi}}{\mathbf{2}})+\dfrac{D}{2\,sin(\mathbf{45^{\circ}}+\frac{\boldsymbol{\phi}}{\mathbf{2}})}$	
Bennett (1998)	Cohesive and non-cohesive soil: $\sigma_n=C_a.\gamma.d_p$	N/A	N/A	$B=d_p$	
ASCE (2000)	Cohesive soil: $W_e=(1\text{-}\frac{24c}{w\times B_{th}})\times\frac{1\text{-}2.718^{-\alpha}}{\alpha}\times w\times\frac{B_{Th}\times H}{12}$ Non-cohesive soil: $W_e=\dfrac{1\text{-}2.718^{-\alpha}}{\alpha}\times w\times\dfrac{B_{Th}\times H}{12}$	N/A	N/A	$B=B_t$	
Staheli (2006)	Non-cohesive soil: $\sigma_v=\sigma_{v\infty}=\dfrac{\gamma\times r\times cos(45^{\circ}+\frac{\phi}{2})}{tan\phi}$	1	$\varnothing$	$2B^*=2r\times cos(45^{\circ}+\phi/2)$	

4.3. Comparison of different methods based on experimental data

4.3.1. Description of experiments

Norris (1992) monitored pipe jacking force at Honor Oak, Lancashire, UK. Three instrumented pipe sections were used for field monitoring: one of them was used as a lead pipe to measure the ground convergence of cohesive soil; another instrumented section was set at a jacking pit. There was flexibility to place the third instrumented pipe section in between lead pipe and jacking pit based on site conditions to extract quality data. Norris (1992) placed the instrumented pipe at different places for four different schemes. Four Cambridge Earth Pressure Cells were placed at the middle of the instrumented pipe to measure radial total stress and shear stress acting on a specific place on pipe surface. The used pressure cells were machined in computer numerical controlled milling machine using aluminium alloy. Three independent strain gauge circuits were connected to the cell where two of them were to sense the total radial stress and another one to sense the shear stress. The cells were bolted and flushed to the main pipe. Caps were used to cover the cells to prevent direct contact with soil. Seals were also used to prevent ground water from penetrating into the instrument. Soil loads were transferred to the data acquisition system via earth pressure cells. Four pore pressure probes were placed near the contact pressure cells to measure the pore water pressure that was later used to determine the effective radial stress. Data was collected remotely using a modular data acquisition system.

Marshall (1998) conducted similar field tests at Leyton, East London, UK. Though the instrumentation procedure remained the same as done by Norris (1992), Marshall (1998) used twelve contact pressure cells instead of only four. Measured total radial stress value from the sensor placed at the top of instrumented pipe was used in this paper for comparison of methods in Table 4.1. Experimental data of scheme 4 collected by Norris (1992) was used in this section for comparison of different methods in non-cohesive soil. In the case of cohesive soil, the data collected by Norris (1992) and Marshall (1998) as presented Schemes 3 and 6 were used.

Soil type for Scheme 4 (Norris 1992) was dense silty sand which was used here as non-cohesive soil for comparison of different methods of Table 4.1. The instrumented pipe was placed very nearly the middle of a 158-m drive length train pipe. The details of Scheme 4 are shown in Table 4.2.

Table 4.2 Parameters of Scheme 4 in Norris (1992)

Parameters	Value
Diameter of pipe, D (m)	1.80
Depth of cover, H (m)	7-10
Unit weight of soil, γ (kN/m^3)	18.0
Friction angle, ϕ (degree)	32

Soil friction angle was assumed to be 32° (Bolton 1979). Cohesion of the soil was assumed to equal zero for calculation purposes. The peak radial stresses obtained at different places throughout the drive length in Scheme 4 were explained as the effect of misalignment and collapse of silty sand on pipe (Milligan and Norris 1999). There was an abrupt peak of radial stress near 60 m distance because of facing boulder clay at that point. Lubrication did not have a significant effect on normal stress values in this scheme.

In Scheme 3, pipes were jacked through highly plastic clay. The result obtained from sensor positioned on the pipe crown was different from the expected one. The soil properties in Scheme 3 are summarised in Table 4.3.

Table 4.3 Parameters of Scheme 3 in Norris (1992)

Parameters	Value
Diameter of pipe, D (m)	2.28
Depth of cover, H (m)	11-21
Unit weight of soil, γ (kN/m^3)	20.0
Friction angle, ϕ (degree)	31
Cohesion, c (kPa)	50.0

In the case of clayey soil having larger cohesion value, the borehole remains stable for most of the installations. However, the soil was highly plastic in this particular scheme creating higher stress values over the pipe crown.

Soil type for the test conducted by Marshall (1998) was also the same as in Scheme 3 (Norris 1992). All the sensors showed similar patterns in detecting normal stress values. The borehole remained stable throughout the majority of drive length, though irregularities in its shape caused few peaks in radial stress values. The details of soil parameters of Scheme 6 (Marshall 1998) are shown in Table 4.4.

Table 4.4 Parameters of scheme 6 in Marshall (1998)

Parameters	Value
Diameter of pipe, D (m)	1.80
Depth of cover, H (m)	6-8
Unit weight of soil, γ (kN/m^3)	19.4
Friction angle, ϕ (degree)	20
Cohesion, c (kPa)	15.3

Soil internal friction angle is assumed as 20˚ and cohesion of the soil is assumed as 15.3 kPa (Skempton 1964). The depth of water table was 3.5 m at the site.

4.3.2. Results for non-cohesive soil

Using the soil properties in Table 4.2 provided by Norris (1992) and the equations presented in Table 4.1, the normal stress values on pipe were calculated. Figure 4.2 shows the calculated normal stress value by different methods as well as the measured values. The depth of cover was not fixed throughout the drive length. So, the normal stresses were calculated using minimum and maximum value of the depth of cover range. Later error was calculated using mean and standard deviation of the data and an error bar was put on every normal stress bar in Figures 4.2, 4.3 and 4.4 to show the variability of data.

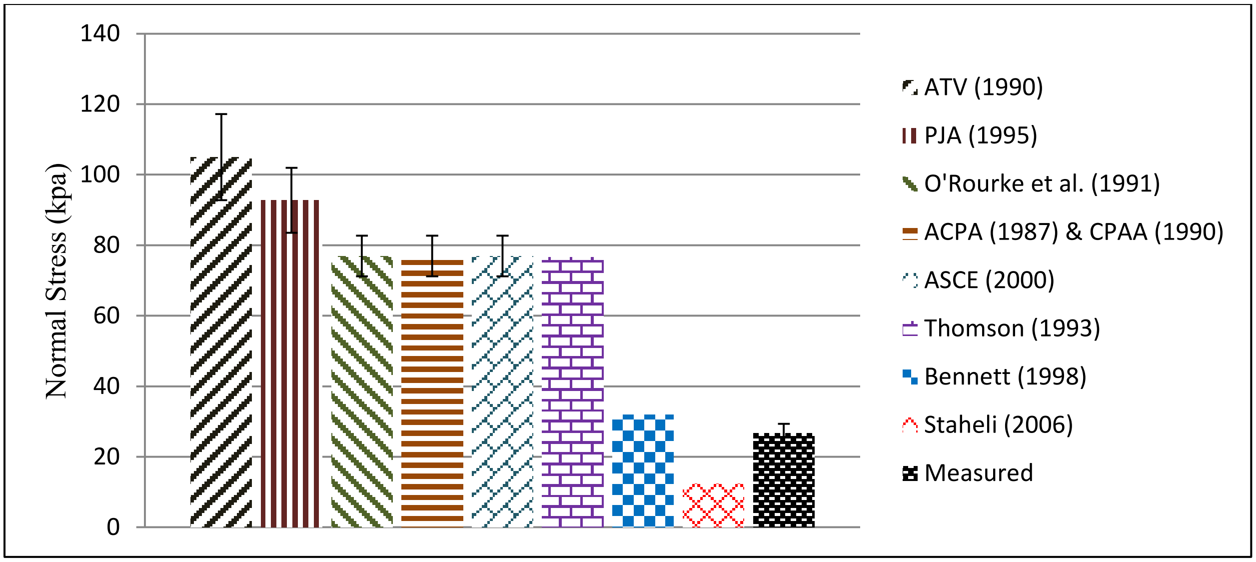


Figure 4.2 Comparison of normal stress values obtained from site (Scheme 4, Norris 1992) with methods in Table 4.1

Figure 4.2 shows that the ATV (1990) overestimates stress value by almost four times than that of the measured value. PJA (1995) model is also conservative by predicting the normal stress value close to ATV (1990). O'Rourke et al. (1991), ACPA (1987), CPAA (1990) and ASCE (2000) predict approximately the same normal stress values. However, the estimated stress value is almost twice as much of the measured value. Bennett (1998) model was able to predict the stress value which remained comparatively close to the field value than others. Staheli (2006) predicted stress value was almost half of measured value. As expected, various assumptions in different methods lead to variations in predicted stress values. ATV (1990) and PJA (1995) put a high emphasis on silo width value in comparison to other methods. Higher silo width forms higher portion of soil volume that contributes to the normal stress. O'Rourke et al. (1991) suggests elliptical collapse of soil over pipe. ACPA (1987), CPAA (1990) and ASCE (2000) consider silo width as the tunnel bore width which is smaller than the silo width assumed by ATV (1990) and PJA (1995). Since ACPA (1987), CPAA (1990) and ASCE (2000) consider full overburden depth of soil above pipe crown and the same silo width, their predicted stress values were almost the same. O'Rourke et al. (1991) recommends that normal stress contributing to soil height extends up to 3-4 times of pipe diameter above pipe crown. In this case study, the height was almost the same as full overburden depth. Thus, the predicted value by O'Rourke et al. (1991) stays close to the value of ACPA (1987), CPAA (1990) and ASCE (2000). Bennett (1998) proposes that soil stress is a function of pipe diameter and unit weight of soil. This model suggests specific arching factor value for particular type of soil based on the field test. Staheli (2006) substituted Terzaghi's (1943) silo width B with a B^* based on field observations. The new B^* is smaller than the pipe diameter itself which leads to lower vertical stress value on pipe than that of the models considering higher silo width value than the pipe diameter. Thus, ATV (1990) and PJA (1995) remain very conservative, while Bennett (1998) predicts stress value close to the measured value.

4.3.3. Results for cohesive soil

For cohesive soil, two sets of data were used for analysis. Using soil parameters from Norris (1992) mentioned in Table 4.3, the normal stresses were calculated using the methods in Table 4.1. Not all of the discussed methods are applicable to cohesive soils. Bennett (1998) includes a specific arching factor value for cohesive soils. ATV (1990) is applicable to cohesive soils although it does not consider the cohesion effect. ACPA (1987), CPAA (1990) and ASCE (2000) provided distinct equations for cohesive soils. Bennett (1998) and O'Rourke et al. (1991) models are not dependent on cohesion values as well as soil internal friction value for predicting normal stress on pipe. On the other hand, ASCE (2000), ACPA (1987) and CPAA (1990) provide different normal stress values for variable cohesion and soil unit weight. Figure 4.3 shows the comparison of the calculated normal stresses with the measured values. Using the data acquired by Marshall (1998) as outlined in Table 4.4, the normal stresses were calculated again to see the pattern of predicting normal stress values by the methods in question for cohesive soil. Figure 4.4 represents the comparison of normal stresses with the measured stresses by Marshall (1998). As the borehole remained stable throughout the drive length, sensors identified no normal stress values on the pipe crown except for the initial phase. Due to irregularities in the borehole shape, the sensors recorded some stress values in the beginning which is used for comparison in Figure 4.4.

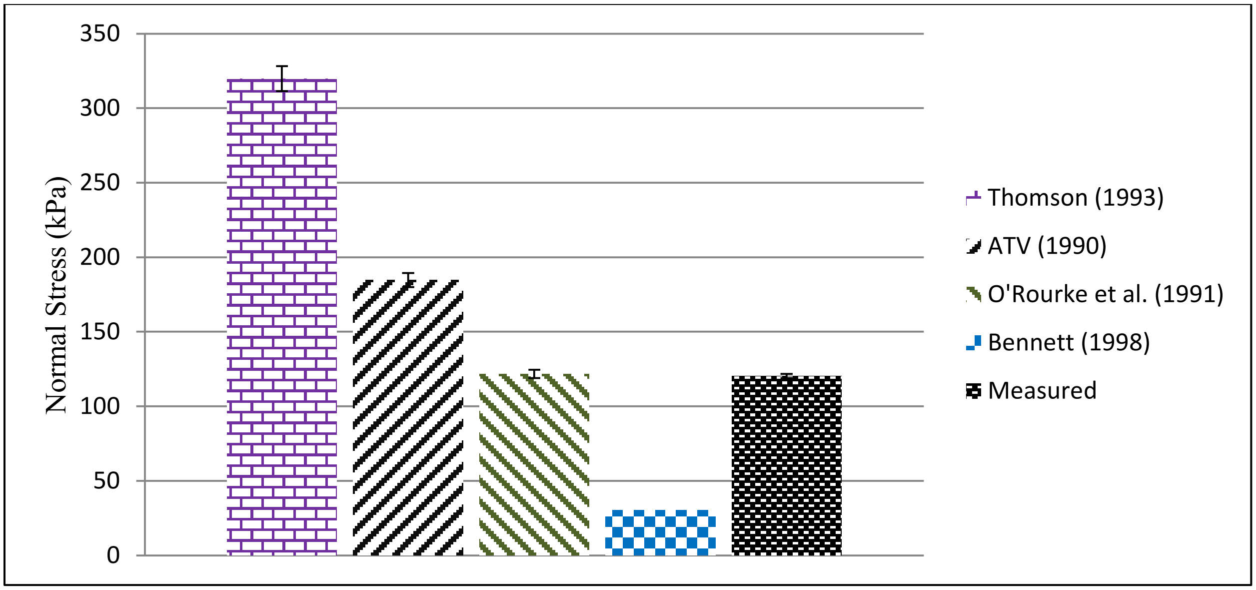


Figure 4.3 Comparison of normal stress value obtained from site (Scheme 3; Norris 1992) with mentioned standards and models

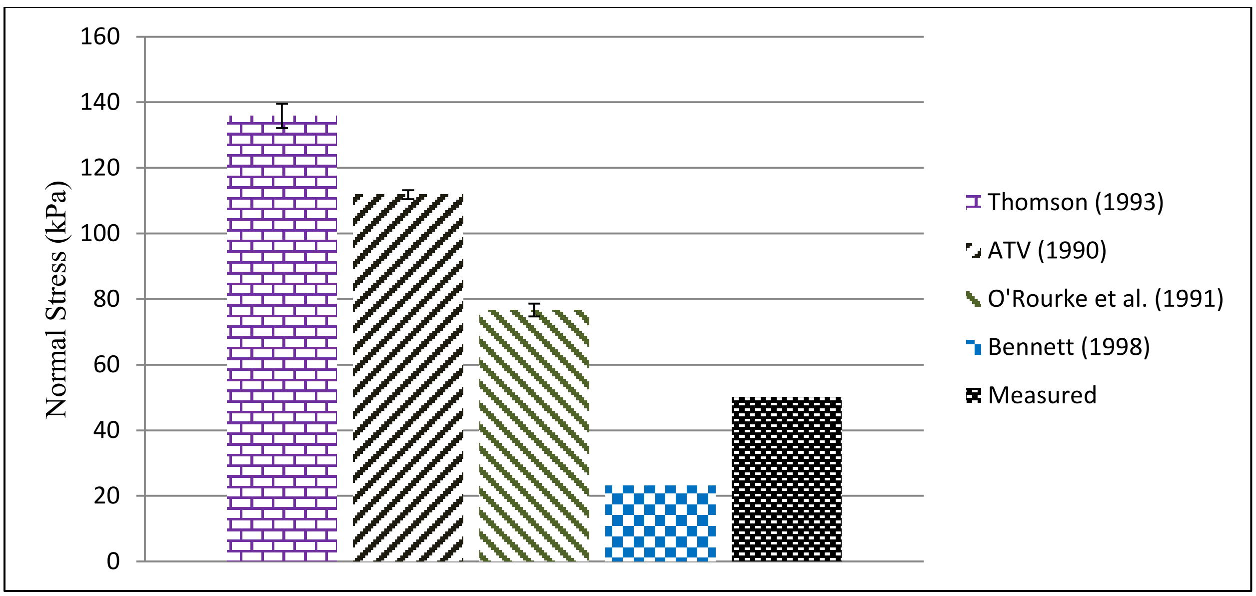


Figure 4.4 Comparison of normal stress value obtained from site (Scheme 6, Marshall 1998) with standards and models mentioned in Table 4.1

Both Figures 4.3 and 4.4 show very similar results. Normal stresses calculated by Thomson (1993) doubles the measured values in both scenarios. Thomson (1993) considers full overburden depth which results in a higher normal stress value. ATV (1990) also gives higher than the measured value but it is lower than that of Thomson's (1993). Although ATV (1990) is applicable to cohesive soil, it does not consider the effect of cohesion that reduces normal stress on pipe crown in calculating normal stress value. On the other hand, O'Rourke et al. (1991) predicts normal stress value comparatively close to that of the measured value than other methods in case of Figure 4.3, but almost fifty percent higher than the measured value in case of Figure 4.4. Although O'Rourke et al. (1991) model is considered reliable for calculating normal stress, occasionally it overestimates stress values as this model does not consider the shear transfer between the loosening soil and elliptical periphery. The model also overlooks soil density and over-consolidation ratio. Bennett (1998) recommended a specific arching factor value for cohesive soil based on field observation; normal stress calculated by this model remained almost half of the measured value or lower.

Similarly, ASCE (2000), ACPA (1987) and CPAA (1990) provide an equation for calculating normal stress value for cohesive soil. The borehole is expected to remain stable for soil having higher cohesion values; there is no normal stress formed on the pipe crown. In the first case study for cohesive soil, the above mentioned standards provide normal stress value that equals to zero on the pipe crown as the cohesion of soil was higher than the second case study. However, the measured value ensures the presence of normal stress. In the second case study, the cohesion of soil is comparatively less so the mentioned standards provide normal stress values on pipe crown which are close to the measured value.

4.4. Parametric study

4.4.1. Parameters

A parametric study was conducted for cohesive and non-cohesive soils to investigate the effect of different parameters on normal stress during pipe jacking. In this study, soil internal friction angle was assumed in the range of 28˚ to 46˚ for non-cohesive soil; this is the typical soil friction angle range for sand (Peck et al. 1974). Pipe jacking is applicable to pipes of small and large diameters. To monitor the consequence of variable pipe diameter on normal stress, a range of 0.3 m to 1.8 m pipe diameter was assumed for calculation. A height of 1.8 m to 11 m was used as overburden depth. Sufficient overburden depth is required for some methods to allow for the occurrence of the arching phenomenon (e.g., O'Rourke et al. (1991) model assumes that soil loosening height varies from 3 to 4 times of borehole diameter over pipe crown). Any overburden depth smaller than that height will contribute to normal stress on pipe. The maximum overburden depth was assumed in a way to include all the methods in this study.

Table 4.5 Summary of assumed parameters for parametric study

Parameters	Non-cohesive soil	Cohesive soil
Internal friction angle of soil (˚)	28-46	0-30
Overburden depth (m)	1.80-11	1.80-12.80
Pipe diameter (m)	0.30-1.80	0.30-2.10
Cohesion (kPa)	N/A	20-100
Unit weight of soil (kN/m^3)	N/A	14-20

A range of 20 kPa to 100 kPa was assumed as cohesion value of cohesive soil for parametric study (Peck et al. 1974). A range of 14 kN/m^3 to 20 kN/m^3 unit weight of soil was considered for analysis (Peck et al. 1974). Although ATV (1990) standards can be applied for cohesive soil, it neglects the effect of cohesion. ATV (1990) provides different results for different soil internal friction values. Soil internal friction angle, ranges from 0˚ to 30˚, was used for calculation (Peck et al. 1974). A height of 1.8 m to 12.8 m as overburden depth and 0.3 m to 2.1 m as pipe diameter was used for calculations.

4.4.2. Results for non-cohesive soil

Using the parameters in Table 4.5, arching factor was calculated using the methods in question. Figure 4.5 shows the variation of arching factor against overburden depth to pipe diameter ratio where the overburden depth changes with keeping the pipe diameter fixed (ϕ = 33˚). The fixed pipe diameter was 1.06 m which is an average of 0.30 m to 1.80 m mentioned in Table 4.5. The assumed soil internal friction angle for calculation was 33˚ which is a random value for non-cohesive soil in between the range referred in Table 4.5. Arching factor decreases as the h/d ratio increases. The pattern of change is almost the same for ATV (1990), PJA (1995), ASCE (2000) and ACPA (1987). Bennett (1998) and Staheli (2006) models show different pattern than other methods. The discrepancy of pattern among the methods in question may be explained by the failure pattern of soil. In case of the Staheli (2006) model, soil collapse zone is influenced by the void over the pipe crown. According to this theory, the vertical loading area over pipe is developed based on the observations of the shear planes of failure above the pipeline during over-excavations and the Mohr-Coulomb failure criteria. This zone is smaller than the failure zones considered by the other methods in question which leads this model to a lower arching factor value. In Bennett (1998) model, soil loading on pipe is a function of pipe diameter; arching factor is based on field observation which tends to be less conservative than the other theoretical standards. Unlike the other methods, O'Rourke et al. (1991) model shows a complete different pattern: soil loosening occurs up to a height of 3 to 4 times of borehole diameter. For a parametric study, the initial assumed overburden depth was less than soil loosening height. So the next higher overburden depth was considered as a starting point for O'Rourke et al. (1991) model in Figure 4.5. For a specific *h/d* ratio value, ATV (1990) predicts the highest value of arching factor, while Staheli (2006) model predicts the lowest value.

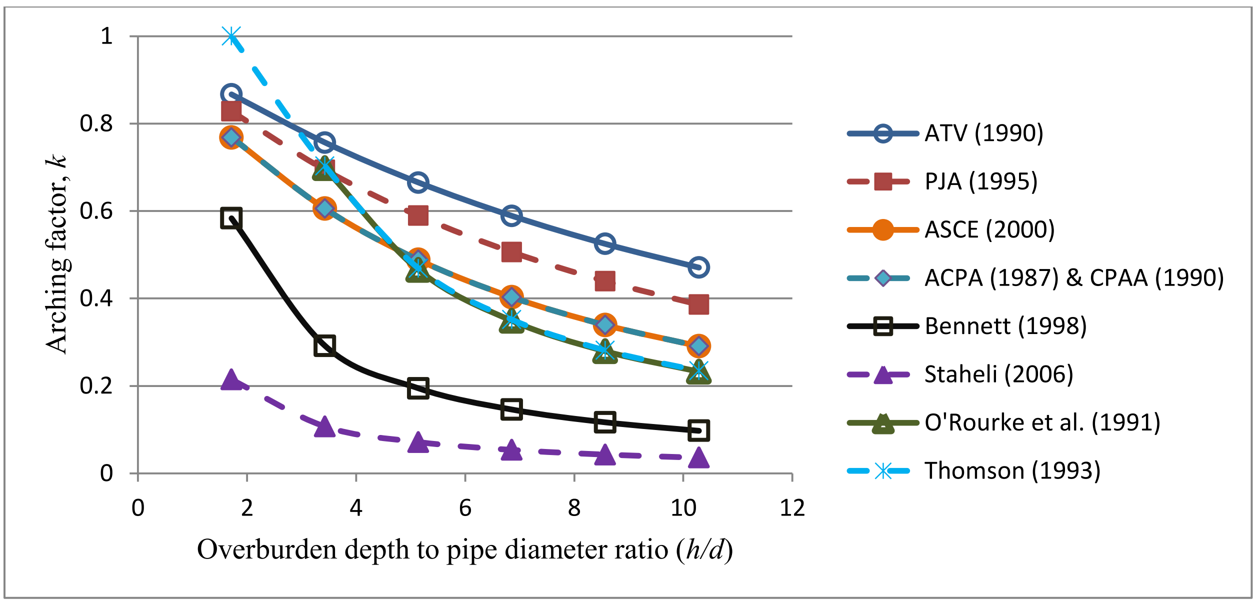


Figure 4.5 Variation of arching factor k vs. h/d for non-cohesive soil (h changes while d remains fixed)

Figure 4.6 presents the variation of arching factor against overburden depth to pipe diameter ratio, but this time pipe diameter changes with keeping overburden depth fixed (ϕ = 33˚). The fixed overburden depth is 6.40 m which is an average of 1.80 m to 11 m mentioned in Table 4.5. Changing pipe diameter produces a larger range of *h/d* ratio in Figure 4.6 than in Figure 4.5, but the pattern of change for every standard or model remains same as the pattern developed by changing overburden depth in Figure 4.5. Each standard or model provides a specific arching factor value for a specific overburden depth to pipe diameter ratio irrespective of overburden depth or pipe diameter change.

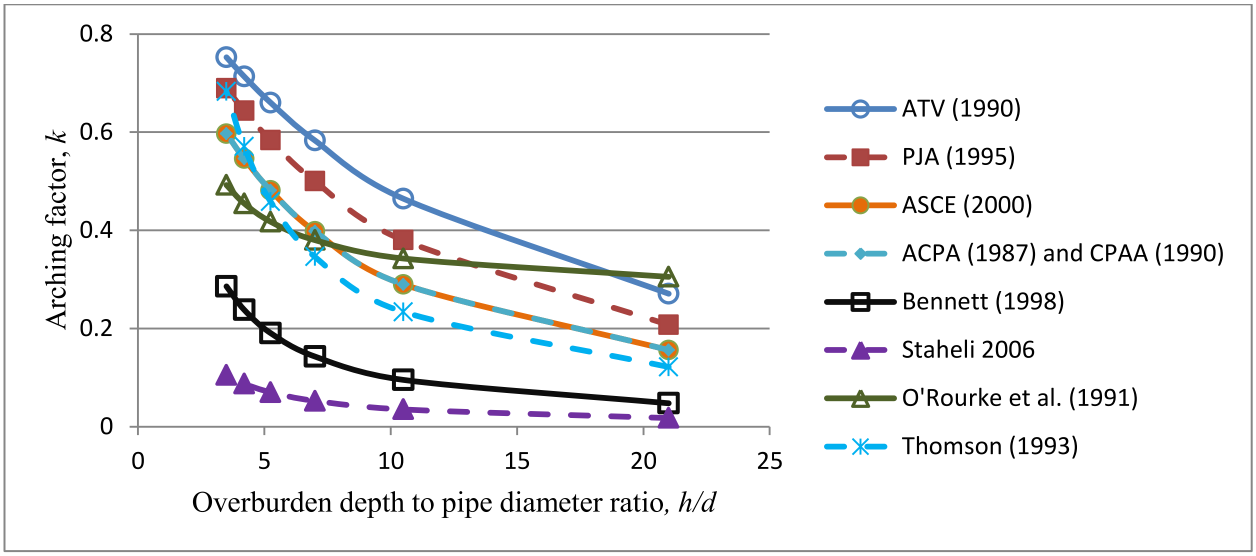


Figure 4.6 Variation of arching factor k vs. h/d for non-cohesive soil (d changes while h remains fixed)

To observe the effect of ϕ on arching factor, the arching factor was calculated again using the methods in question; the same parameters outlined in Table 4.5 were used. The overburden depth and pipe diameter used for calculation were 6.40 m and 1.06 m respectively. These are the average values of overburden depth and pipe diameter mentioned in Table 4.5. Figure 4.7 shows the variation of arching factor vs. soil internal friction angle ϕ. Arching factor is sensitive to soil internal friction angle in ATV (1990), PJA (1995) and Staheli (2006). Among these three, ATV (1990) standard represents higher rate of change in arching factor with the increase of soil internal friction angle. The rate of change of arching factor is almost constant for PJA (1995) model. Staheli (2006) model suggests much lower value of arching factor than the others for a specific ϕ value, while ATV (1990) suggests the highest arching factor value. For other standards or models except ATV (1990), PJA (1995) and Staheli (2006), soil internal friction angle does not have any effect on arching factor.

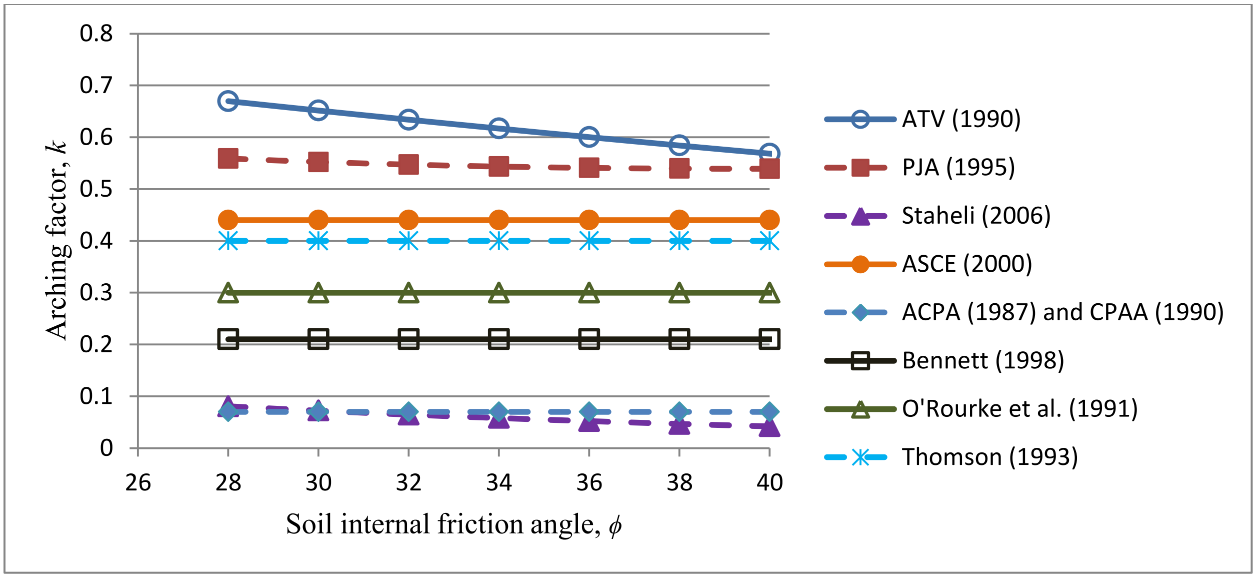


Figure 4.7 Variation of arching factor k vs. ϕ for non-cohesive soil

4.4.3. Results for cohesive soil

Using the above mentioned parameters for cohesive soil in Table 4.5, the arching factor was calculated, as shown in Figure 4.8 which represents the variation of arching factor *k* vs. *h/d* for cohesive soil while changing overburden depth with keeping pipe diameter fixed. The fixed pipe diameter was 1.22 m; average of 1.80 m to 12.80 m referred in Table 4.5. 15˚ was assumed as soil internal friction angle for calculation. For the assumed cohesion value, the borehole remains stable and the arching factor equals zero in ACPA (1987), CPAA (1990) and ASCE (2000). ATV (1990) recommends the highest value of arching factor for different *h/d* values and the change of arching factor is linear with the change of *h/d* ratio. Bennett (1998) model suggests higher value of *k* for smaller value of *h/d* and the arching factor reduces exponentially as the *h/d* value increases. O'Rourke et al. (1991) predicts higher *k* value than Bennett (1998) but lower than ATV (1990). O’Rourke et al. (1991) model also shows exponential change of arching factor with increasing *h/d* ratio but the exponential part extends more than Bennett (1998). Thomson (1993) does not consider arching effect in case of cohesive soil.

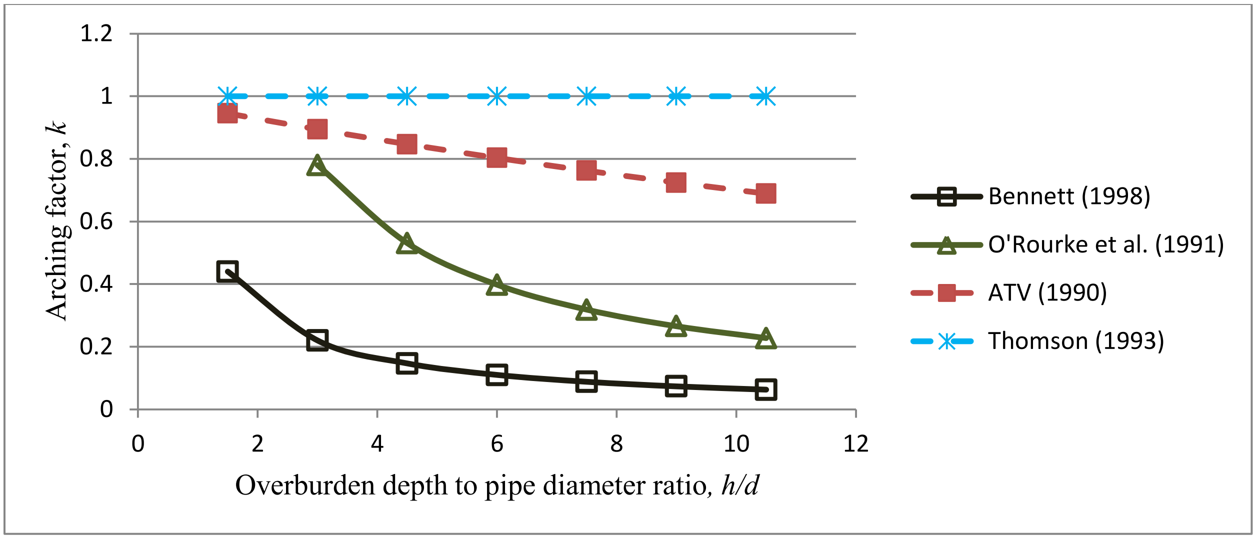


Figure 4.8 Variation of arching factor k vs. h/d for cohesive soil (h changes while d remains fixed)

Figure 4.9 represents the variation of arching factor for various *h/d* ratios while changing the pipe diameter with keeping the overburden depth fixed ($\phi = 15^{\circ}$). The fixed overburden depth for calculation was 7.32 m. The plots in Figure 4.9 shows the change of arching factor in larger range of *h/d* values than Figure 4.8, but both figures provide same pattern of change of arching factor for a specific standard or model. The prediction of arching factor for a fixed *h/d* ratio remains same regardless of pipe diameter or overburden depth change.

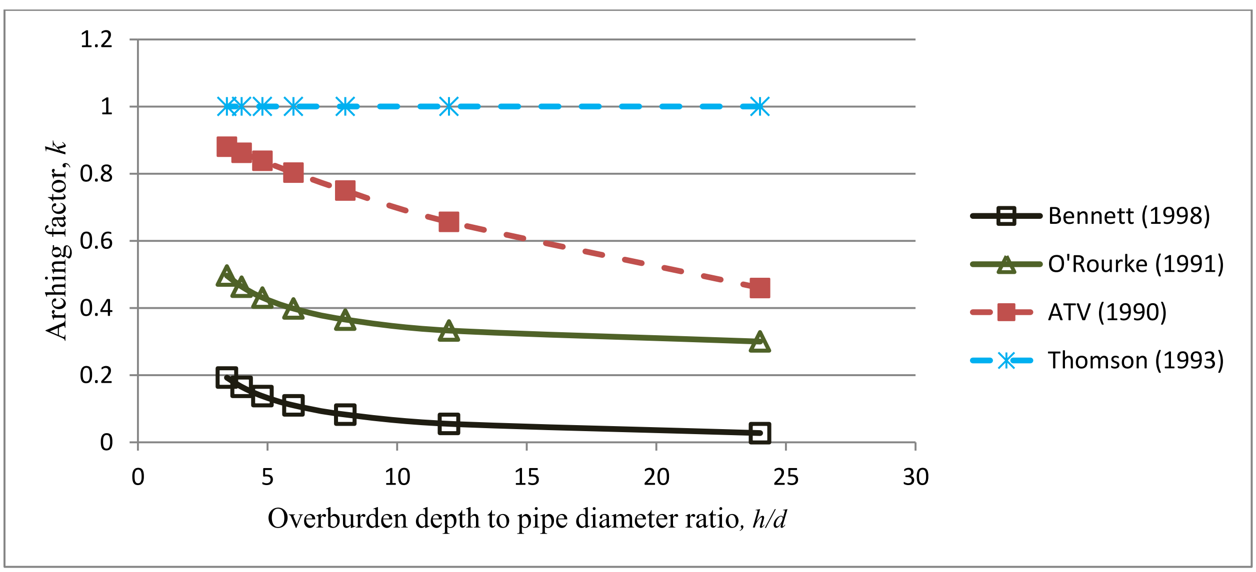


Figure 4.9 Variation of arching factor k vs. h/d for cohesive soil (d changes while h remains fixed)

Figure 4.10 shows the effect of soil internal friction angle on arching factor for cohesive soil. The used overburden depth and pipe diameter for calculation were 7.32 m and 1.22 m respectively. The assumed parameters in Table 4.5 provide that the bore hole will remain stable for ASCE (2000), ACPA (1987) and CPAA (1990). Bennett (1998) and O'Rourke et al. (1991) provide a specific arching factor regardless of soil internal friction angle. ATV (1990) exhibits a linear decreasing pattern of arching factor against increasing soil internal friction value. Thomson (1993) does not consider arching phenomenon in case of cohesive soil.

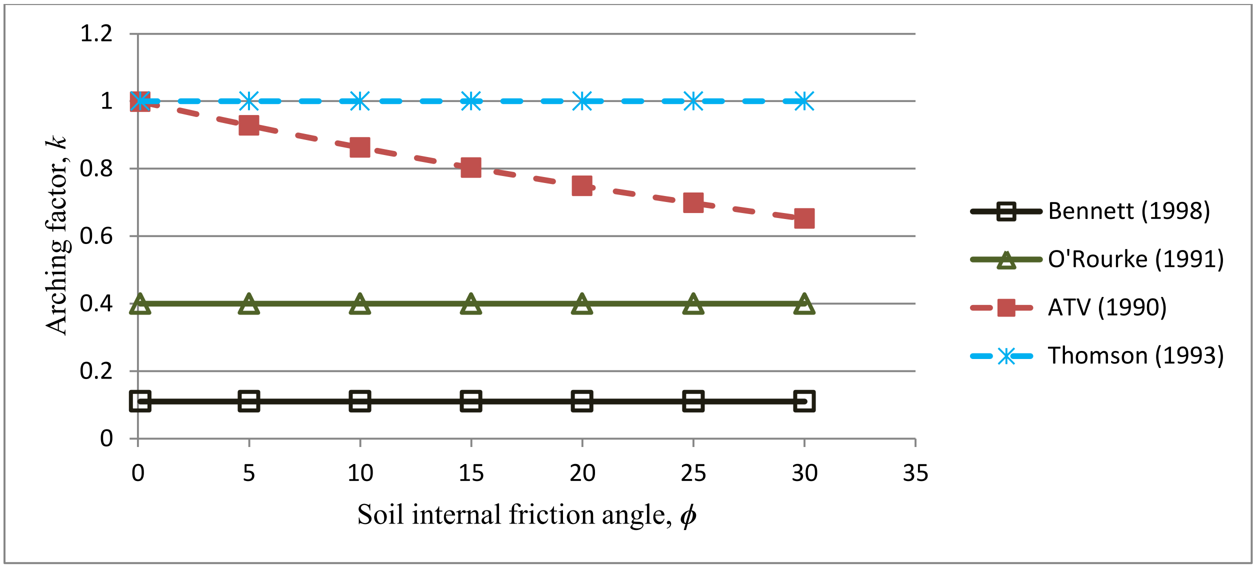


Figure 4.10 Variation of arching factor k vs. ϕ for cohesive soil

4.5. Conclusion

In this paper, a comparative study of nine distinct methods in predicting normal stress value during pipe jacking/microtunnelling has been conducted. It was found that only Bennett (1998) model predicted the normal stress values that were close to measurement for non-cohesive soil, when the other methods except Staheli (2006) overestimated normal stress values. Parametric study also revealed that the calculated arching factors for non-cohesive soil using Bennett (1998) model remain in the lower range than the other methods. For cohesive soil, the normal stress calculated using O'Rourke et al. (1991) model is closer to the measured values than the others. The predicted arching factor from parametric study also supports the case studies by remaining in the lower range. For higher cohesion of soil, ASCE (2000), ACPA (1987) and CPAA (1990) assume that the borehole remains stable. In Scheme 6 (Marshall 1998), the sensors identified no normal stress except for the starting phase which matches the recommendation. Thomson (1993) equation always leads to maximum normal stress on pipe as it considers full overburden depth for cohesive soil. ATV (1990) always remains conservative as it neglects the cohesion of soil. Nevertheless, more field data is still needed for further confirmation of the findings in this study.

5. Chapter 5: Summary, conclusions, and recommendations

5.1. Summary

GBM is one of the most precise trenchless methods for pipeline installation; however, the budget constraints of this method necessitate research on this sector to make it more cost effective. Soil investigation is one part of the process that is often ignored so as to prevent extensive initial costs. But lack of knowledge of drilled soil often creates problems and makes the installation process costly and lengthy. Researchers are trying to develop an alternative geotechnical investigation tool that can provide the same quality data as traditional tests, but at a more reasonable cost. Drilling indices may serve this purpose since change of drilling parameters is directly related to change in soil strength. This idea has proven to be effective in vertical drilling, but still has not been implemented in horizontal drilling. Consequently, this study analyzes the potency of those indices as geotechnical investigation tools. Field data from a GBM project in Edmonton, Alberta, Canada, was used to plot the indices in question against time. This study illustrated the applicability of those indices in horizontal drilling using recorded data from a GBM project. The best index for subsurface exploration was also recommended.

Furthermore, pipe jacking and microtunnelling were discussed. These methods are effective at maintaining line and grade of pipeline in congested urban areas. Predicting proper normal stress on pipe crown is vital during pipeline installation by pipe jacking or microtunnelling as it directly affects required jacking force and jacking pipe selection. Several methods can calculate normal stress, but calculating normal stress value for a specific soil condition by those methods is not consistent. A validation based on field-measured values is necessary for choosing the best method for normal earth pressure calculation. Nine standards and models for calculating normal stress on pipe crown were illustrated in this study. Next, those standards and models were validated against measured values. Based on field-measured soil parameters, normal stress on pipe crown was calculated and later compared with actual normal stress value. Based on this study, the most suitable standard or model for non-cohesive and cohesive soil was proposed separately.

5.2. Conclusions

In this research, five indices were used to explore the subsurface profile. Field-collected data from a GBM project were used to inspect the potency of indices in locating different soil layers. The analysis reveals that:

- The transition of soil layers can be identified by any of the discussed five indices. Any of the indices can provide trenchless personnel comparative strength profiles of soil, which can help decide on drilling bits, proper jacking force selection, etc. for the next reaming stages.
- Alteration index provides a clearer pattern of soil strength variation than other indices. The change of Somerton index and Γ-hardness parameter values is not acute at soil layers' transition points. MSE and drilling energy shows noticeable changes at various soil layers. The changes are more visible than Somerton index and Γ-hardness parameter. Alteration index is recommended for profiling since it produces the best results of soil strength variation along the drive length.
- The limitation of this study is that indices can provide only a relative soil strength profile, not specific soil strength based on a fixed index value. Further studies are required to make exact correlations between soil strength and index value. Still, subsurface profiling using indices can be very effective where costly geotechnical investigations are not feasible.

Normal earth pressure on jacking pipe was calculated using nine standards and models. One case study for non-cohesive soil and two case studies for cohesive soil were used to compare the aforementioned methods. The study discloses that:

- Bennett (1998) predicted that normal stress remains close to the measured value for non-cohesive soil. The other seven standards and models, except Staheli (2006), overestimate the normal stress value for non-cohesive soil. Staheli (2006) predicted that stress value was almost half of the measured value.
- In the case of cohesive soil, O'Rourke at al. estimated that normal stress is comparatively more precise than others with respect to field values. Occasionally it overestimates stress values as this model does not consider the shear transfer between the loosening soil and

elliptical periphery. ACPA (1987), CPAA (1990) and ASCE (2000) predict no normal stress on pipe for higher cohesion values. Normal stress calculated by the Bennett (1998) model remained almost half of the measured value or lower. Thomson (1993) does consider the effect of arching for normal stress calculation by assuming full overburden depth. ATV (1990) does not consider cohesion of soil, which might be the reason for overestimation of stress by this standard.

- Parametric study reveals that the difference in assumptions for normal stress calculating parameters led to inconsistency in predicted normal stress values by different methods.

5.3. Recommendations

- A limitation of GBM study is that a specific value of a specific index for a specific soil strength has not been defined yet. Further studies are required to correlate index value to soil type and strength.
- Normal stress calculating standards and models were validated using one case study for non-cohesive soil and two case studies for cohesive soil. More field data are required to increase the confidence level of the validation.

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