

AN INVESTIGATION OF PIPE JACKING LOADS IN TRENCHLESS TECHNOLOGY

by

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December 19, 2011

ABSTRACT

AN INVESTIGATION OF PIPE JACKING LOADS IN TRENCHLESS TECHNOLOGY

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Pipe jacking is a trenchless technology method of installing pipes under existing facilities such as roads and railroads. Predicting jacking forces is important for planning, design, and construction phases of these projects. The jacking forces dictate shaft or pit locations, thrust block or backstop design, jacking equipment, use of intermediate jacking stations, and pipe bearing capacity. Calculating jacking forces depends on several site and project parameters such as soil and site conditions, lubrication, size of overcut, and steering corrections. Excessive jacking forces can damage the pipe and destabilize the drive shaft, which may stop project progress. There are different methods of calculating jacking loads presented by researchers and industry organizations. By using different models, a high discrepancy can be observed in results creating unreliability when precise calculation is required. Even though these methods are supported by numerous valid experimental tests and recorded values based on real projects, the source of created resistant force is not certain and more research is required in this field. To cover the unknown, some methods introduce higher bound limit. For example, ASCE 27 recommends using experimental values to calculate frictional forces and do not consider the face pressure. Other researchers have considered detailed analysis of soil conditions at the face, recommend including project specific conditions such as pipe depth, and bore stability. This research presents an analysis of literature on

estimating loads in pipe jacking operations and compares recorded data from pipe jacking projects with a series of selected jacking force prediction models. The results compare reliability of different prediction models to accurately estimate jacking loads.

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CHAPTER 1

INTRODUCTION

This chapter presents a brief introduction to trenchless technology used in installing new underground pipeline. Advantages of the trenchless construction method over the traditional open cut method are enumerated, and the needs and objectives of this research are presented.

1.1 Background

Trenchless technology is a process for construction, renewal, and replacement of underground pipelines and utilities with minimal surface and subsurface disruptions (Najafi, 2010). Trenchless technology methods are divided into three main areas: (1) construction and installation of new underground pipelines and utilities (2) renewal; and (3) replacement of existing, old, and deteriorated underground pipelines and utilities. Installation of new pipelines and utilities using pipe jacking are further divided into two categories of worker-entry and non-worker-entry installation. Figure 1-1 highlights the subcategories of pipe jacking method.

Pipe Jacking can be used to place a large precast reinforced concrete or steel pipe or box culvert horizontally through the ground, usually beneath a road or railroad that cannot be interrupted. The major advantage of the process is its inherent non-disruptive nature. In these methods, only the exact portion of soil that will be moved is excavated and no intermediate ground supports are needed. The pipe or box segments are built away from the roadway, at the plant, without the constraints of shoring and traffic controls. When the pipe or box segments are manufactured and transported to the jobsite, by a specific trenchless construction method, are placed into the final position. Dependent on the project and site conditions, this process may take a few weeks. During this time, traffic proceeds overhead normally unaware of the

construction below. Advantages of pipe jacking over the open cut method can be enumerated as follows:

- minimizes possibility of pavement heave, settlement, cracks and the need for replacement of road surface,
- minimizes double handling of soils,
- minimizes dewatering requirements,
- provides little impairment to traffic flow and neighboring businesses especially in inner urban areas, and
- provides only short-term noise and emission pollution while excavating the starting and target shafts.

The non-disruptive nature of the process together with its inherent safety, simplicity and economy makes pipe jacking the method of choice for roadway crossings and culvert installations.

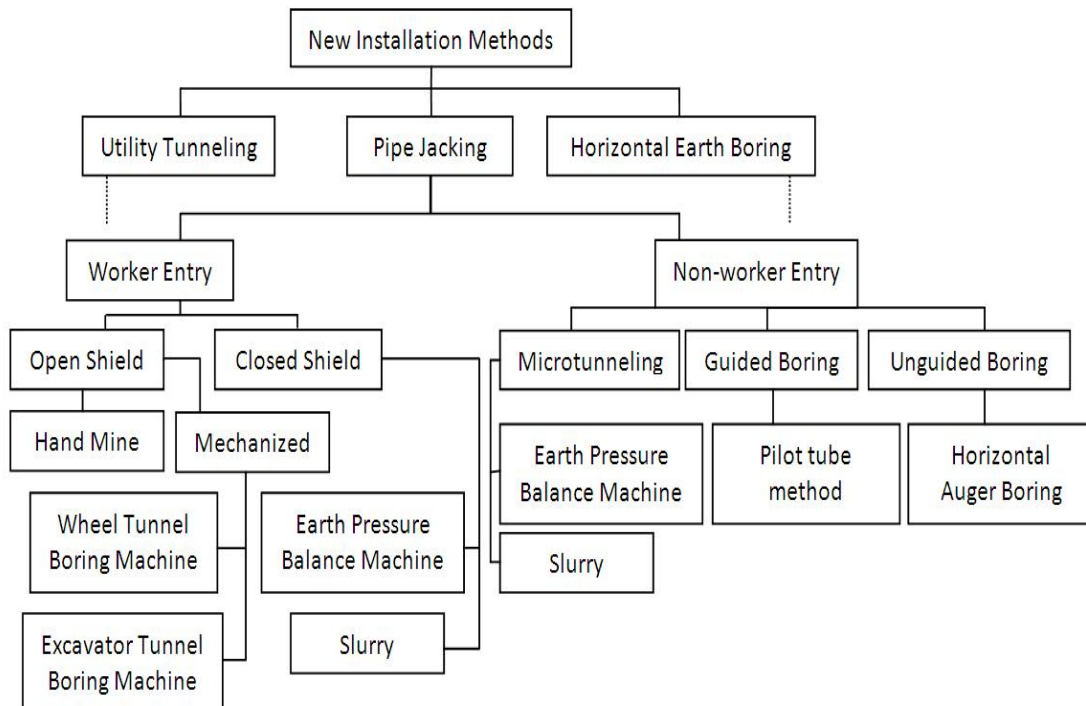


Figure 1-1 Classification of Pipe Jacking
(Modified from Najafi, 2010)

Several parameters affect selecting a suitable alternative among trenchless construction methods. The selection of a specific method is highly dependent on site and subsurface conditions. In addition to adequate specifications and guidelines for contractors to follow, a thorough soil investigation and an accurate underground utility location plan are critical for minimizing subsequent construction problems and claims.

1.2 Need Statement

To assure appropriate selection of trenchless construction methods, site investigation must include a soil test and topography survey as well as a precise design plan. At the present time, for most trenchless construction methods (TCMs)¹, standard guidelines defining special provisions have been developed, but few of them have a detailed design practice code, which makes the success of projects more heavily dependent on the experience of the contractor.

Pipe jacking is one of the most common and cost effective TCMs in installing underground pipelines, drainage culverts, and utilities. However, predicting jacking forces is important for planning, design, and construction phases of trenchless projects. Jacking forces dictate shaft or pit distances, thrust block or backstop design, jacking equipment, use of intermediate jacking stations, and pipe bearing capacity. Calculating jacking forces depends on several site and project parameters such as soil and site conditions, lubrication, size of overcut, and steering corrections. Excessive jacking forces can damage the pipe and stop project progress. This research compares pipe jacking guidelines and predicting models in the U.S. and other countries to search reliable and practical procedures to estimate jacking loads.

1.3 Objectives

The objectives of this research are (i) to gather and organize the research in pipe jacking construction methods (ii) to compare design code and guideline specifications in different countries, and (iii) to verify the predicting models with data obtained in pipe jacking projects.

¹For detailed description of trenchless construction methods, refer to Najafi (2005) and Najafi (2010).

1.4 Methodology

A comprehensive literature search was conducted to identify and review pipe jacking research, codes and standard guidelines. The searched sources included published reports, books, journal articles, theses and dissertations, as well as Websites. The subjects included (i) trenchless construction methods (TCMs), (ii) applicable TCMs in pipe and box culvert installations, and (iii) pipe jacking and microtunneling applications. Also, several surveys were conducted to gather data from consultants and contractors.

1.5 Thesis Organization

Chapter 1 presents an introduction to trenchless technology and specifically methods in new installation of underground pipelines and presents the objectives and methodologies of this thesis. Chapter 2 introduces the pipe jacking method and its main components; and a literature review of previous studies in pipe jacking. Chapter 3 presents the methodology of this research. In chapter 4, theoretical analysis of jacking load estimation is conceptually compared with case study data, and two hypothetical projects were presented. Finally, chapter 5 contains conclusions and recommendations of this research.

1.6 Expected Outcome

By evaluating different procedures to calculate jacking forces in trenchless construction methods, this research provides a comparison of prediction models. The results compare reliability of different prediction models to accurately estimate jacking loads.

1.7 Chapter Summary

In this chapter an introduction to this thesis is presented and background, objectives, research needs, methodology and expected outcome were described. Estimating accurate jacking force is an important parameter in pipe jacking projects. The research methodology will consist of a comprehensive literature search and compilation of guidelines to achieve the objectives of this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter reviews findings of a comprehensive literature search conducted as part of this research. First, a brief description of the pipe jacking construction method is presented, and then main components of pipe jacking operations are introduced. The literature search included (i) trenchless construction methods, (ii) pipe jacking construction methods, and (iii) investigation of parameters affecting jacking loads.

2.2 Pipe Jacking

Box or pipe jacking is a trenchless technology method, which utilizes thrust boring to push a box or pipe with hydraulic jacks under existing facilities such as roads or railroad tracks, with simultaneous soil excavation. The spoil is transported out of the jacking box or pipe manually or mechanically. Different parts of a pipe jacking system are illustrated in Figure 2-1. For worker-entry method, the minimum recommended diameter for pipes installed by this operation is 42-in. (1,000-mm). However, sometimes it is possible to install Reinforced Concrete Pipes (RCPs) with 36-in. (910-mm) (I.D.). Cohesive soils are the most favorable soil conditions for pipe jacking. It is possible to use pipe jacking in unstable soil conditions by considering special precautions to counterbalance the ground pressure during operation. Most box jacking is constructed for culverts and drainage structures. For diameters less than 98-in. (2,500-mm), circular pipe jacking is often utilized, but for larger sizes, generally rectangular cross sections are used (Lynn, 2006).

A cyclic procedure uses the thrust power of the hydraulic jacks to force the box or pipe forward through the ground as the face is excavated. The spoil is transported through the inside of the box or pipe to the driving shaft, where it is removed and disposed of. After each box or

pipe segment has been installed, the rams of the jacks are retracted so that another box or pipe segment can be placed in position for the jacking cycle to begin again. Because of the large jacking forces required to push large diameter pipe through the ground, the proper design and construction of the jacking thrust block supports are critical. The shaft floor and thrust reaction structure (thrust block) must be designed to withstand the frictional resistance of heavy box or pipe segments being placed ([Purdue Website](#), 2011).

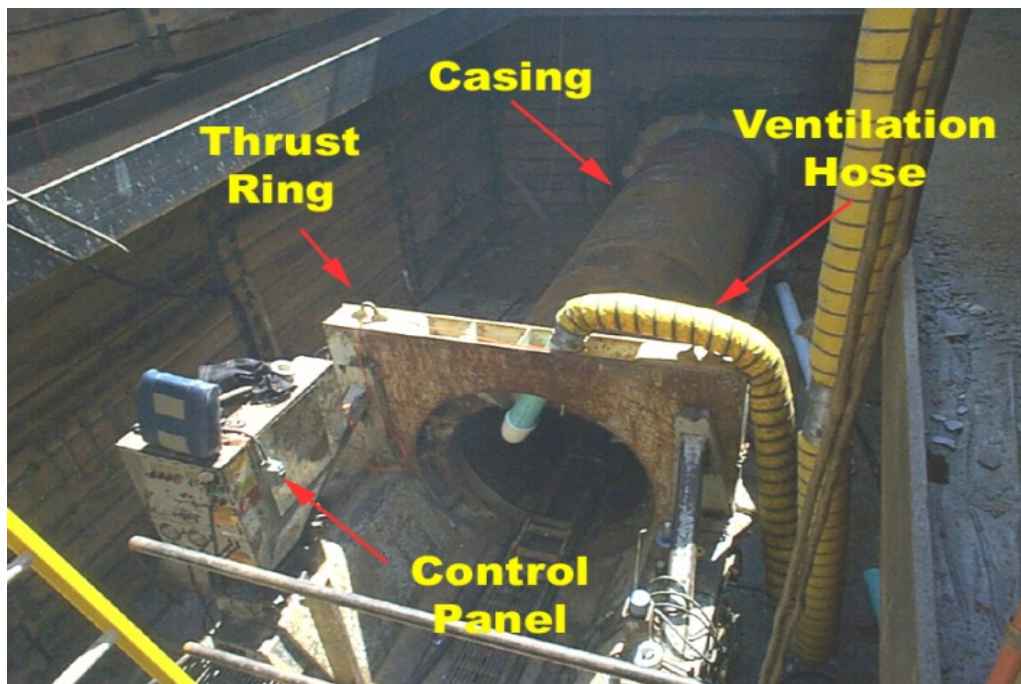


Figure 2-1 Different Parts of Pipe Jacking System
([Purdue Website](#), 2011)

Microtunneling is considered as a special type of pipe jacking. The main difference between pipe jacking and microtunneling methods is that there is no need for worker-entry in microtunneling, since all construction operations are handled outside the pipe, making it possible to install smaller diameters (less than 36-in. [910-mm]) pipes.

2.2.1 Introduction of Different Parts of Pipe Jacking System

Figure 2-2 illustrates a schematic diagram of pipe jacking operation, with more information provided in the following sections.

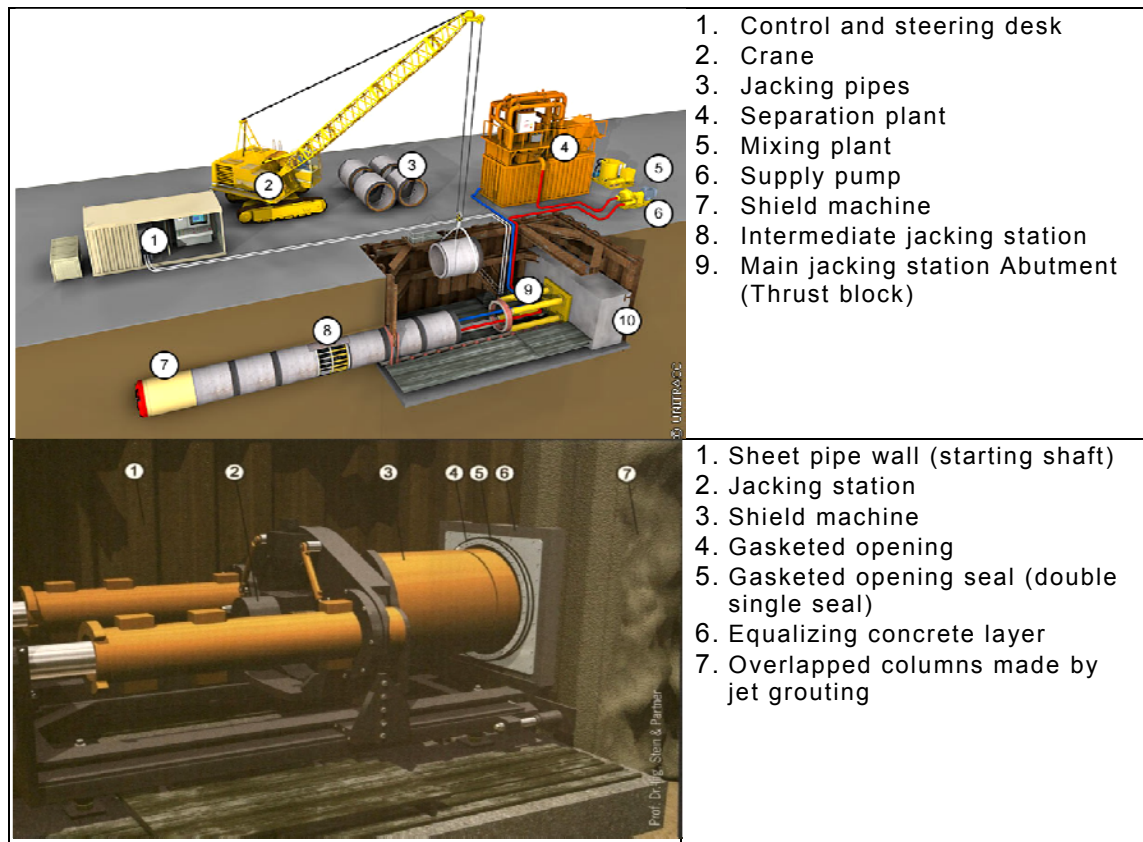


Figure 2-2 Pipe Jacking Method
(Stein, 2003)

2.2.1.1 Driving and Reception Shafts

To install a pipeline using pipe jacking, driving and reception shafts are constructed at permanent manhole positions, if possible. The dimensions and construction of a driving shaft vary according to the specific requirements of the pipe jacking installation, such as pipe length and diameter requirements. A reception shaft of sufficient size for removal of the jacking shield or the tunnel boring machine (TBM) is normally required at the completion of drive.

2.2.1.2 Thrust Block

A thrust block is constructed to provide support for jacking operation. In poor ground, piling or other special arrangements may have to be employed to increase the reaction capability of the thrust block. Where there is insufficient depth to construct a thrust block, as is

the usual case with jacking under road embankments, the jacking loads have to be resisted by means of a structural framework having adequate restraint provided by piles, ground anchors or other such methods for transferring the horizontal jacking loads.

2.2.1.3 Thrust Ring

To ensure jacking forces are distributed around the circumference of a pipe being jacked, a thrust ring is used. The jacks are interconnected hydraulically to ensure that the thrust from each is the same. The number of jacks used, varies based on the pipe diameter, length to be installed and anticipated frictional resistance.

2.2.1.4 Tunnel Boring Machine (TBM) or Tunneling Shield

Tunnel boring machine (TBM) or tunneling shield is the first part of a pipe jacking operation that goes into the ground. The TBM has the following functions (Stein 2003):

- creates the necessary cavity so that the jacking pipe string can be jacked in with a minimum degree of soil deformation and with the lowest possible skin friction,
- protects the cavity until the jacking pipes can finally carry all the loads and forces,
- secures the working face against collapsing soil and rock as well as groundwater, and
- steers the jacking operation along the designed line and grade while adhering to the permissible deviations.

Based on diameter of the pipe, watertable, and soil type, different types of TBMs or shields are recommended. Figure 2-3 presents different types of shields.

2.2.1.5 Jacking Pipes

Concrete is the most common pipe material used in pipe jacking, with the largest standard ranges, having diameters from 18-in. (450-mm) to 120-in. (3,000-mm) or greater if required. For smaller diameters high strength vitrified clay pipes (VCP) are commonly used (Stein, 2003). Vitrified clay pipes are available from 6-in. (150-mm) to 28-in. (700-mm) diameter. Steel pipes with 42-in. (1,000-mm) to 144-in. (3,000-mm) diameter and glassfiber-reinforced pipe (GRP) with 18-in. (450-mm) to 144-in. (3,000-mm) diameters may be used for jacking

(Najafi, 2005). The choice of a pipe material can be influenced by many factors, such as diameter, length of drive, ground conditions, and end use of the pipeline. Several common types of pipes are shown in Figure 2-4.

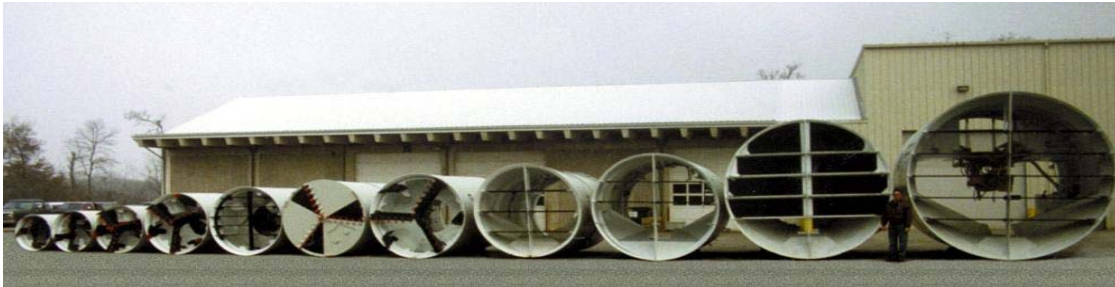


Figure 2-3 Shield Machine Types (Akkerman Inc., 2011)



a)



b)



c)



d)

Figure 2-4 Common Jacking Pipes

a) Reinforced Concrete Pipe, b) Vitrified Clay Pipe, c) Glassfiber Reinforced Pipe, d) Steel Pipe (Stein, 2003)

2.2.1.6 Pipe Joints

Pipe segments separated by elastic packers made of plywood or hardboard. Materials with a high Poisson's ratio like rubber and plastic are not suitable, as they cause spalling of the joint edges (CPAA, 2008). The joints of jacking pipes must meet the following criteria:

- tightness against internal and external pressures,
- transferrable longitudinal forces that ensure lateral stability, and
- corrosion protection of the pipe ends.

Some pipe jacking joints are illustrated in Figure 2-5.

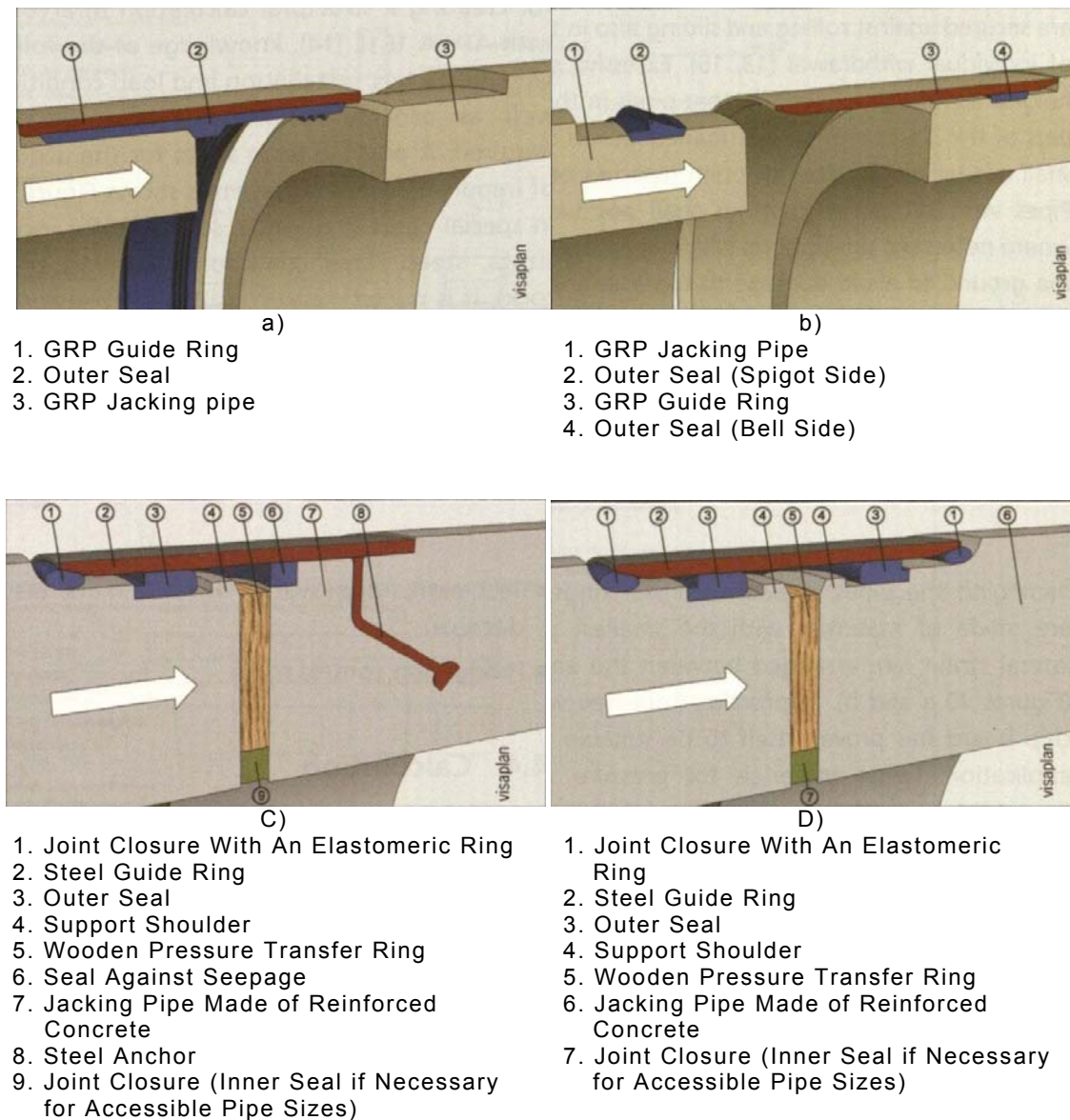


Figure 2-5 Some Pipe Jacking Joints, a) With GRP Guide Ring, b) With GRP Guide Ring and Outer Seal, c) Joint Closure with an Elastomeric Ring with Seal Against Seepage, and d) Joint Closure with an Elastomeric Ring (Stein, 2003)

2.2.1.7 Steering and Alignment Monitoring System

The initial alignment of the pipe jacking is obtained by accurately positioning guide rails within the driving shaft. To maintain accuracy of alignment during pipe jacking, it is necessary to

use a steerable shield or TBM, which must be frequently checked for line and grade from a fixed reference. For short or simple pipe jacks, these checks can be carried out using traditional surveying equipment. Remote-controlled techniques (microtunneling) require sophisticated electronic guidance systems using a combination of lasers and screen-based computer techniques. In Figure 2-6, some of the causes of misalignment are presented. Misalignment on may increase pipe jacking loads, which may damage the pipe and/or the pipe joint. Pipe joint damage may lead to eventual exfiltration and/or infiltration of the pipeline system.

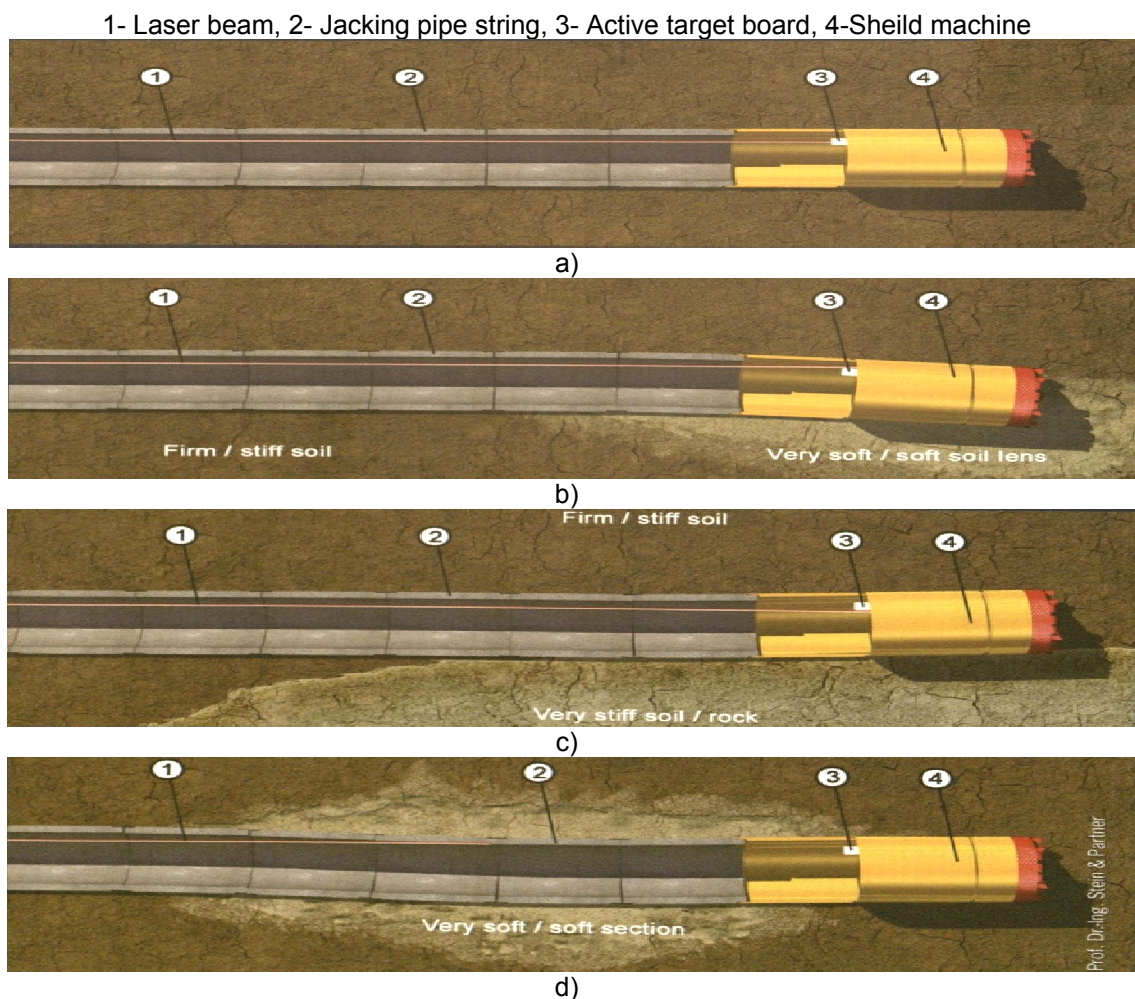


Figure 2-6 Possible Causes for Misalignment and Failure of the Measuring System
a) Straight path in homogenous subsoil conditions (correct acquisition of the laser beam), b) Sinking of the shield machine in very soft or loosely compacted soil, c) Grinding cut through differing hardnesses of stratified soils or encountering of a rock horizon, d) Grinding-in of the pipe string
(Stein, 2003)

2.2.1.8 Headwall

When the pipe jack or microtunnel is carried out below the watertable or it is required to apply high pressure lubrication, usually a headwall and seal assembly is used within each driving and reception shaft. The use of these tools prevents seepage of ground water into the shaft and associated ground loss, and retains pipe annular space lubricated. In Figures 2-7, 2-8, and 2-9, different types of head wall and their applications are shown.

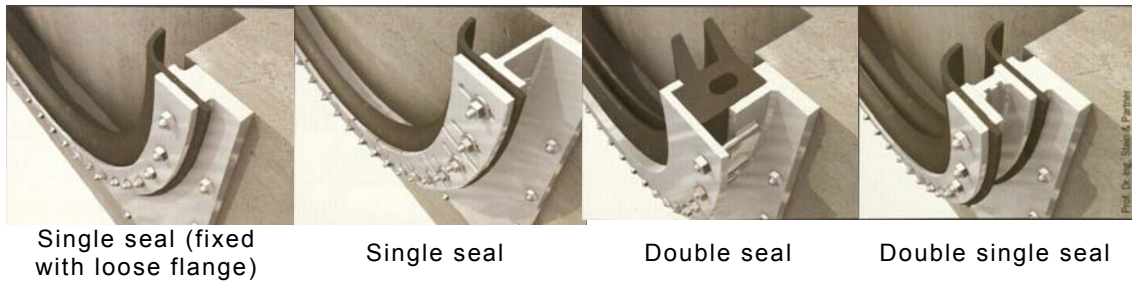


Figure 2-7 Elastomeric Seal Ring for Gasketed Openings, Sealing Exit, and Entry Openings (Stein, 2003)

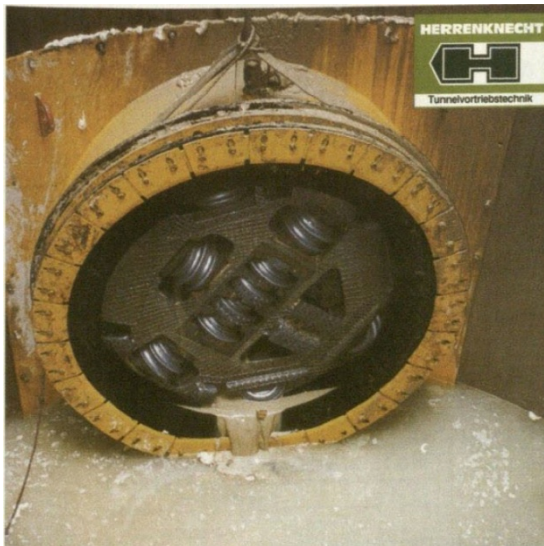


Figure 2-8 Entry Through an Entry Top Hat Into a Sinking Shaft of Steel (Stein, 2003)

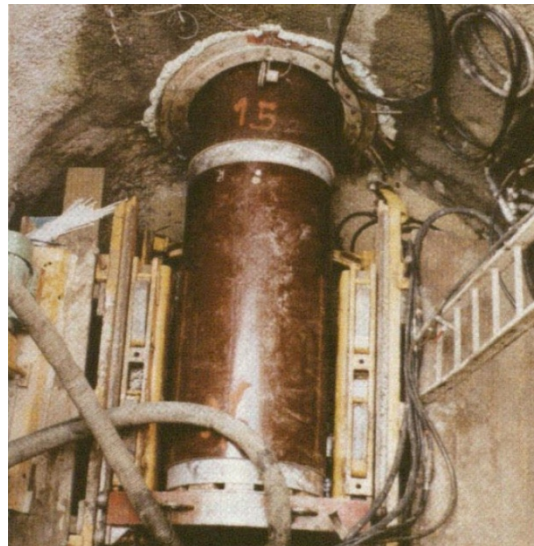


Figure 2-9 Existing out of a Starting Shaft Secured with Sprayed Concrete (Stein, 2003)

2.2.1.9 Intermediate Jacking Stations (IJS)

To redistribute the total required jacking force on the jacking pipe, intermediate jacking stations (IJS) are frequently used between the driving shaft and the tunneling machine (Figure 2-10). The IJS is useful where ground conditions at the drive shaft are poor or of low inherent strength, when the frictional resistance exceeds the bearing capacity of the jacking pipe or the jacking frame or the thrust block.

According to the British Pipe Jacking Association:

“A special twin pipe set incorporating an increased length steel collar, which slides over a corresponding length spigot, is introduced into the pipeline. Hydraulic jacks are placed between the two opposing pipes such that when activated they open the gap between the leading and trailing pipes. The inter-jack station is then moved forward with the pipeline in the normal way until it becomes necessary to supplement the jacking forces available from the shaft. To reach the design value or when the available thrust force is insufficient to move the pipeline forward, then the pipes behind the intermediate jacking station are held stressed back to the thrust wall in the launch pit. The jacks in the intermediate jacking station are then opened, thus advancing the forward section of the pipeline. At completion of the stroke of the inter-jacks, the main jacks in the thrust pit are actuated, advancing from the rear of the pipeline to its original position relative to the leading pipes, and thereby closing the intermediate station jacks. The sequence is then repeated for the duration of the pipe-jack and, on completion, the jacks and fittings are removed and the inter jack closed up. Inter jack stations are not only used to increase the jacking lengths achievable, but also to reduce the loads that are transmitted to the shaft structure” (PJA, 1995).

2.2.1.10 Lubrication

The pipe jacking shield or TBM is designed to produce a small overcut on the external diameter of the jacking pipe. By injecting a lubricant into this overcut, the jacking pipe can, at least in theory, be jacked freely through a fluid medium (Figure 2-11). In practice, however, lubrication losses may occur into the surrounding ground. If lubrication losses can be controlled, it may result in considerable reductions in jacking forces and, therefore, longer jacking lengths (PJA, 1995).



Figure 2-10 Intermediate Jacking Stations
(Akkerman Inc., 2011)

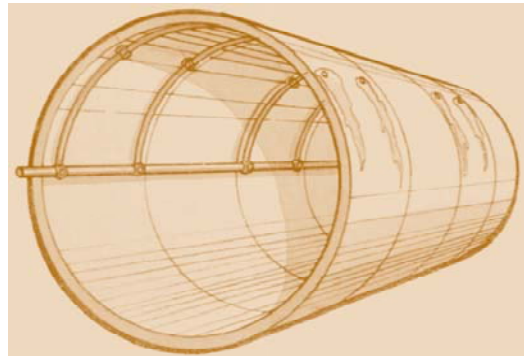


Figure 2-11 Typical Lubrication
Arrangement (PJA, 1995)

2.2.2 Non-worker Entry Pipe Jacking

Some non-worker entry techniques, such as Pilot Tube Microtunneling (also known as Pilot Tube or Guided Boring), use pipe jacking construction methods. In this method, a steerable pilot pipe jacking (Figure 2-12), are used, and the jacking operation is carried out in two or maximum of three sequential phases.

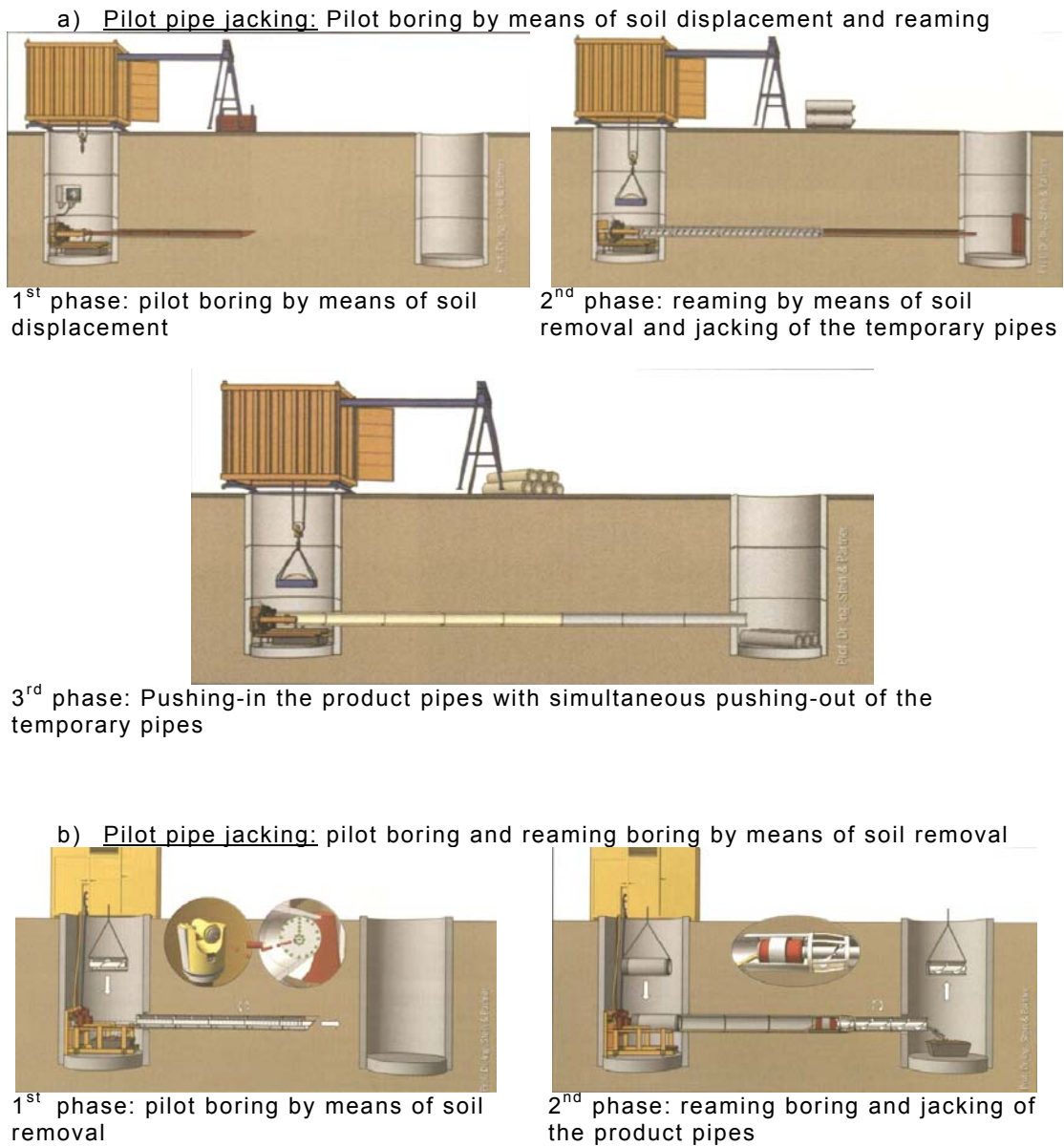


Figure 2-12 Pilot Tube Microtunneling
(Stein, 2003)

2.3 Jacking Loads

As said earlier, designing and selecting segments of a pipe jacking operation, such as jacking pipe, jacking frame, thrust block, intermediate jacking stations, and lubrication system, depend on a reliable jacking force calculation. In a jacking process, extensive and permanent

damages can result (Figure 2-13). The risk of damaging the pipes by overloading is high due to incorrect steering movements or exertion of increased jacking forces. Although extensive soil investigations can reduce these risks, failures and damages cannot completely be excluded. Based on a study carried out in Germany in 1993, (Figure 2-14), 80% of damages in pipe jacking operations were caused by face failure and excess stress in jacked pipe segments (Stein & Partner GmbH, 2011).

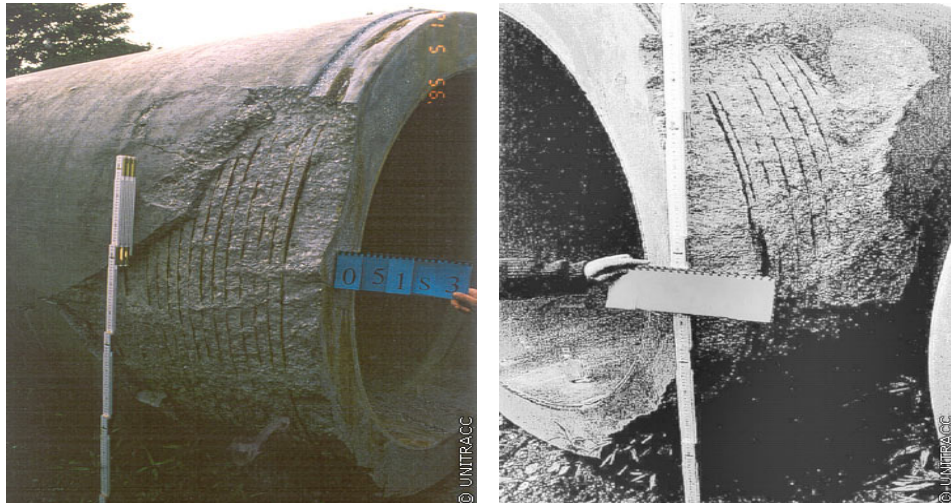


Figure 2-13 Example of a Reinforced Concrete Spalling Resulting from Exceeding the Concrete Compressive Strength (Stein & Partner GmbH, 2011)

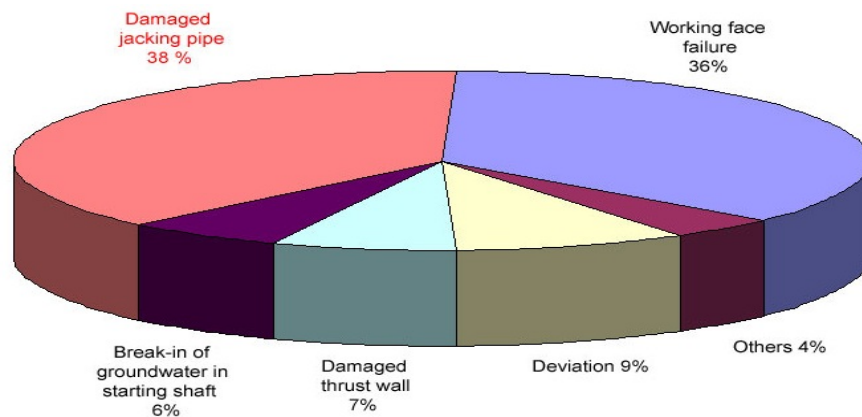


Figure 2-14 Distribution of Damages at Pipe Jacking in Germany (Stein & Partner GmbH, 2011)

2.3.1 Previous Reserach

There are few publications from Japan that are in English. Nanno (1996) at Kyoto University, Sugimoto (2002, 2010) at Nagaoka University of Technology, Khazaei (2004) at Kyushu University have published some of their research studies in English. The above research included curved pipe jacking, modeling shield behavior under different soil conditions, as well as lubrication effects in reducing jacking loads.

In the United Kingdom, a state-of-the-art work completed by Ripley (1989), Norris (1992), Zhou (1998), Chapman (1999), Borghi (2006) and Milligan et al (1989, 2010) advanced understanding of jacking concrete pipe. The synthesis of this research later became the British Pipe Jacking Guidelines. Research in Germany into pipe jacking and microtunneling has been summarized in English by Stein et al, in several publications (1989, 2003, and 2005). In France, several full-scale pipe jacking projects were monitored to provide data on jacking force by Kastner et al. (1996-2002). Broere (2003) studied the tunneling face stability at the Delft University, and Barla (2006), at the University of Politecnico di Torino Italy, conducted some studies about applicability of microtunneling technology in Italy's urban subsoil.

In the United States, a series of microtunneling tests were conducted by Bennett and Taylor (1993) at the U.S. Army Waterways Experiment Station (WES), Vicksberg Mississippi. Several studies and tests by Najafi et al. (1993) at Louisiana Tech University were carried out for the evaluation of a new microtunneling propulsion system. Coller et al. (1996) presented jacking loads measured on seven full-scale pipe jacks installed at various locations in the United States. Staheli and McGillivray studied jacking force (2006) and lubrication effects (2009) at Georgia Tech University. The following sections provide more details on previous research.

Ripley (Oxford University, 1989) conducted a series of tests on scaled (1:6 and 1:10.5) reinforced concrete pipes to assess performance of the pipe sections, joints and use of joint packing materials as well as pressure changes in soil, pipe geometry and strains induced in pipes. He analyzed pipe deformation, deflection angles between consecutive pipes, distribution

of stress concentrations, and the effects of joint packing materials on allowable jacking loads as well as induced stress magnitudes in the pipes.

Norris (Oxford University, 1992) studied pipe jacking method in full scale. This study involved monitoring a series of five pipe jacking operations. In each case, a heavily instrumented pipe was incorporated into the pipe string to measure pipe joint stresses, pipe and joint compressions, and contact stresses between pipe and ground. Total jacking loads and movements of the pipe string were measured and all results correlated with a detailed site log, full tunnel alignment surveys, and observed ground conditions.

The main objectives of above research were to determine effective radial pressures affected by soil in-situ stresses, stiffness and strength, ground water conditions, rate of progress, pipeline misalignment and use of lubrications. He investigated the relationship between pressure distributions at pipe joints and measured tunnel alignments, and noted that small angular deviations between successive pipes caused severe stress concentration on their ends. His back analysis showed that the linear stress approach of the Concrete Pipe Association of Australia can adequately match the measured stresses and could be used by pipe manufacturers to provide design data on allowable jacking forces on the basis of pipe size, packer properties, concrete strength and angular alignment.

Zhou (Oxford University, 1998) worked on the performance of the concrete pipes during jacking under working conditions to seek possible improvements in the design of pipe and pipe joints by numerical modeling. He investigated several factors affecting the pipe performance, such as, stiffness of the surrounding soil, the misalignment angle at the pipe joint, and the interaction between the pipe and surrounding soil. Based on the numerical results, several joint designs for improving the pipe strength were proposed and tested in the laboratory. Both the laboratory tests and the back analyses suggested that reinforcement at the pipe section end and the prestressed band at the pipe joint improved overall pipe performance during jacking.

Marshal's (Oxford University, 1998) study was the third phase of a research program on the performance of concrete pipes during installation by pipe jacking. His research was a continuation of the on-site monitoring of full-scale pipe jacks during construction. He experimented with different excavation methods, which included hand tools, slurry machines and an open face tunnel-boring machine with four types of soil, and he used pipes with an internal diameter ranging from 42-in. (1,000-mm) to 70-in. (1,800-mm). His main objective was to collect information on jacking loads and stresses at the pipe-soil interface to provide a better understanding for future designs.

Bennett (University of Illinois at Urbana-Champaign, 1998) investigated three microtunneling systems by incrementing a 340-ft (115-m) long test facility with six different soil sections. He measured machine performance and ground response and evaluated the impacts of geotechnical, operational, and geometric factors that influence jacking loads and ground deformations.

Staheli (Georgia Institute of Technology, 2006) focused on mechanisms that control interface shearing between pipes and granular materials and developed a model to predict jacking forces. By employing existing theoretical prediction model for non-lubricated jacking forces, she introduced a procedure for predicting both non-lubricated and lubricated jacking forces.

Lubrication is essential to reducing the frictional resistance generated at the pipe-soil interface. Although lubrication is widely utilized, there is not a clear understanding of the conditions required to obtain the full benefits of lubrication. Successful lubrication is achieved by appropriately adjusting lubrication properties to restrict drainage into the soil and prevent pressure dissipation. Borghi (University of Cambridge, 2006) made a physical replica of the pipe jacking process at laboratory scale to identify key interactions and measure the effects of lubrication chemistry on the radial effective stresses between pipe and clay soils. Another work by McGillivray (Georgia Institute of Technology, 2009) focused on bentonite slurry

characteristics and interface behavior under different lubricating conditions with the goal of further understanding the mechanisms responsible for the large friction reductions observed in the field.

2.4 Chapter Summary

In this chapter, pipe jacking components were explained in details. A complete literature review was presented on research conducted worldwide on jacking loads, pipe and joint performance, effects of lubrication, and pipe soil interactions.

CHAPTER 3

METHODOLOGY

3.1 Introduction

Calculating jacking force depends on several parameters such as pipe and joint configurations, soil parameters like friction and face penetration resistance, and soil resistance due to misalignment. Thrust block capacity has an effect on limiting jacking force at the driving shaft. Lubrication, operation continuity, overcut, and watertable levels have great impacts on required jacking force. In this chapter, the research and guidelines regarding pipe jacking force calculation will be presented. The current ASCE 27 (2000) is compared with other countries pipe jacking provisions. In this comparison, the ASCE guidelines are considered as the base while differences and complementary concepts from the other standards are presented.

3.2 Jacking Loads

Jacking force must overcome the frictional resistance of the pipe surface and jacking machine (TBM), as well as the face resistance of TBM. Pipe jacking is calculated as follows (Najafi, 2005) (Figure 3-1):

$$V \geq PE + \Sigma R \quad (3.1)$$

Where:

V Jacking force [ton, kN]
PE Peak resistance [ton, kN]
 ΣR Frictional resistance of the TBM + pipe string (skin friction) [ton, kN]

The factors influencing the value of the jacking force are:

- length, alignment, and outside dimensions of the pipeline to be jacked,
- weight of pipe,
- height of overburden,
- nature of soil and watertable and effects of dewatering,

- loads on shield (both skin and face pressure on the shield or TBM),
- whether operation is continuous or interrupted,
- size of overcut, and
- amount and type of lubrication.

1. Shield (TBM), 2. Jacking pipes, 3. Hydraulic jacks, and 4. Driving shaft

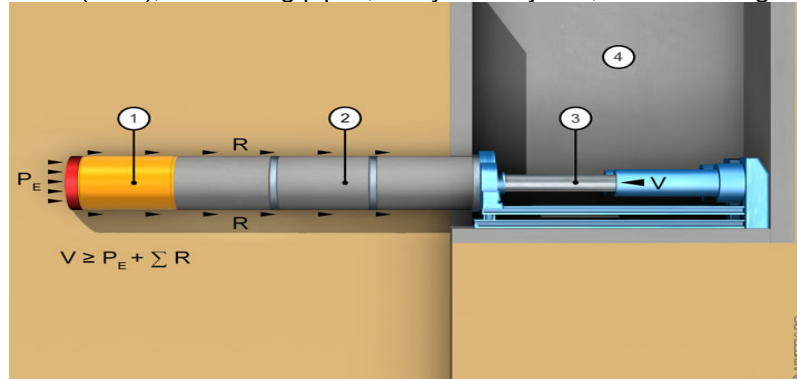


Figure 3-1 Penetration Resistance and Jacking Force
(Stein & Partner GmbH, 2011)

The correct estimation of the required jacking force is very important for:

- selecting the jacking pipes based on permissible jacking force given by the manufacturer,
- determining the jacking length and thus the spacing of the shafts or intermediate jacking stations, and
- designing of the driving shaft and the thrust block.

3.2.1 Jacking Resistances - Composition

3.2.1.1 Resistance Due to Friction

Eq 3.2 calculates total friction on the pipe surface.

$$\Sigma R = \mu \cdot N \quad (3.2)$$

where μ is the coefficient of friction [-] and N [kN, ton] is the normal force, which acts radially on the pipe due to earth pressure.

3.2.1.1.1 Coefficient of Friction

The surface roughness and hardness of different pipe materials affect the interface friction between the pipe material and soil at the interface. In most literature, estimating the friction coefficients between the soil and pipe material is related to the friction angle of the soil (ϕ'), and use of a reducing factor from $\frac{1}{4} \phi'$ to $\frac{3}{4} \phi'$ on the soil's internal friction angle to develop the frictional coefficient. Table 3-1 presents experimental values for frictional coefficients (Staheli, 2006).

Table 3-1 Pipe-soil Interface Friction Coefficients for Residual Soil Friction Angles from 25 to 40 degrees on Select Pipe Materials (Staheli, 2006)

Soil at Interface	Interface Friction Coefficient between Soil and Pipe					
Residual Friction Angles	Hobas	Polycrrete	Permalok Steel	Wet Cast Concrete	Vitrified Clay pipe	Packerhead Concrete
25	0.37	0.40	0.38	0.43	0.42	0.49
26	0.39	0.41	0.40	0.45	0.44	0.50
27	0.41	0.42	0.42	0.47	0.46	0.52
27.9 Ottawa 20/30	0.43	0.43	0.44	0.48	0.48	0.53
28	0.43	0.43	0.44	0.48	0.48	0.53
29	0.45	0.44	0.46	0.50	0.50	0.55
30	0.47	0.45	0.48	0.51	0.52	0.56
31	0.49	0.46	0.51	0.53	0.54	0.57
32	0.51	0.47	0.53	0.55	0.56	0.59
33	0.53	0.48	0.55	0.56	0.58	0.60
34	0.55	0.49	0.57	0.58	0.60	0.61
34.6 Atlanta Blasting	0.56	0.49	0.58	0.59	0.61	0.62
35	0.57	0.49	0.59	0.60	0.62	0.63
36	0.59	0.50	0.61	0.61	0.64	0.64
37	0.61	0.51	0.63	0.63	0.66	0.65
38	0.62	0.52	0.65	0.65	0.68	0.67
39	0.64	0.53	0.67	0.66	0.70	0.68
40	0.66	0.54	0.69	0.68	0.72	0.69

3.2.1.1.2 Normal Forces

Calculating normal forces on pipe during pipe jacking operation is very controversial. There is little understanding regarding mechanisms of arching effect in soil during pipe jacking

operations. As a result, there is a wide range of approaches proposed to calculate the normal force in jacking force models that have been developed to date. Some models do not allow for soil arching and base the normal force on the soil unit weight and depth of cover. Nevertheless, most of the models have been based on trap door experiments or the silo model conducted by Terzaghi's Arching Theory (Terzaghi, 1943) to estimate vertical stresses on the pipe.

Terzaghi's Arching Theory

Terzaghi's arching theory is based on his experiments on dry, cohesionless materials placed on a platform between two plates; as illustrated in Figure 3-2. The platform had a trap door mounted to a scale that would measure the stresses. When trap door was removed, the soil began to yield. Terzaghi found large decreases in the vertical stresses for very small displacements of the trap door. He explained this phenomenon as creation of arching effect in the soil mass above the door. This model does not simulate an exact circular tunnel, but it illustrates the relationship between the movement of the soil mass into the annular space around the pipe and the vertical stress reduction on the pipe.

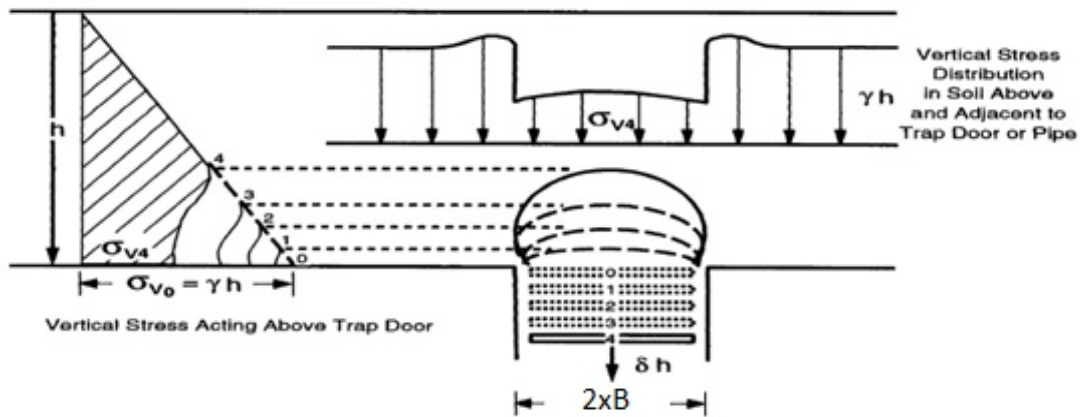


Figure 3-2 Terzaghi's Trap Door Experiment (Terzaghi, 1943)

As the trap door displaces downward, shear stresses develop in the yielding column of soil above the trap door (Figure 3-3); thus, vertical soil stress (Terzaghi 1943) is:

$$P_{Ev} = \frac{B \left(\gamma - \frac{c}{B} \right)}{K \times \tan \delta} \left(1 - e^{\frac{-K \times \tan(\delta) \times h}{B}} \right) \quad (3.3)$$

where

B = half width of the opening [m]
 h = height of soil cover over the opening [m]
 γ = soil density [kN/m^3]
 c = soil cohesion [kN/m^2]
 ϕ = soil angle of friction [$^\circ$]
 K = coefficient of soil pressure [-]

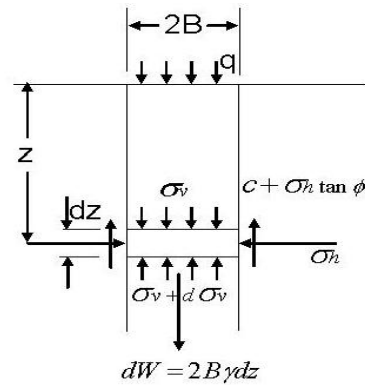
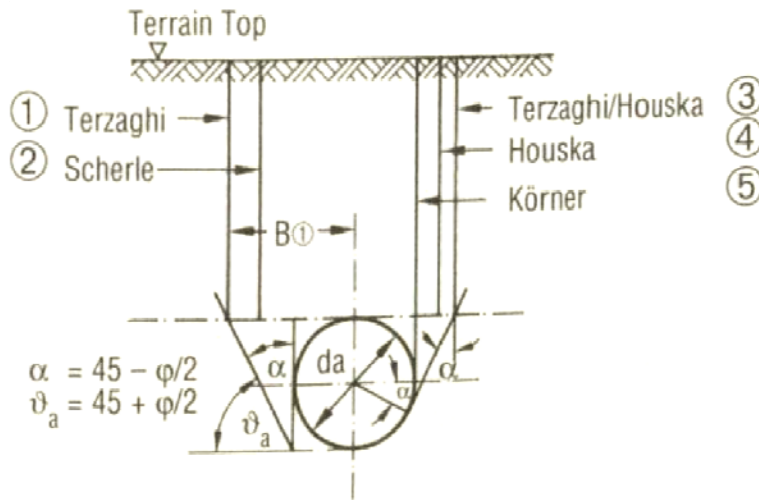


Figure 3-3 Terzaghi's Trap Door Model
 (Staheli, 2006)

Many researchers have used Terzaghi's theory and applied it to calculate the normal stress acting on the pipe during pipe jacking operations. The variations in the methods are primarily in the choice of the width of Terzaghi's Trap Door, $2B$, and how that is applied to pipe jacking, correlating the Trap Door width to the pipe diameter (Figure 3-4).



1. $B = da(0.5 + \tan \alpha)$
2. $B = \frac{da}{2}(1 + \tan \alpha)$
3. $B = \frac{da}{2}(\sec \alpha + \tan \alpha)$
4. $B = \frac{da}{2}(1 + \sec \alpha^* + \tan \alpha^*)$; $\alpha^* = 45 - \vartheta/2$; $\tan \varphi^* = \frac{2}{3} \tan \varphi$
5. $B = \frac{da}{2}(0.5 + \tan \alpha)$

Figure 3-4 Terzaghi's Arching Theory by a Variety of Researchers for the
 Calculation of Normal Stresses (Stein, 1989)

Cavity Contraction Model

The other model is the cavity contraction model introduced by Atkinson and Potts (1977). They found that during tunneling excavation, a considerable stress distribution occurred. In this model, the failure envelope is defined by the angle 2ψ , where ψ is the dilation angle of the soil (Figure 3-5). The cavity contraction model is a good representation of actual conditions in granular material, as the stress path at the tunnel crown reaches failure at the predicted failure envelope. By using the same concept, comparing and compiling different results of case histories of actual projects and findings of previous researchers in the laboratory and testing, Staheli (2006) concluded:

- Evaluation of normal stresses indicates that predictive models previously proposed (see Table 3-2) over-predict normal stresses acting on the pipe.
- A model can be developed to calculate normal stresses above the pipe by focusing on the redistribution of stresses around the pipeline as the pipe shield machine (or TBM) excavates through the soil.

Atkinson and Potts used dilation angle of the soil, as shown in Figure 3-5. However, because the soil is collapsing onto the pipeline, it appears that the soil that would likely remain intact above the pipeline would be above the shear plane of failure (Figure 3-6).

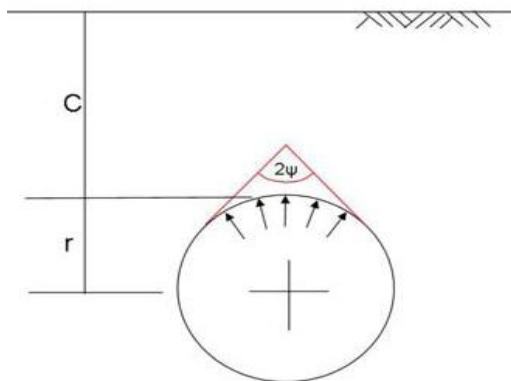


Figure 3-5 Predicted Failure Envelope Based on Cavity Collapse Model (Atkinson and Potts, 1977)

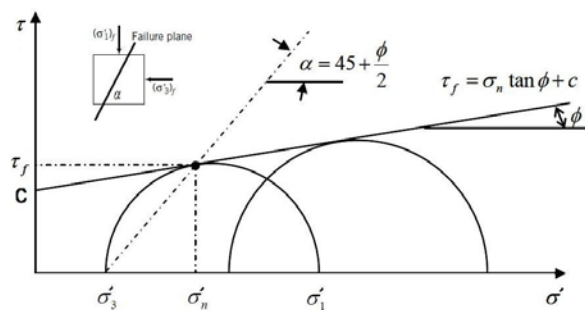


Figure 3-6 Mohr-Coulomb Failure Model (Modified from Das, 2007)

Table 3-2 Existing Predictive Models for Predicting the Frictional Component of Jacking Forces
(Modified from Staheli, 2006)

Model Developer	Frictional Component of Jacking Force (kN)	Symbols and Definition
Helm (1964)	Circular Cross Section: $\mu \cdot \gamma \cdot h \cdot \frac{K_a + 1}{2}$ Rectangular Cross Section: $\mu \cdot \gamma \cdot h \cdot \frac{b_a + K_a \cdot d_a}{b_a + d_a}$	K_a = Active earth pressure coefficient b_a = External width of the microtunneling shield or machine d_a = External height or diameter of microtunneling shield or machine
Walendky / Möncke (1970)	$\gamma \cdot h \cdot \sqrt{\frac{K_o^2 + 1}{2}} \cdot \tan \delta$	$\delta = \phi \div 2$ δ = Wall friction angle K_o = Coefficient of lateral earth pressure at rest h = Cover depth
Szentandrási (1981) Scherle (1977)	$\mu \left[H_w + \frac{d_a}{2} \cdot \frac{K_{Sch} + K_{k1} + K_{k2} + K_{So}}{4} + \frac{W_s - F_a}{4d_a} \right]$	H_w = Effective cover depth W_s = Dead weight of pipe F_a = Buoyancy
Iseki	$\mu(q + w_s) + c$	q = Loading vertical to pipe axis [kN/m ²] $\mu = \tan \frac{\phi}{2}$
Solomo (1979)	Circular Cross Section: $\gamma \cdot \left(h + \frac{d_a}{2} \right) \cdot \sqrt{K_m} \cdot \tan \delta$ Rectangular Cross Section: $\mu \cdot \gamma \cdot h \cdot (b_a + K_m \cdot d_a) \cdot \frac{1 + \frac{d_a}{2h}}{b_a + d_a}$	For a very dense compacted sand. K_m = Effective earth pressure coefficient
Weber (1981)	Circular Cross Section: $\mu \cdot \sqrt{p_v \cdot p_h}$ Rectangular Cross Section: $\mu \left((k \cdot b_a) + (K_o \cdot d_a) + K_o \frac{d_a^2 / 2h}{b_a + d_a} \right)$	Slurry Boring Method $\mu = 0.46$ p_v = Vertical earth pressure p_h = Horizontal earth pressure
Weber (1981)	With Stiffness Modulus from Ohde: $E_s = \vartheta \cdot \left(\frac{\sigma}{\sigma_1} \right)^w$	Auger Boring Method with Steel Pipes (318, 508 and 711-mm diameter): v, w = Stiffness coefficients Δd_a = Deformation dimension of the pipe string
Ebert (1990)	$0.5 \cdot \mu \cdot \left(a \cdot \gamma \cdot h + \left[\gamma \cdot \left(h + \frac{d_a}{2} \right) + p_a \right] \cdot K_e + W_s \right) \cdot \frac{10}{L_R}$	γ = Soil density a = Active load coefficient P_o = Surface loads K_e = Earth pressure coefficient at rest L_R = Pipe length

Continued Table 3-2

Hasan (1985)	$\tan \delta \cdot \left(K_m \cdot \gamma \cdot \frac{h}{2} \cdot \left(1 + \frac{d_a}{2} \cdot K_m \cdot h \right) \right) + \frac{w_s}{4d_a}$					$\tan \delta$ = Coefficient of friction κ_m = Reduction factor
	Jacking ethod	h/d _a	Over cut	Non-Cohesive	Cohesive	
	Open hield	≤ 2	With or Without	$\kappa_m = 1$	$\kappa_m = 1$	
		≥ 2	With or Without	$\kappa_m = (1+\kappa)/2$	$\kappa_m = (1+\kappa)/2$	
	Closed Shield	≤ 2	With	$\kappa_m = 1$	$\kappa_m = (1+\kappa)/2$	
		≤ 2	Without	$\kappa_m = 1$	$\kappa_m = 1$	
≥ 2		With	$\kappa_m = (1+\kappa)/2$	$\kappa_m = (1+\kappa)/2$		
Herzog (1996)	$\mu \cdot \gamma \cdot \left(h + \frac{d_a}{2} \right) \cdot \frac{K_o + 1}{2}$					
Chapman (1999)	$a + 3.8d_a$					Based on Statistical Evaluation of 198 Slurry Microtunneling Projects. a= 1.53 for clay a= 2.43 for sand a= 3.43 for sand/gravel
Bennett (1998)	$F_r = C_a \gamma' d_p \tan(C_f \cdot \phi_r) A_p \cdot L$					γ' = Effective soil unit weight, d _p = Pipe diameter; ϕ_r = Residual soil friction angle, A _p = Pipe circumference, L = Length of tunnel. C _a = Arching reduction factor C _f = Friction reduction factor
Osumi (2000)	$f_o = \beta(\pi B_c q + W) \mu' + \pi B_c C'$ C' = Adhesion of Pipe and Earth (8kN/m ² for N<10 and 5kN/m ² for N>10)					β = Jacking force reduction factor B _c = Diameter of the pipe q = Normal force W = Pipe weight

Based on the observations of the shear planes of failure above the pipeline during over excavations and Mohr-coulomb failure criteria, Staheli proposed the interpretation of the area over the pipeline that the vertical loading is developed. She introduced the factor B* (Figure 3-7) and replaced B in Terzaghi's Equation 3.3. She found that for a cohesionless soil, the vertical stress was independent of depth, and presented the following equation:

$$P_{Ev} = \frac{B\gamma}{K \times \tan \delta} \quad (3.4)$$

Results of the other investigations, (Norris, 1992, and Marshal, 1998) show that average normal stresses are fairly constant and generally evenly distributed around the pipeline unless sharp steering corrections are made or the machine encounters hard soil on one side of the pipe. With the findings of the distributions of normal stresses based on Milligan et al (1998), Staheli proposed the following predictive model for calculating the frictional component of jacking forces:

$$JF_{frict} = \mu_{int} \frac{\gamma \cdot r \cdot \cos(45 + \frac{\phi_r}{2})}{\tan \phi_r} \cdot \pi \cdot d \cdot l \quad (3.5)$$

where:

- JF_{frict} = Frictional Component of Jacking Force [kN]
- μ_{int} = Pipe-Soil Residual Interface Friction Coefficient [-]
- γ = Total Unit Weight of the Soil [kN/m³]
- ϕ_r = Residual Friction Angle of the Soil [degrees]
- d = Pipe Diameter [m]
- r = Pipe Radius [m]
- l = Length of the Pipe [m]

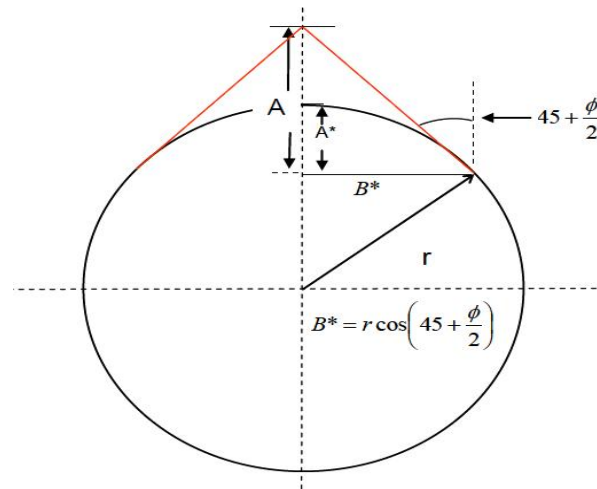


Figure 3-7 B* Factor for use in Vertical Stress Calculations (Staheli, 2006)

Bennett's Predictive Model

Bennett (1998) developed a model for predicting jacking forces in both cohesive and granular soils based on a series of field tests conducted at the US Army of Engineers which were instrumented and constructed with six soil types, and two microtunneling machines

(Figure 3-8). To introduce his experimental model, data for 39 other projects were considered. Bennett's model is based on the concept that the total jacking force is related to surface area of the pipe multiplied by a normal force and a friction coefficient. He concluded that the normal force is a function of pipe diameter and the effective unit weight of the soil, and that the depth of pipe dose not have an effect on it. Bennett introduces two factors, C_a and C_f , which he called the arching reduction factors. Bennett established upper bound, best-fit, and lower bound coefficients for the arching and friction reduction factors (see Equation 3.6).

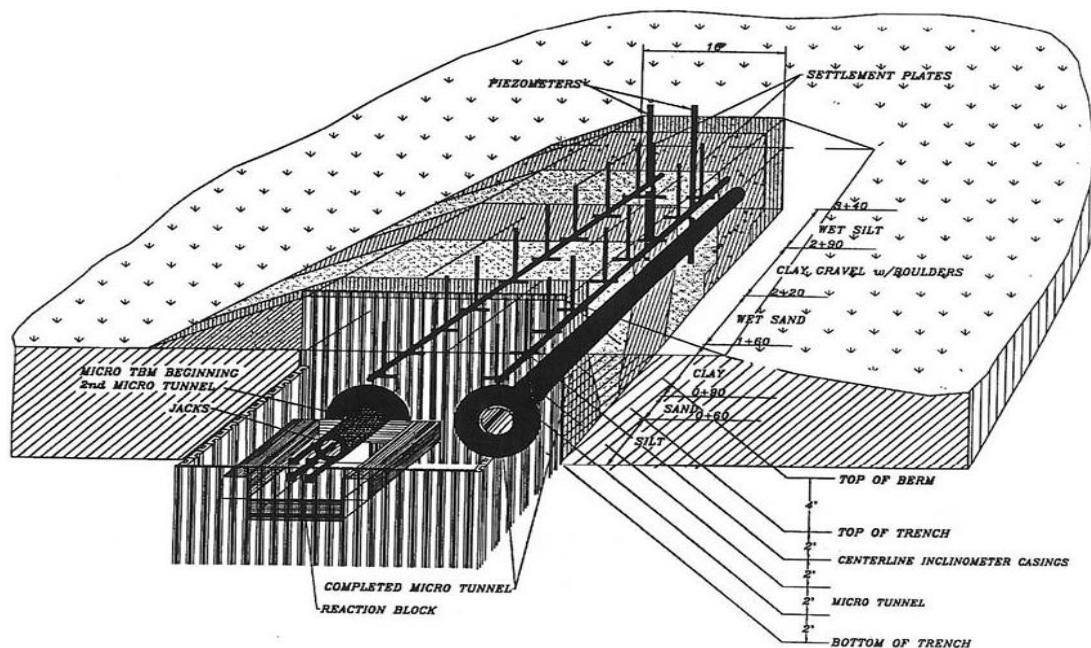


Figure 3-8 Microtunneling Test Bed for Controlled Field Test
(Bennett, 1998)

Further, Bennett separates his recommendations for the arching and friction reduction factors based on whether the microtunnel is in sand or clay and in whether in lubricated or unlubricated conditions. These factors are summarized in Table 3-3. Bennett also analyzed the effects of steering corrections, delays, face pressures, loss of face stability, overcut and lubrication. To account for the impact of delays, steering corrections and misalignment effects, he recommended increasing the jacking force by 1/3.

In Bennett's model, the skin friction component is evaluated using the relationship:

$$F_r = \sigma'_n \mu A_c \quad (3.6)$$

where

σ'_n : Average effective normal stress = $C_a \gamma' D$ [kN/m²]

C_a : Arching Factor (Table 3-3)

μ : Residual friction factor = $\tan(C_f \phi)$

C_f : Friction Reduction Factor (Table 3-3)

A_c : Unit length pipe's surface [m²]

Table 3-3 Arching and Friction Reduction Factors in Bennett Model (1998)

Bennett's Model for Calculation of Frictional Jacking Forces				
	Initial Dewatered, Non-Lubricated Interval		Lubricated Non-Dewatered Interval	
	Arching Reduction Factor C_a	Friction Reduction Factor C_f	Arching Reduction Factor C_a	Friction Reduction Factor C_f
SANDS				
Upper Bound	1.5	1.0	1.0	0.66
Best Fit	1.0	1.0	0.66	0.66
Lower Bound	0.75	1.0	0.5	0.5
STIFF TO HARD CLAY				
Upper Bound	1.0	1.0	0.66	0.66
Best Fit	0.66	1.0	0.5	0.5
Lower Bound	0.33	0.66	0.5	0.5
SOFT TO MEDIUM CLAY				
Upper Bound	1.0	1.0	3.0	1.0
Best Fit	0.66	1.0	1.5	1.0
Lower Bound	0.5	1.0	1.0	0.5

3.2.1.1.3 Effects of Lubrication in Jacking Forces

The skin friction resistance is the main portion of jacking force, which is important in making several critical decisions, as mentioned earlier. Frictional forces on the pipeline may be reduced by applying a lubricant under a nominal pressure above that of the ground water pressure. Figure 3-9 illustrates lubrication viscosity, penetration into the void space around the pipe, and the port injection points.

If high frictional resistance is anticipated which may exceed the available capacity of the jacking frame, thrust block and pipe thickness after lubrication, it is recommended that intermediate jacking stations be placed at regular intervals in the pipeline.

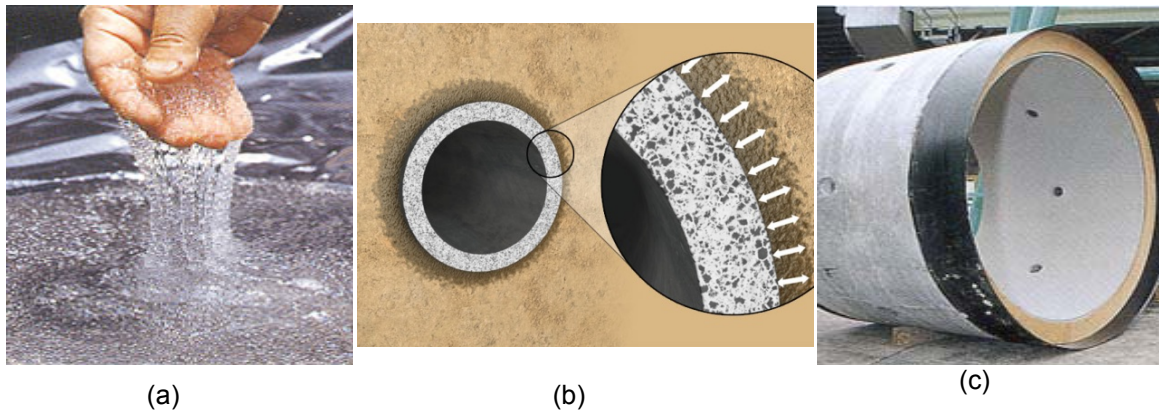


Figure 3-9 (a) Viscosity, (b) Lubrication Penetration (c) Ports in Pipe
(Stein & Partner GmbH, 2011)

There are a number of lubrication materials that are available for use with microtunneling and pipe jacking systems, including polymers and bentonite. However, bentonite is the most commonly used material due to proven effectiveness, ease of availability, and low cost. Usually lubrication is pumped from the tail section of the open-shield tunneling machine, and from each 10 to 20-ft (3 to 6-m) pipe segment back to the pipe. Therefore, the distribution of lubrication over the pipeline is gradual as the pipe jacking process continues forward. Lubrication port can be inserted along the pipeline at any circumferential location.

Based on several case histories, Staheli (2006) observed that the lubricated interface friction coefficient, $\mu_{\text{int.lube}}$, is about 10% of the non-lubricated interface friction coefficient (90% reduction compared with a non-lubricated pipe) (Figure 3-10). However, for design purposes, Najafi (2005) and ASCE 27 recommend 30-50% reduction factor due to lubrication in jacking force calculations.

3.2.1.1.4 Overcut

The space between the maximum excavated diameter and the outer diameter of the installed pipe sections is referred to as overcut or annular space. Overcuts of between 1 to 2-in. (20 to 50-mm) on the diameter [i.e., 10 to 25-mm on the radius] are typical. The overcut is necessary to reduce frictional forces, facilitate steering of the tunneling machine, and to allow injection of lubrication into the annular space if required.

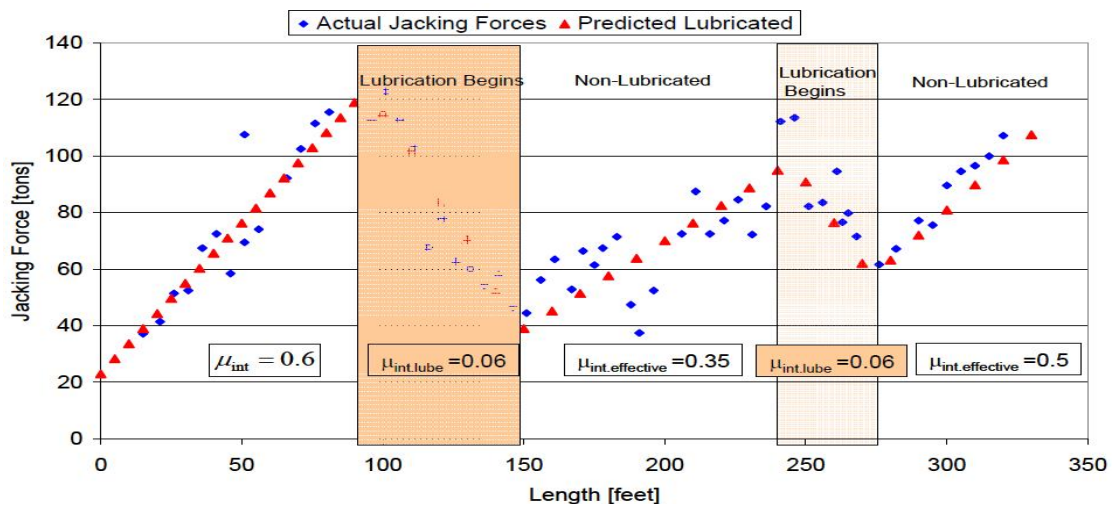


Figure 3-10 Length vs. Actual and Predicted Lubricated Jacking Forces through 340-ft (114-m) for the Clearview Snohomish River Crossing (Staheli, 2006)

3.2.1.2 Jacking Resistance Due to Face Pressure

There are guidelines for calculating the face stability pressure as the shield machine penetrates through the ground (Najafi, 2005, Bennett, 1998, and German Code). The face pressure is highly dependent on the production goals and allowable machine torque and is usually a small component of overall jacking forces, but can be significant for large diameter drives under high groundwater pressure. The face stability pressure depends on the depth, geotechnical and hydro-geotechnical conditions of the project (Figure 3-11).

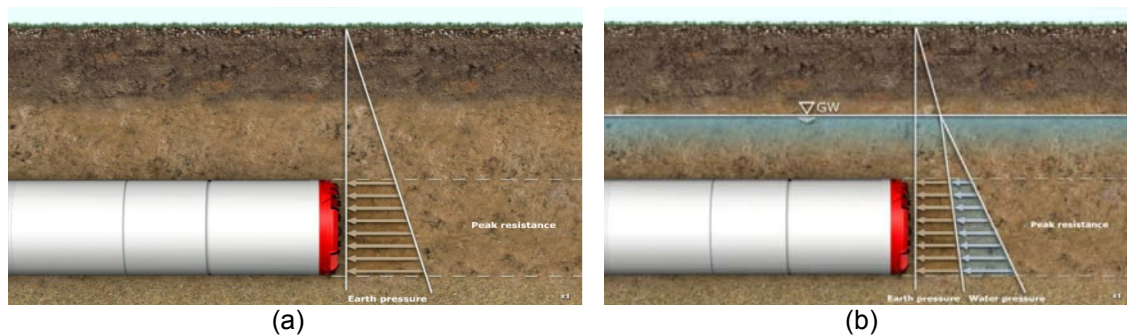


Figure 3-11 Face Pressure Distribution (a) No Water (b) Underwatertable (Stein & Partner GmbH, 2011)

Type of shield, slurry viscosity and pressure, and face pressure are important design factors to prevent instability at the surface. Face pressure should be greater than the active

earth pressure P_a of soil to prevent subsidence at the ground surface and should be less than the passive earth pressure P_p of soil to prevent heaving (Figure 3-12). The optimum value for the face resistance is in the range of the earth pressure at rest P_o .

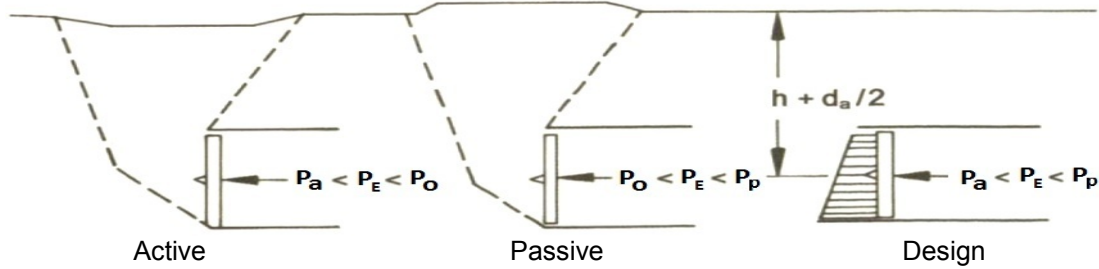


Figure 3-12 Surface Deformation Due to Face Pressure Force
(Modified from Stein, 2003)

The face pressure is calculated based on active and passive earth pressure and ground water. The related equations follow (Das, 2007):

$$\text{Active Earth Pressure [kN/m}^2\text{]: } p_a = \gamma' \cdot h \cdot k_a - 2 \cdot c \cdot \sqrt{k_a} \quad (3.6)$$

$$\text{Passive Earth Pressure [kN/m}^2\text{]: } p_p = \gamma' \cdot h \cdot k_p + 2 \cdot c \cdot \sqrt{k_p} \quad (3.7)$$

$$\text{Groundwater Pressure [kN/m}^2\text{]: } p_w = \gamma_w \cdot h_w \quad (3.8)$$

The total peak pressure force will be governed by the following equation:

$$P_e = \frac{\pi d^2}{4} \left(\frac{(p_a + p_p)}{2} + \gamma_w \cdot h_w \right) \quad (3.9)$$

where:

h = Soil cover depth over the pipeline, [m]

γ' = effective density, [kN/m²]

k_a = active earth pressure coefficient, [-]

k_p = passive earth pressure coefficient, [-]

c = cohesion, [kN/m²]

γ_w = density of water, [kN/m³]

h_w = height of ground water over the crown of the pipeline, [m]

Active and passive coefficients are calculated with the following equations:

$$k_a = \tan^2 \left(45 - \frac{\phi'}{2} \right) \quad (3.10)$$

$$k_p = \tan^2 \left(45 + \frac{\phi'}{2} \right) \quad (3.11)$$

ϕ' = effective internal angle of friction of the soil, [deg]

d = outer diameter of the pipe, [m]

3.2.1.2.1 Silo Theory

As stated previously, ground conditions must be carefully assessed to anticipate possible face collapse, which will cause surface instability (Figure 3-13). Particularly in cohesionless soils below the watertable, soft clays, silts and mixed soils, considerations should

be given to the use grouting, freezing or chemical stabilization of the soil, or the use of earth-pressure balance or slurry support tunneling machines. Figure 3-14 illustrates an example of how a working face is protected mechanically by pressing trapezoidal flaps of steel plate against the soil with hydraulic jack.



Figure 3-13 Excavation Failure During Microtunneling in Fine Sands (Jebelli, 2010)

Computational models to calculate the stress of the face in pipe jacking is frequently based on the effective silo theory. This approach is based on active face slope with a vertical, chimney-shaped limited load (see Figure 3-15).



Figure 3-14 Securing Tunnel Face by Hydraulically Actuating Plate Flap (Schad et.al, 2008)

A safety factor should be considered in estimating face pressures (see force S in Figures 3-15 and 3-16). There are three possibilities to calculate face pressure as follow (Schad et.al, 2008):

- 1) Sliding method, which can be applied for a variety of boundary conditions (see Figures 3-15 and 3-16),
- 2) Experimental equations for simple conditions (which is out of scope of this thesis),
- 3) Numerical analysis using the finite element method for different boundary conditions (which is out of scope of this thesis).

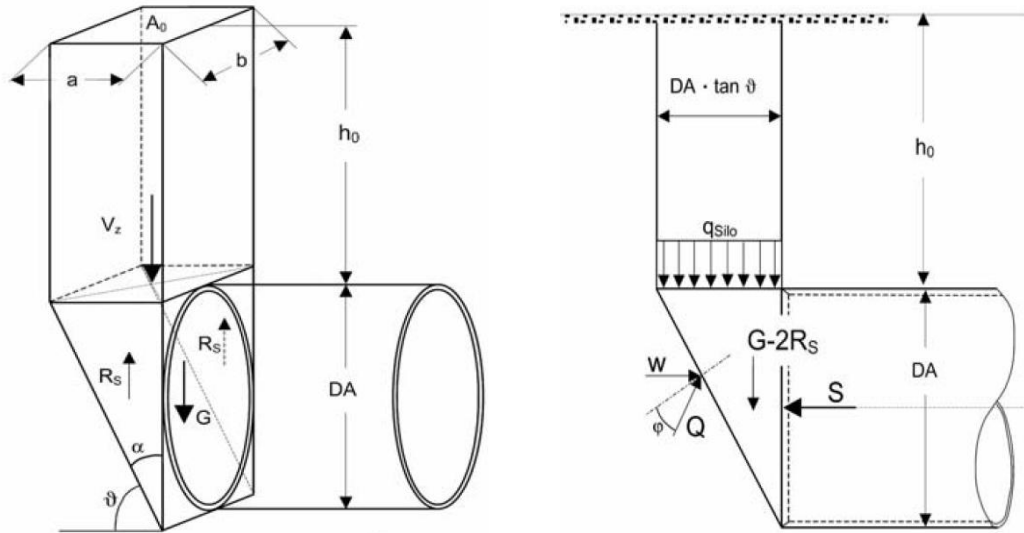


Figure 3-15 Silo Block Method to Determine the Reaction Force (Schad et.al, 2008)

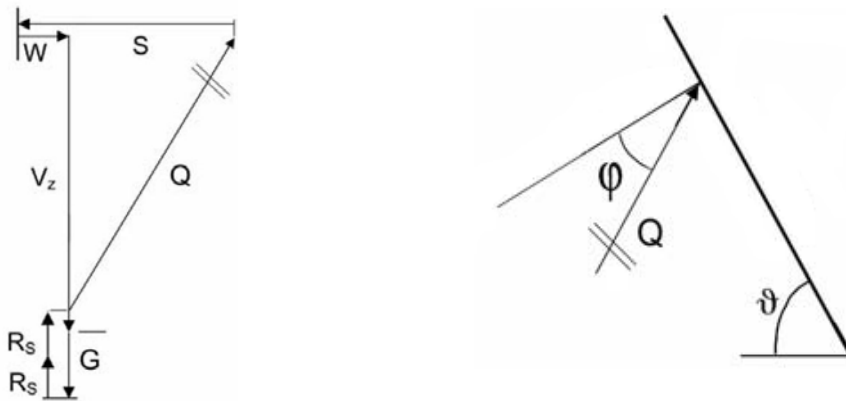


Figure 3-16 Model for the Stress of the Pipe Tunnel Face Due to Soil Pressure (Schad et.al, 2008)

Schad et.al (2008) provide the following equations:

$$\text{Vertical force [kN]: } V_z = q_{\text{Silo}} \cdot DA^2 \cdot \tan(90^\circ - \vartheta) \quad (\vartheta: \text{Sliding surfaces angle}) \quad (3.12)$$

$$\text{Silo pressure [kN/m}^2\text{]: } q_{\text{Silo}} = \frac{A \cdot \gamma^* - U \cdot c}{U \cdot K_h \cdot \tan \varphi'} \left(1 - e^{-h_o \cdot \frac{U}{A} K_h \cdot \tan \varphi'} \right)$$

$$\begin{aligned} \text{Water pressure: } hw \leq DA : W &= \frac{1}{2} \gamma \cdot w \cdot h_w^2 \cdot DA \\ hw \geq DA : W &= \frac{1}{2} \gamma \cdot w \cdot DA [h_w^2 - (h_w - DA)^2] \end{aligned}$$

$$\text{Footprint: } A = a \cdot b = DA^2 \cdot \tan(90^\circ - \vartheta)$$

$$\text{Circumference: } U = 2a + 2b = 2DA [1 + \tan(90^\circ - \vartheta)]$$

$$\text{Cohesion: } c$$

$$\text{High of cover: } h_o$$

$$\text{Weight of soil: } \gamma^* \text{ (Depending on the location of watertable level } \gamma \text{ or } \gamma')$$

$$\text{Height of the water level over the pipe invert } h_w$$

$$\text{Horizontal earth pressure coefficient } K_h \text{ (} 0.75 \leq K_h \leq 1.5 \text{) approximately } K_h = 1$$

The above-mentioned model is used for preliminary calculations of the pressure nozzle, as follows:

$$Q = \frac{V_z + G - 2R_s}{\sin(90^\circ + \varphi - \vartheta)} \quad (3.13)$$

$$S = W + Q \cos(90^\circ + \varphi - \vartheta) \quad (3.14)$$

To determine the maximum vertical force, the angle difference ϑ should be calculated.

A suitable starting value for the iteration of ϑ in most cases is $\vartheta = 45^\circ + \frac{\varphi}{2}$

The safety factor that is included in the calculated reaction force depends on using earth pressure K_h factor. For $K_a \leq K_h \leq K_0$ safety factor is already included. For $K_0 \leq K_h \leq K_p$, the reaction force should be determined based on φ_{mob} and c_{mob} which are (η is safety factor and ≥ 1.3) (Schad et.al, 2008):

$$\tan \varphi_{\text{mob}} = \frac{\tan \varphi}{\eta} \quad c_{\text{mob}} = \frac{c}{\eta} \quad (3.15)$$

3.2.1.3 Effects of Pipeline Misalignment During Pipe Jacking

The effects of misalignment on jacking loads are difficult to assess due to large parameters involved. These parameters include length of pipe, magnitude of jacking force, soil type, lubrication type, slurry type and pressure, overcut, and level of misalignment.

Haslem (1986) attest that it is not possible to model the pipe string as a curve, as small varying angular deviations at each joint produces complex local loading distribution on individual pipe sections (Figure 3-17). Interaction of these parameters creates a multitude of different ground reaction distributions on pipe while it passes through the ground. Thus, some experimental equations for exceptional cases have been developed (Stein, 2005). Bennett (1998) recommends increasing the calculated pipe jacking force based on friction by 30% to cover this issue. Misalignment creates two problems, increasing the total jacking loads, and creating concentration loads on the pipe joints, which are explained in more detail in the following sections.

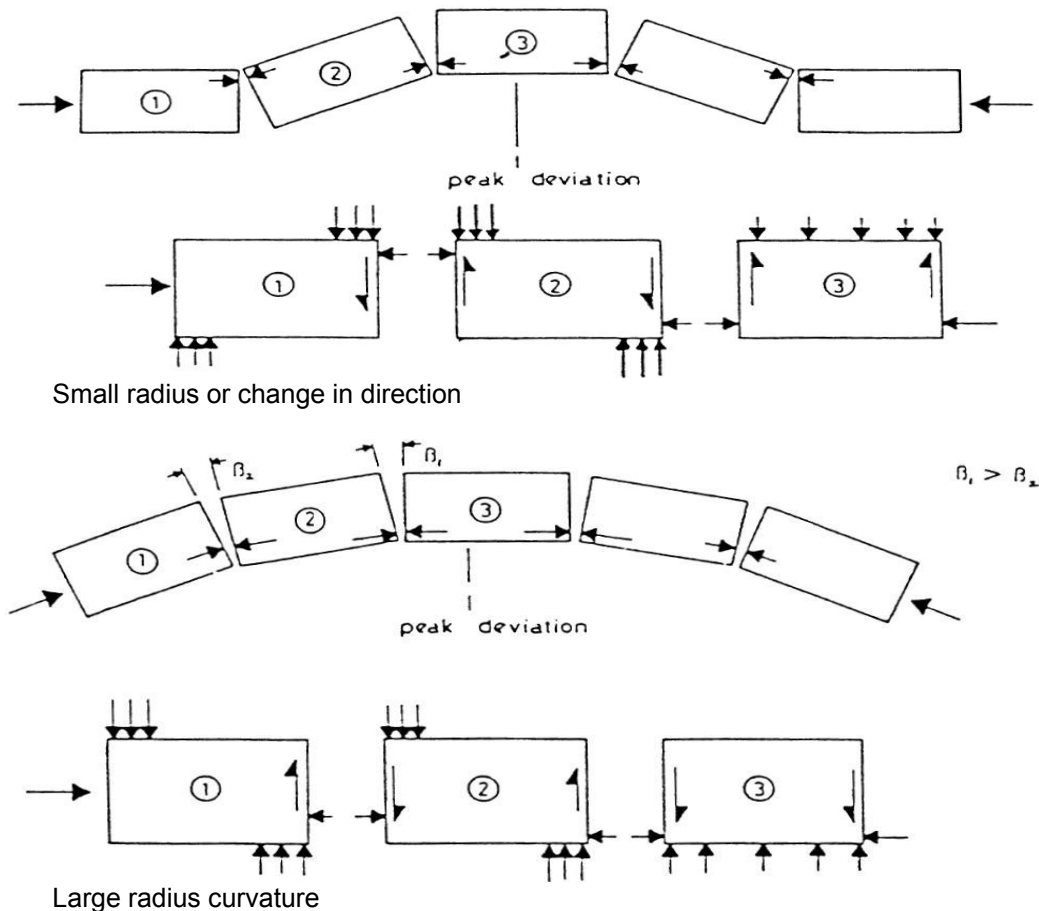


Figure 3-17 Theoretical Misalignment Forces
(Norris, 1992)

3.2.1.3.1 The Effects in Jacking Load- Additional Jacking Resistances

Misalignment means there is angular deviation between the central axes of successive pipes. The misalignment angle, β , is defined in Figure (3-18). In an ideal pipe jacking operation, there is no such deviation, but in practice, irregularities in ground conditions, excavation methods, and operator error can cause front shield deviation from ideal path. To maintain line and grade accurately, steering corrections are continuously made, resulting in the pipe string following a zigzag pattern. The effects of repeated steering errors or corrections make the condition worse, because they lead to a disproportionate increase of the jacking forces (up to a point to stop the project). Figure 3-19 illustrates this multiple misalignment.

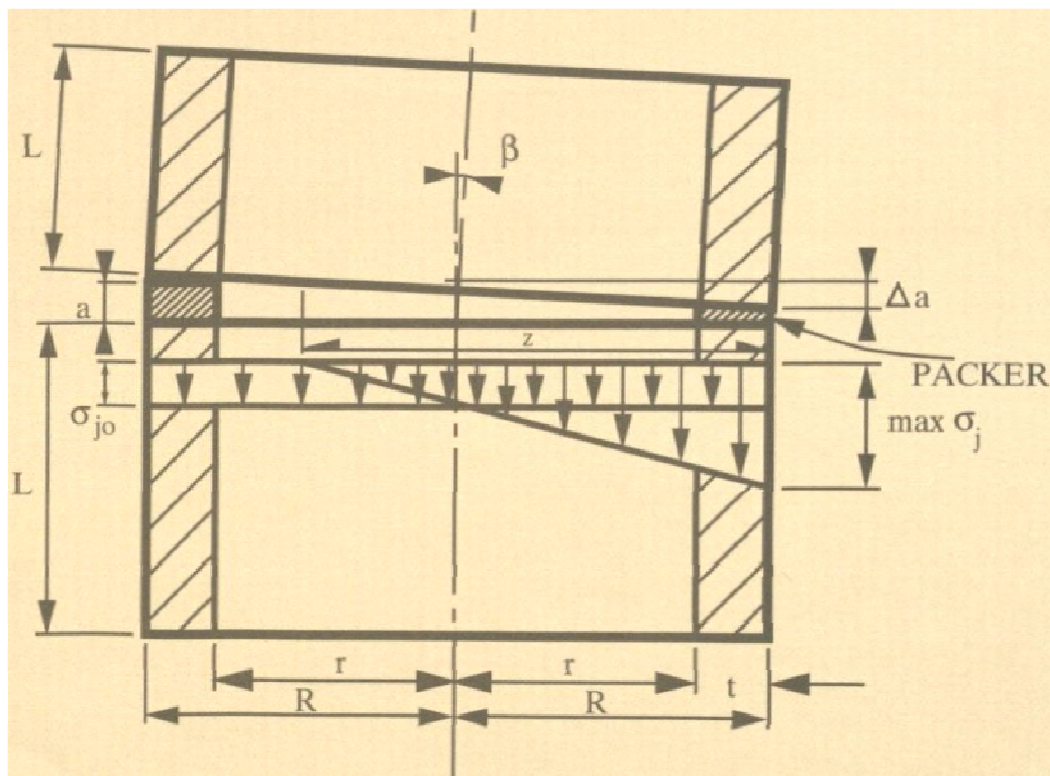


Figure 3-18 Effects of Misalignment in Pipe Joint and Its Consequences in Jacking Forces Distribution in a Linear Joint Stress Model (PJA, 1995)

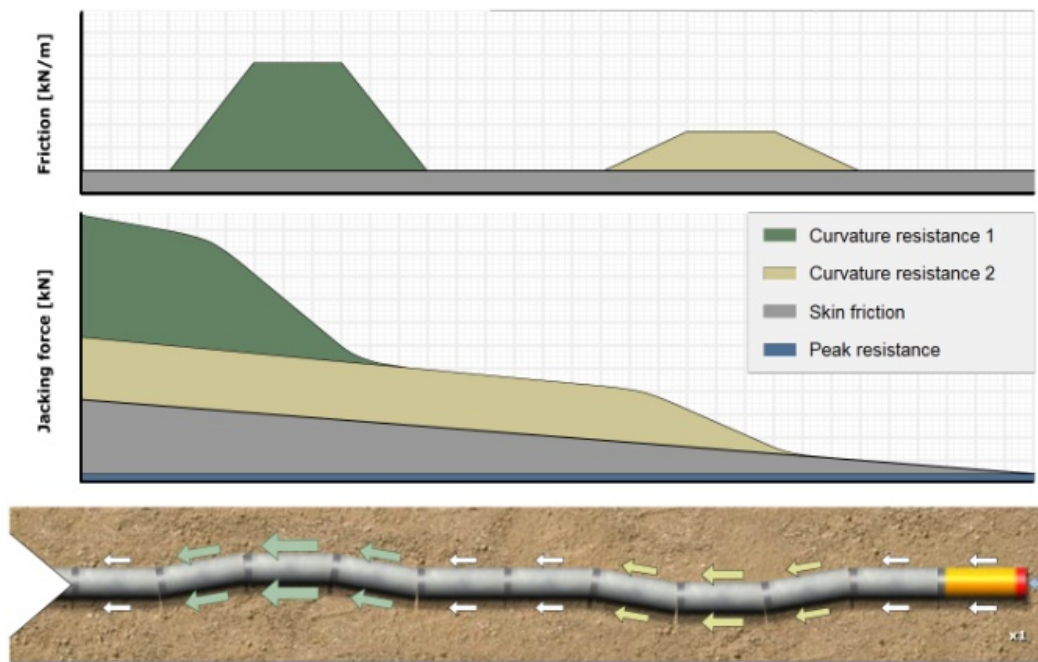


Figure 3-19 Cumulative Increase in Jacking Force Due to Friction and Misalignment
(Stein & Partner GmbH, 2011)

3.2.1.3.2 The Effects on Jacking Pipe

The critical situation for jacking pipes usually arises from edge and diagonal loadings when angular deviation resulting from a zigzag path, becomes larger than allowable bearing stress of the pipe and the pipe joint. The measurement of actual angular deviations achieved in practice should be limited to a reasonable value in terms of misalignment angles and line and level tolerances.

As said earlier, to uniformly distribute the jacking forces at pipe joints, and specifically during angular deflections, pressure transfer rings made of plywood or wooden materials are inserted. Wooden ring stress and strain behavior is non-linear on the resulting stress level. A high proportion of plastic deformation causes permanent deformation and an increased hardening of the pressure transfer to ring during the jacking process (CPAA, 2008).

The hardening of the pressure transfer ring is often the main cause for joint damage that occurs during the jacking process. This is frequently due to spalling of concrete pipe at the

outer end faces of the pipe, which are often not visible from the inside and remain unnoticed at the project's final inspection (CPAA, 2008). To calculate concentrated load due to misalignment, some guidelines (e.g., CPAA, 2008) recommend three times capacity of pipe barrel for the pipe joint. The Australian pipe jacking guidelines provide an accurate concentration force estimation method, which is presented in Section 3.3.5 of this thesis.

3.2.2 Determination of the Thrust Block Capacity

The jacking forces are transferred by the jacking cylinders to the thrust block (Figure 3-20). Usually a thick steel plate is used over the concrete thrust block to distribute the jacking loads. The soil stresses are the result of the size of the jacking force, the shaft depth, the soil properties, and the geometric dimensions of the thrust block. If the jacking load is exceeded beyond the passive earth pressure, due to less soil support at the top of the thrust block, it will cause the thrust block to rotate towards the top. The tilting result of the jacking cylinders can lead to misalignment of the pipe jacking operation, incorrect loading of the jacking pipes, and buckling of the jacking cylinders and the jacking frame.



Figure 3-20 Steel Plate Lined Thrust Block
(The City and County of Honolulu, 2009)

For road crossings, if the pipe jacking project takes place near ground surface (at shallow depths), due to absence of a deep shaft, the abutment must be designed to be a free standing support system (Figure 3-21). The jacking force is then transferred into thrust block, and in turn to supporting braced frame, and driven piles in the soil. Figure 3-22 illustrates another system, which jacking loads are transferred to the guide rails, and then transferred through the guide rails to the thrust block.



Figure 3-21 Thrust Block, Concrete Pad and Headwall for (2) 120-in. (3,040-mm) Tunnels for the City of Fort Collins, Colorado (BT Construction Website, 2011)



Figure 3-22 Rail Traction System (Akkerman Inc., 2011)

Equation 3.16 describes a special case of thrust block supported by the soil, without shaft wall protection system or lining. In this case, the surface soil pressure p behind the thrust block, which results from jacking force, can be calculated, see Figure 3-23 (Stein, 2005).

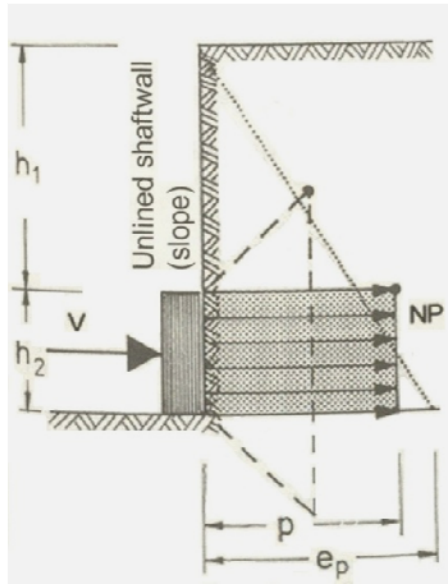


Figure 3-23 Stress Distribution Resulting from the Jacking Force, for Unlined Shafts (Stein, 2005)

$$p = \frac{V}{b * h_2} \quad (3.16)$$

where: p = Soil Pressure [kN/m²]
 V = Jacking force [kN]
 h_2 = Height of the thrust block [m]
 b = Width of the thrust block [m]

In case of thrust block supported by sheet piling or some other types of shaft wall protection system, the soil stresses can be reduced because of the stiffness of the lining wall. The soil stress distribution in this case can vary depending on the geometry of the shaft, the type of lining, the design of the braces (such as walers) and tie back anchors, the geological and hydro-geological conditions as well as the size and points of application of the jacking force. In case of concrete lined or sheet piled driving shafts, without braces or anchors the reduced soil pressure, p_{red} , can be calculated based on the following formula (Stein, 2005). Stress distribution in this case, is illustrated in Figure 3-24.

$$p_{red} = \left(\frac{2h_2}{h_1 + 2h_2 + h_3} \right) \cdot p \quad (3.17)$$

where: P_{red} = Reduced soil stress [kN/m^2]
 h_1 = Height of upper edge of abutment to the ground surface [m]
 h_2 = Height of the abutment [m]
 h_3 = Embedment depth [m]

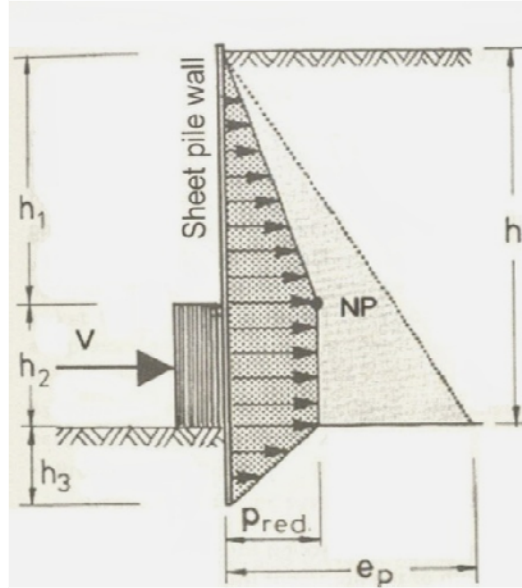


Figure 3-24 Stress Distribution Resulting from the Jacking Force for Lined Shafts (Stein, 2005)

The permissible soil stress depends on passive earth pressure behind the thrust wall.

This can be calculated:

$$e_p = K_p \cdot \gamma \cdot h \quad [\text{kN/m}^2] \quad (3.18)$$

where K_p = Coefficient of the passive earth pressure [-]
 γ = Density of the soil [kN/m^3]
 h = Shaft depth [m]

In the determination of the passive earth pressure K_p a wall friction angle between lining wall and subsoil is not considered. Thus allowable jacking force can be calculated from following equations (Stein, 2005):

Unlined shafts:

$$V_{perm} = \frac{K_p \cdot \gamma \cdot h}{\eta} \cdot b \cdot h_2 \quad [\text{kN}] \quad (3.19)$$

Lined shafts:

$$V_{perm} = \frac{K_p \cdot \gamma \cdot h}{2 \cdot \eta} \cdot b \cdot (h_1 + 2 \cdot h_2 + h_3) \quad [\text{kN}] \quad (3.20)$$

$$K_p = \text{Rankine passive earth pressure coefficient} \quad k_p = \tan^2 \left(45 + \frac{\phi'}{2} \right) \quad (3.21)$$

$\eta = 1.3$ safety factor

A more accurate estimating of thrust block capacity in lined shafts is presented by Najafi (2005). This model provides the basic passive earth pressure for soil directly behind the thrust block by utilizing frictional resistance from the angle of internal friction and cohesion components of the soil. The soil above the wall is converted to a uniform load applied to the top of the soil behind the wall, as shown in Figure 3-25. When this transformation is made, the frictional resistance from this transformed soil can no longer be considered. Thus, the coefficient, α , is applied to account for the actual resistance conditions.

$$Q = \alpha * B * \left[\gamma \cdot H^2 \cdot \left(\frac{K_p}{2} \right) + 2CH\sqrt{K_p} + \gamma HhK_p \right] \quad (3.22)$$

Where

- Q = allowable thrusting force, [kN, tons]
- α = coefficient =2 (commonly set at 2)
- B = thrust block or abutment width, [m]
- γ = soil density, [kN/m³]
- K_p = coefficient of passive soil pressure, [-]
- ϕ = angle of internal friction, [°]
- C = soil cohesion, [kN/m²]
- H = thrust block height, [m]
- h = distance to top of thrust block from ground surface, [m]
- $q_s = \gamma \cdot h$, [kN/m²]

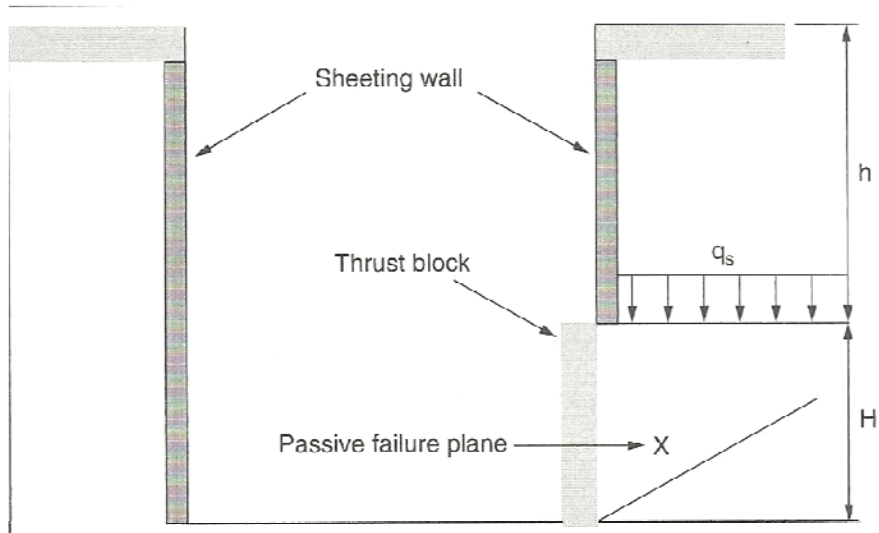


Figure 3-25 Calculation of Thrust Block Capacity
(Najafi, 2005)

If the shaft cannot accommodate the induced jacking forces, then the jacking force must be reduced. Possible solution is increasing the passive earth resistance by means of soil stabilization behind the shaft wall, such as soil injection, see Table 3-4.

Table 3-4 Possible Soil Injection Applications Based on Soil Types
(Modified from Stein, 2005)

Cavities in	Water permeability coefficient $k[m/s]$	Grout	Injection purpose (S = Sealing, C=Consolidation)
Gravel Coarse sand Gravel, sandy	$>5 \cdot 10^{-3}$	Cement suspension Clay Cement Suspension Clay Cement Suspension and Silicate gel	C S, C S, C
Sand Sand silty	$5 \cdot 10^{-3}$ to $5 \cdot 10^{-6}$	Silicate gel Synthetic resin	S, C S, C
Fine sand Coarse silt	$5 \cdot 10^{-4}$ to $5 \cdot 10^{-7}$	Silicate gel Synthetic resin	S S

3.2.3 Work Stoppage Effects

In clays with high plasticity, work stoppage may result in significant increase in the initial jacking load to advance the pipe string after the work stoppage. The work stoppage is less significant in stiff, low plasticity clays, and negligible in soft clay and cohesionless soils (Marshal, 1998).

3.3 Jacking Load Standards

In this section, an investigation of different pipe jacking standards is presented. Next chapter presents a comparison of this investigation. The guidelines by ASCE 27 are considered as the baseline.

3.3.1 ASCE 27 Standard Practice for Direct Design of Precast Concrete Pipe for Jacking in Trenchless Construction²

3.3.1.1 Materials (ASCE 27-11.0)

3.3.1.1.1 Concrete (ASCE 27-11.1)

Concrete shall conform to the requirements of ASCE Practice 15. The minimum concrete compressive strength shall be 5,000 psi (34.5 MPa).

² Sections 3.3.1 and 3.3.2 are excerpted from ASCE 27 without any changes.

3.3.1.1.2 Reinforcement (ASCE 27-11.2)

Reinforcement shall consist of cold-drawn steel wire conforming to ASTM Specification A 82 or ASTM Specification A 496, or of cold-drawn steel welded wire fabric conforming to ASTM Specification A185 or ASTM Specification A497, or of hot rolled steel bars conforming to ASTM Specification A 615.

The use of cold-drawn steel or cold-drawn steel welded wire fabric design strengths exceeding ASTM Specification values may be approved by the owner when the reinforcing manufacturer's mill test report certifies a higher minimum yield, and ultimate strength steel is being provided. The other requirements of the appropriate ASTM specifications shall be met by the higher minimum strength steels. The yield strength shall not be taken greater than 86% of the ultimate strength, or 80 ksi (560 MPa), whichever is lower.

ASCE 27 Section 2.1 does not apply to wire sizes with nominal diameters of less than 0.080 -in. (2-mm) or nominal cross-sectional areas of less than 0.005-in.² (3-mm²). The use of ASTM Specification Grade 40 hot rolled steel bars with strengths exceeding ASTM specification values may be approved by the owner when the reinforcement manufacturer's mill test report certifies that higher minimum yield and ultimate (tensile) strength steel is being provided. The allowable combinations of increased yield/ultimate strength, in ksi (MPa), shall be 45/75 (310/520), 50/80 (350/560), and 55/85 (380/590). The other requirements of ASTM Specification A 615 shall be met by the higher minimum strength steel.

3.3.1.1.3 Joint Cushioning (ASCE 27-11.3)

The contact surfaces of all pipe joints that transmit the axial (longitudinal) jacking forces shall be separated by a packing (cushion) of plywood with a minimum thickness of 1/2-in. (13-mm) for pipe 36-in. (900-mm) in diameter or smaller and 3/4-in. (19-mm), or another material of equivalent or lesser stiffness that can transmit the axial jacking forces uniformly and without producing significant transverse splitting forces. Common joint configuration are displayed in Figure 3-26

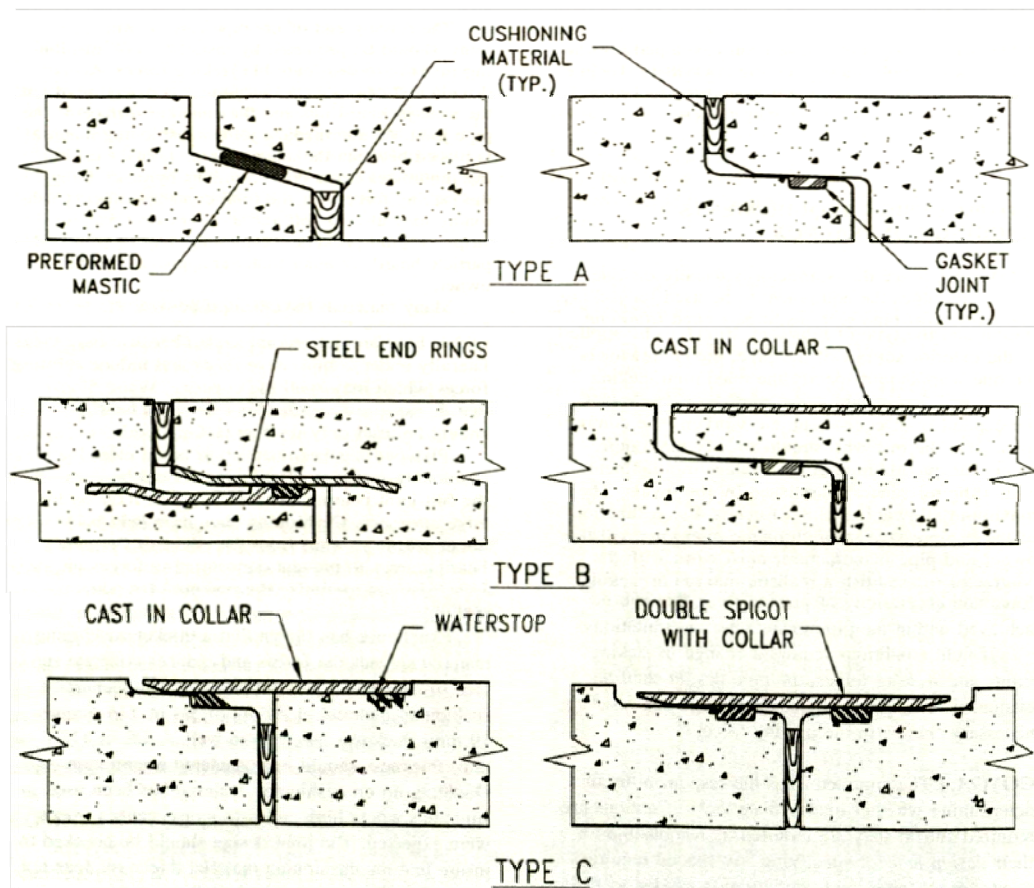


Figure 3-26 Common Joint Configurations
(ASCE 27)

3.3.1.2 Estimating Required Jacking Force

3.3.1.2.1 General (ASCE 27-C1.0)

The resistance that has to be overcome during the jacking operation varies considerably, therefore, only ranges can be estimated. The factors influencing the value of the jacking force are:

- length, alignment, and outside dimensions of the pipeline to be jacked
- weight (mass) of pipe
- height of overburden
- nature of soil and watertable and effects of dewatering
- loads on shield
- whether operation is continuous or interrupted
- size of overbore
- lubrication

3.3.1.2.2 Jacking Resistance (ASCE 27-C2.0)

When the jacking operation is stopped, the resistance builds up very quickly in some soils. Jacking force increases of 20-50% can be expected after delays of as little as 8 hours. Jacking resistance per unit area of external surface ranges from 0.3 to over 6.5 psi (2 to 45 kPa). Typical values for various ground conditions are listed in Table 3-5. It is imperative to have sufficient jacking capacity to cope with the potential for interrupted operations and high jacking resistance.

Table 3-5 Frictional Jacking Resistance for Various Ground Conditions (ASCE 27 C.2.1)

Ground Condition	Resistance, psi of Surface Area	Resistance, kPa of Surface Area
Rock	0.3-0.4	2-3
Boulder clay	0.7-2.6	5-18
Firm clay	0.7-2.9	5-20
Wet clay	1.4-2.2	10-15
Silt	0.7-2.9	5-20
Dry loose sand	3.6-6.5	25-45
Fill	Up to 6.5	Up to 45

3.3.1.3 Lubrication and Coatings (ASCE 27-B8.0)

3.3.1.3.1 Lubrication (ASCE 27-B8.1, C3.1)

Injection of a lubricant into the overcut annulus may be used to reduce friction between the jacking pipe and the soil. This may be accomplished through special ports through the pipe wall or at the shield, or both. Lubrication is generally accomplished with water, as the carrier fluid, mixed with bentonite, polymers, bentonite-polymer mixtures, or other lubricants.

Factors affecting lubricant use and selection include soil type, control of lubricant water loss to surrounding soil, control of soil stability around the pipe, environmental compatibility, and mechanical means of filling the overcut annulus. Lubricant injection pressures should be monitored both at the pump and port locations. The volume of material pumped should be monitored in relation to annulus size with an allowance for loss of lubricant to the surrounding soil. In cohesive soil, a substantial portion of the resistance is ground adhesion, which can be reduced by lubrication. The most commonly used lubricant is bentonite, which is injected through ports in the jacking head and along the pipe wall. Repeated lubrication may reduce the

required jacking force by more than 50%, but more commonly, the average reduction may be about 30%.

3.3.1.3.2 Coatings (ASCE 27-B8.2)

A coating may be applied to the exterior of the pipe to aid in reducing skin friction. Coatings may be pre-applied. Pre-applied coatings include resin based compounds, epoxies, and paints.

3.3.1.4 Shields (ASCE 27-B3.1)

The leading edge of the pipe shall be equipped with a jacking head or shield anchored to prevent wobble of the lead pipe and undue variation in grade or alignment during the jacking operation. The shield located at the face or front of a jacking operation provides protection for the excavation operation and steering control to maintain alignment.

3.3.1.5 Overbore (ASCE 27-B3.3)

The amount of overbore is the difference between the maximum allowable dimension of the outside diameter of the shield and the outside diameter of the pipe being jacked. Generally overbore annulus should be kept to approximately 0.5-in. (13-mm) for diameters less than 36-in. (900-mm) and 1-in. (19-mm) for diameters 36-in. (900-mm) and greater. Over breaks outside of these recommendations may be necessary or desirable depending on soil conditions and contractor's equipment, and should be evaluated for possible effects on jacking loads, earth loads, and surface settlement.

3.3.2 Design Calculation Based on ASCE 27

Pipe thickness control based on jacking forces is dependent on the following:

3.3.2.1 Minimum Thickness (ASCE 27-10.1.3.3)

The minimum design concrete cover over the reinforcement shall be 1-in. (25-mm) in pipe having a wall thickness of 2 1/2-in. (63-mm) or greater and 3/4-in. (19-mm) in pipe having a wall thickness of less than 2 1/2-in. (63-mm).

3.3.2.2 Design for Axial Forces from Jacking (ASCE 27-16.0)

The area of the concrete contact surface for the applied jacking force shall be sufficient to transfer the required maximum jacking force without exceeding the maximum permissible compressive strength on the contact surfaces, f_{p1m} and f_{p2m} .

3.3.2.3 Maximum Permissible Contact Compressive Stress Produced by Jacking Thrust Force (ASCE 27-16.1)

The maximum jacking thrust, P_j , shall not produce longitudinal compressive concrete stresses, f_p , on the joint contact surfaces that exceed the strength limits specified in this section. The maximum concrete compressive stress on the joint contact surface f_{p1m} when the jacking force is uniformly applied (concentric jacking force) as shown in Figure 3-27-a shall not exceed:

$$f_{p1m} = \frac{0.85\phi_j f'_c}{LFJ_1} \quad (3.23)$$

The maximum concrete compressive stress at the point of greatest stress on the joint contact surface f_{p2m} , when the jacking force is applied with eccentricity, as shown in Figure 3-27.b shall not exceed.

$$f_{p2m} = \frac{0.85\phi_j f'_c}{LFJ_2} \quad (3.24)$$

In the above equations parameters are:

ϕ_j = Capacity reduction factor for compression produced by jacking thrust = 0.9

f'_c = Design compressive strength of concrete, lbs/in.² (MPa)

LFJ_1 = Load factor for jacking thrust-concentric load causing uniform stress = 1.5

LFJ_2 = Load factor for jacking thrust-eccentric load causing non-uniform stress = 1.2

3.3.2.4 Design Maximum Permissible Jacking Thrust Force (ASCE 27-16.2)

The maximum jacking thrust for uniform stress (concentric thrust), as shown in Figure 3-27.a is:

$$P_{j1m} = f_{p1m} A_p \quad (3.25)$$

For circular pipe, when the contact surface has the maximum non-uniform stress f_{p2m} , at one point on the edge and zero stress on the opposite edge (Figure 3-27.b), the maximum jacking thrust for non-uniform stress is:

$$P_{jm} = 0.5 f_{p2m} A_p \quad (3.26)$$

where:

A_p =contact area between joint packing and concrete surface with no joint separation,-
in.² (mm²)

For circular pipe, when there is no separation of the contact surface and the contact surface has non uniform compressive stress across the full joint area (Figure 3-27.b), the maximum jacking thrust depends on the average stress across the contact surface, f_{p1} as follows:

(a) when $f_{p1} = f_{p1m}$ and $f_{p1} < f_{p2m}$ the maximum jacking thrust is:

$$P_{jm} = 0.5f_{p1m}A_p \quad (3.27)$$

(b) when $f_{p1} < f_{p1m}$ and $f_{p2} = f_{p2m}$ the maximum jacking thrust is:

$$P_{jm} = f_{p1}A_p \quad (3.28)$$

For circular pipe, when there is separation across a portion of the contact surface, as shown in Figure 3-27.c, stresses become zero in the region of separation, and a special analysis (i.e., 3.32) is required to determine the region of zero contact, the zero contact dimension, Z, the contact surface properties A'_p and the magnitude of f_{p2} , and the jacking force coefficient, k_j The maximum jacking thrust is:

$$P_{jm} = k_j f_{p2m} A'_p \quad (3.29)$$

where

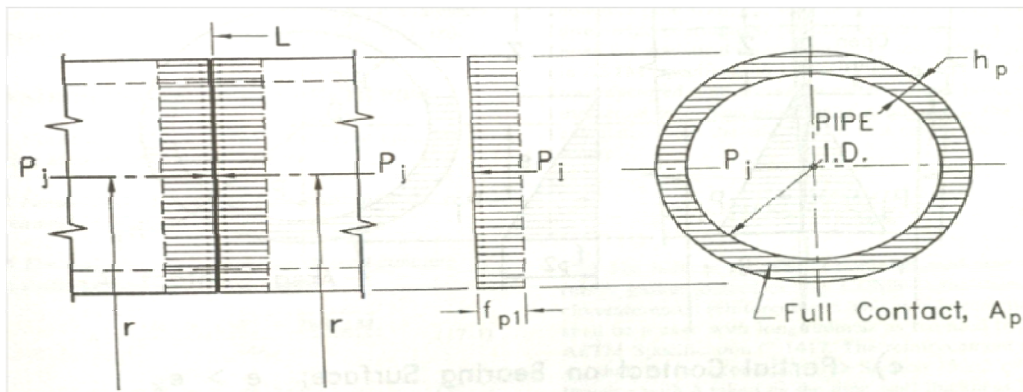
k_j = coefficient for determining maximum eccentricity of jacking thrust application without separation at edge of joint packing with zero stress

3.3.2.5 Design Angular Deviation from Straight Line and Associated Eccentricity of Jacking Thrust Force (ASCE 27-16.3)

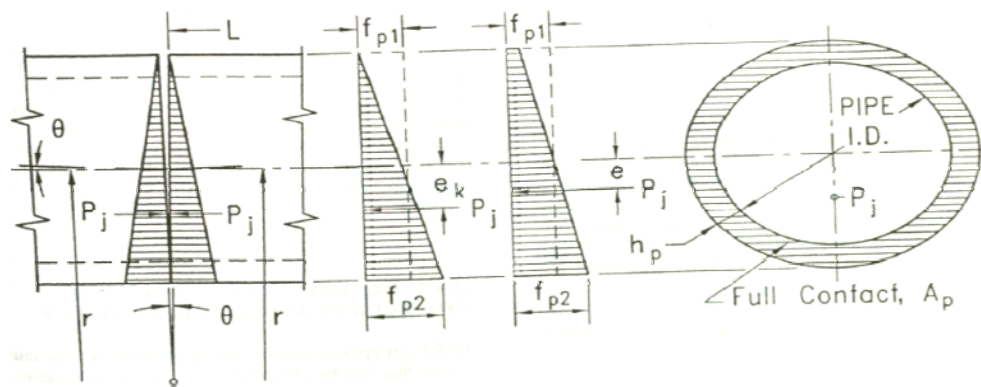
The design angle of deviation from a straight line, θ , is:

$$\tan \theta = \frac{2(f_{p2} - f_{p1})}{D_{po}} \left[\frac{a}{E_p} + \frac{h_p L / h}{E_c} \right] \quad (3.30)$$

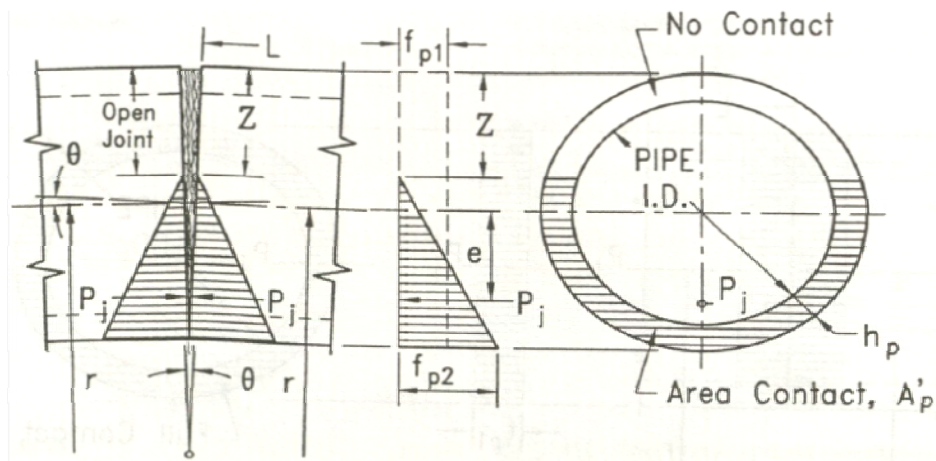
The modulus of elasticity of the joint packing material, E_p , after multiple loadings, should be obtained from tests. In the absence of such information for a specific packing material, an average value of E_p for 3/4-in. (19-mm) nominally thick plywood that has been subjected to multiple loading up to about 0.6 f_c' may be taken as 20,000 psi (138 MPa).



(a)



(b)



(c)

Figure 3-27 Contact Surface a) Full Concentric Contact, b) Full Contact on Bearing Surface; $e < e_k$, c) Partial Contact on Bearing Surface; $e > e_k$ (ASCE 27)

3.3.2.6 ASCE 27 Observations³

An approximate estimate of the radius of curvature that can be obtained for bends using non-uniform contact is (see Figure 3-27.c):

$$r = \frac{(L + a) \cdot (D_{po} - z) / D_{po}}{\tan \theta} \quad (3.31)$$

When there is a contact across the full joint (Figure 3-27.b), $z = 0$

When the joint completely opens, the pipe jacking operation would be unstable, and $r = 0$

In this case, the total jacking force in partial contact on bearing surface can be calculated by following equation ($e > e_k$ Figure 3-27.c):

$$P_i = \frac{D_{po} \cdot h_p \cdot f_{p2}}{1 + \cos \theta_1} \cdot [\cos \theta_1 \cdot (\pi - \theta_1) + \sin \theta_1] \quad (3.32)$$

θ_1 is The angle from crown to the contact area, as follows:

$$\theta_1 = \cos^{-1} \left(\frac{D_{po}/2 - z}{D_{po}/2} \right) \quad (3.33)$$

3.3.3 Pipe Jacking Association (British Code)⁴

In these guidelines, tunnel stability concept for different types of soils has been used to calculate face stability (face resistance) and frictional jacking loads.

3.3.3.1 Face Stability in Cohesive Soils

In cohesive soils, the pressure σ_T required to maintain stability of the tunnel face is given by:

$$\sigma_T > \gamma(H + D_e/2) - T_c \cdot s_u \quad (3.34)$$

Where:

γ = unit weight of soil [kN/m³]

S_u = undrained strength of soil [kN/m²]

T_c = stability ratio - see Figure 3-28.

In pipe jacked tunnels, the unsupported length P is usually small or zero, and $P/D_e=0$.

³ This section is not provided by ASCE-27.

⁴ This section is partially excepted from Pipe Jacking Association (1995).

To prevent surface blowout due to excessive face pressure,

$$\sigma_T < \gamma \left(H + \frac{D_e}{2} \right) + T_c \cdot s_u \quad (3.35)$$

In both cases, a factor of safety of 1.5 to 2.0 on S_u is needed to limit heave and settlement in soft clays.

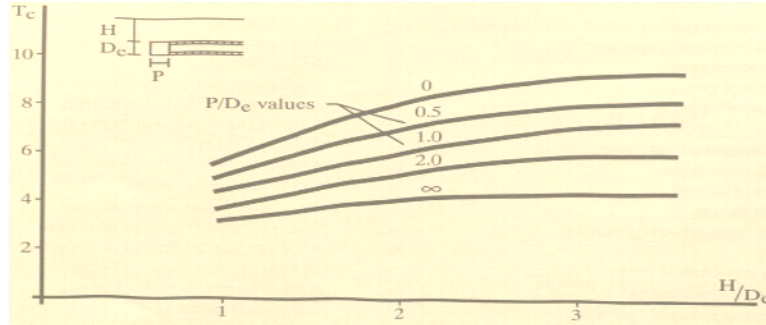


Figure 3-28 Tunnel Stability Ratio
(PJA, 1995)

3.3.3.2 Tunnel Stability

3.3.3.2.1 Tunnel Stability in Cohesive Soils

For the tunnel stability behind the shield, the conditions correspond to the case in Figure 3-28 of $P/D_e \rightarrow \infty$. The equation for calculating the support pressure can be given as:

$$\frac{\sigma_T}{S_u} = \frac{\gamma D_e}{S_u} (H/D_e + 1/2) - T_c \quad (3.36)$$

Values of σ_T less than zero indicate that the tunnel is stable. For most cases in microtunnels $\gamma D_e/S_u \ll 1.0$, the tunnel bore will normally be stable (see Figure 3-29).

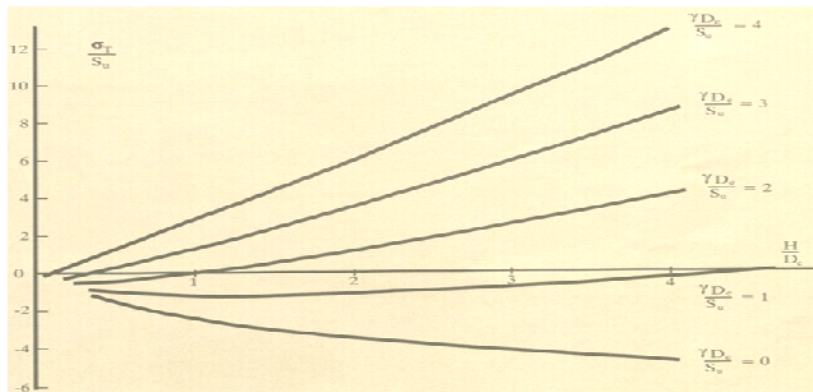


Figure 3-29 Normalized Tunnel Stability Ratio
(PJA, 1995)

3.3.3.2.2 Tunnel Stability in Cohesionless Soils

In cohesionless soil conditions without a surcharge on the surface, the required support pressure is independent of the cover depth, and is given by:

$$\sigma_T = \gamma D_e T_\gamma \quad (3.37)$$

Where T_γ is the stability number given by Figure 3-30, and it is a function of Φ , the friction angle of the soil.

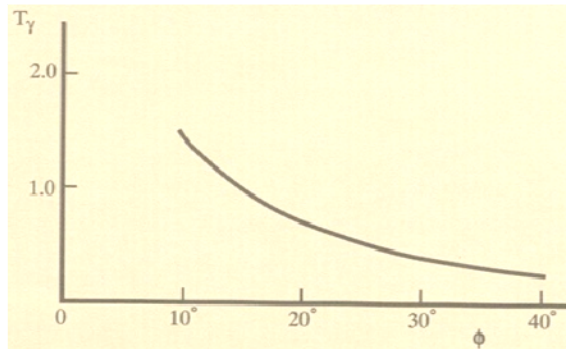


Figure 3-30 Tunnel Stability
(PJA, 1995)

Alternatively, if the tunnel is at shallow depth and a large surcharge σ_s acts on the surface, the weight of soil may be neglected and then,

$$\sigma_T = \sigma_s T_s \quad (3.38)$$

With the stability number T_s as given by Figure 3-31.

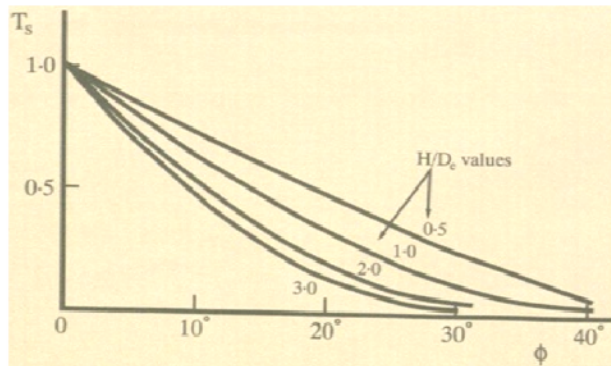


Figure 3-31 Tunnel Stability at Shallow Depth
(PJA, 1995)

Note that both solutions apply to dry soil. Water pressure, if present, must be added to σ_T and the buoyant weight of soil used in the above equation.

3.3.3.3 Friction Jacking Load per Unit Length

3.3.3.3.1 Model for Ground Loading in Cohesive with a Stable Bore (Haslem, 1986)

$$F = \alpha S_u b \text{ [kN/m]} \quad (3.39)$$

Where:

$\alpha.S_u$ is the "adhesion" between the pipe and clay soil;
 b is the contact width between pipe and ground, see Figure 3-32.

$$b = 1.6(P_u k_d C_e)^{\frac{1}{2}} \quad (3.40)$$

where

P_u = contact force per unit length [kN/m]
 E_s = elastic modulus of soil [kN/m²]
 E_p = elastic modulus of the concrete pipe [kN/m²]
 D_e = internal diameter of the cavity [m]
 D_p = external diameter of the pipe [m]

ν_s, ν_p = Poisson's ratios

$$k_d = \frac{D_e D_p}{(D_e - D_p)}$$

$$C_e = \frac{(1-\nu_s^2)}{E_s} + \frac{(1-\nu_p^2)}{E_p}$$

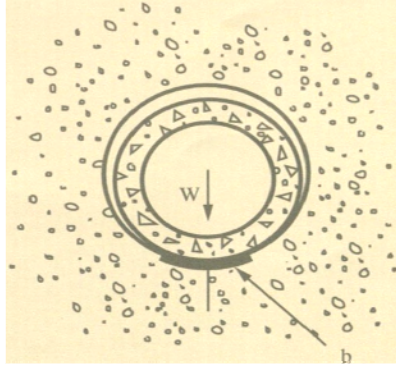


Figure 3-32 Contact Width Between Pipe and Ground
(PJA, 1995)

3.3.3.3.2 Model for Ground Loading in Cohesionless Soil

Based on Terzaghi's theory, the vertical and horizontal stresses on the pipe are according to Figure 3-33.

$$\sigma_v = \frac{\gamma B}{k \tan \phi} \left(1 - e^{-k \tan \phi \frac{H}{B}} \right) \quad (3.41)$$

$$\sigma_h = k(\sigma_v + 0.5\gamma D_e) \quad (3.42)$$

The radial stress around the pipe is
$$\sigma_p = \frac{(\sigma_v + \sigma_h)}{2} + \frac{(\sigma_v - \sigma_h)}{2} \cos 2\theta \quad (3.43)$$

and the total frictional resistance is $F = \frac{\pi D_e}{2} (\sigma_v + \sigma_h) \tan \delta$ (3.44)

Where Φ is the angle of internal friction of the soil, and δ is the angle of friction between the pipe and the soil. When watertable is present at depth H_1 , the expression for σ_v becomes:

$$\sigma'_v = \sigma'_{v_1} e^{-k \tan \phi (H-H_1)/B} + \frac{\gamma' B}{k \tan \phi} (1 - e^{-k \tan \phi (H-H_1)/B}) \quad (3.45)$$

where

$$\sigma'_{v_1} = \sigma_{v_1} = \frac{\gamma B}{k \tan \phi} \left(1 - e^{-\frac{k \tan \phi (H_1)}{B}}\right) \quad (3.46)$$

$$\sigma'_h = k(\sigma'_v + 0.5\gamma' D_e) \quad (3.47)$$

Note that γ is bulk unit weight (above watertable), and γ' is submerged unit weight (below watertable), and

$$B = \frac{D_e}{2} \tan(45^\circ - \phi'/2) + \frac{D_e}{2 \sin(45^\circ + \phi'/2)} \quad (3.48)$$

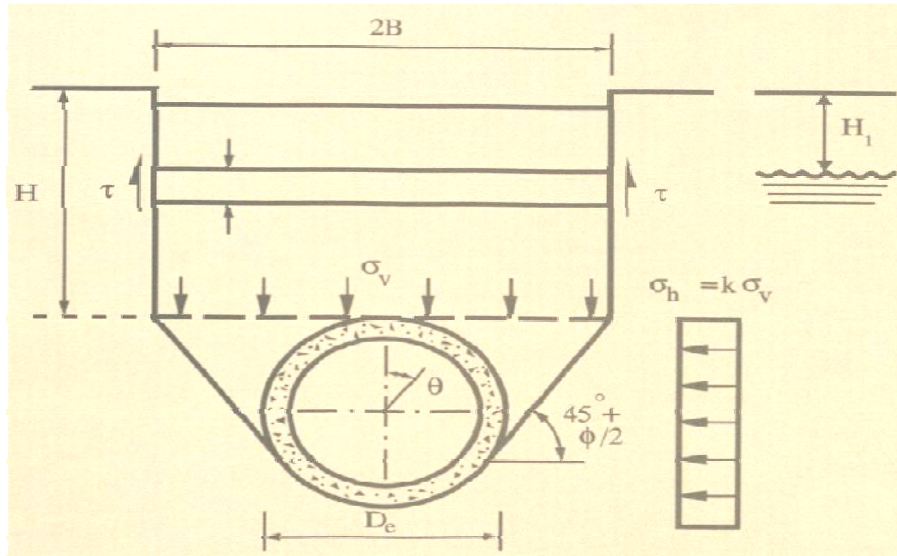


Figure 3-33 Normal Pressure Based on Terzaghi Model (PJA, 1995)

3.3.3.4 Jacking Forces from Linear Joint Stress Model

This part of British code has been borrowed from the Australian Concrete Pipe Association linear stress approach Figure 3-34. Thus the concept will be explained completely there. The only difference is that the allowable jacking load is showed in graph.

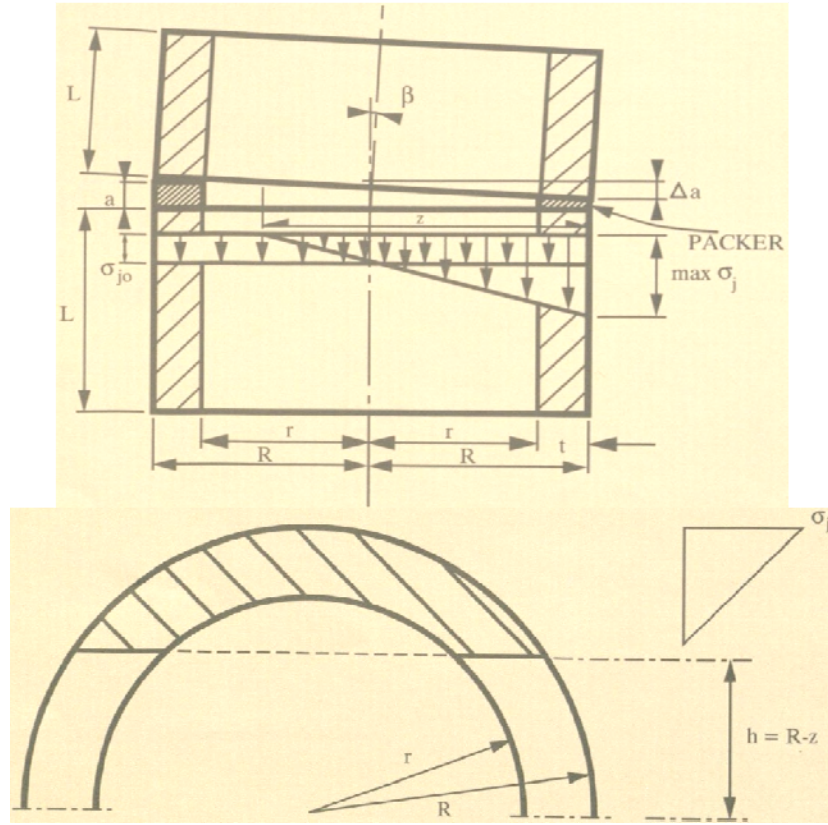


Figure 3-34 Permissible Jacking Load (PJA, 1995)

$$Z = \frac{180}{\pi} \frac{a}{E_j} \frac{\max \sigma_j}{\beta} \quad (3.49)$$

Where

$$E_j = \frac{at E_D E_P}{at E_P + t_j E_D L} \quad (3.50)$$

Permissible jacking load is:

$$= \frac{\sigma_j}{(R-h)} \left\{ 2/3 [(R^2 - h^2)^{3/2} - (r^2 - h^2)^{3/2}] - h \left[\frac{\pi}{180} R^2 \cos^{-1} \left(\frac{h}{R} \right) - \frac{\pi}{180} r^2 \cos^{-1} \left(\frac{h}{r} \right) \right] \right. \\ \left. + h^2 [(R^2 - h^2)^{1/2} - (r^2 - h^2)^{1/2}] \right\} \quad (3.51)$$

When $h > r$, permissible jacking load is:

$$= \frac{\sigma_j}{(R-h)} \left\{ 2/3 [(R^2 - h^2)^{3/2}] - h \left[\frac{\pi}{180} R^2 \cos^{-1} \left(\frac{h}{R} \right) \right] + h^2 [(R^2 - h^2)^{1/2}] \right\} \quad (3.52)$$

3.3.4 Calculation of Jacking Force by ATV DWA A-161 and ATV DWA A-125 (German Code)

The ATV DWA A-161, which was introduced in 1990, standardizes the structural analysis of pipe jacking. The standard design loads, which occur by using technologies of

steerable and non-steerable pipe jacking which are given in ATV DWA A-125 (2008), are described in the following sections.

In this standard, the theory of silo (diminishment of vertical earth pressure because of arching effects) is used, and soil density and consistency are taken into account by the parameters of soil mechanics K (earth pressure coefficient), d (wall friction angle), and c (soil cohesion). Based on ATV DWA A-161 E, the frictional resistance is influenced by:

External diameter of the jacking pipes [m]:	d_a
Jacking length [m]:	L_j
Medium depth of cover over the pipe crown [m]:	h_e
Coefficient of friction between pipe and subsoil [-]:	$\tan \delta$
Weight per unit volume of the soil [kN/m ³]:	γ_s
Internal friction angle of the soil [°]:	Φ
Earth pressure condition over the pipe crown according to ATV DWA A-161 E [-]:	k_1
Earth pressure condition below the pipe crown according to ATV DWA A-161 E [-]:	k_2

Silo effect according to Terzaghi [-]:

$$K = \frac{1 - e^{\left(-2 * k_1 * \tan\left(\frac{\Phi}{2}\right) * \frac{h_e}{d_a * \sqrt{3}}\right)}}{2 * k_1 * \tan\left(\frac{\Phi}{2}\right) * \frac{h_e}{d_a * \sqrt{3}}} \quad (3.53)$$

Factor for the consideration of curvature:

$$f_k = 1 + f_{k1} * \left(\frac{L_j}{100m}\right)^2 \quad (3.54)$$

(e.g., in steering movements, curved jacking), with:

$$\begin{aligned} f_{k1} &= 0.01 \text{ (good steering)} \\ f_{k1} &= 0.02 \text{ (normal steering)} \\ f_{k1} &= 0.03 \text{ (bad steering)} \end{aligned}$$

With Equation 3.55, the maximum required jacking force V can be estimated

(parameters were determined previously):

$$V = (k * \gamma_s * h_e) * \left[\frac{1}{2} * (1 + k_2) * \left[\left(\frac{d_a^2 * \pi}{4} \right) + (L_j * d_a * \pi * \tan \delta) \right] \right] * f_k \quad (3.55)$$

In the cohesive *soil*, a substantial portion of the resistance is ground adhesion, which can be reduced by lubrication. The most commonly used lubricant is bentonite, which is injected

through ports in the jacking head and along the pipe wall. Repeated lubrication may reduce the required jacking force by more than 50%, but more commonly, the average reduction may be about 30%.

3.3.5 Concrete Pipe Association of Australia (CPAA, 2008)⁵

In these guidelines, the same concept and values in ASCE 27 are used to calculate jacking force. To consider joint over-stressing caused by misalignment (angular deflection), and to avoid damage due to stress concentration by the jacking force, a safety factor of 3 is recommended. For detail analysis, the method introduced by Lenz and Moller (1970) is recommended. It is assumed in these guidelines that the pipes are separated by elastic packers of wood or hardboard.

3.3.5.1 Jacking Forces from Linear Joint Stress Model

The thickness of these packers before permanent deformation is a' . Packer thickness after permanent deformation, $a = 0.6 a'$.

Pipe length segment: L ,

Total packer and pipe deformation can now be written:

$$\Sigma \Delta a = \Delta a + \Delta L,$$

where Δ represents the dimensional change.

The deformations can be related to the stresses:

$$\sigma_j \frac{a}{E_j} = \sigma_j \frac{a}{E_p} + \sigma \frac{L}{E_c} \quad (3.56)$$

where σ_j is the stress in the joint and σ in the pipe wall.

E_p and E_c are the corresponding elasticity coefficients, and E_j an equivalent joint elasticity coefficient taking into consideration pipe wall elasticity.

$$\sigma = \sigma_j \frac{t_j}{t} \quad (3.57)$$

hence:

$$\sigma_j \frac{a}{E_j} = \sigma_j \frac{a}{E_p} + \sigma_j \frac{t_j}{t} \frac{L}{E_c} \quad (3.58)$$

⁵ Section 3.3.5 is partially excerpted from CPAA, 2008.

and:

$$E_j = \frac{at E_D E_P}{at E_P + t_j E_D L} \quad (3.59)$$

The problem is now reduced to that of the stress distribution in an annular cross-section where the tensile stresses are disregarded.

The ratios of inner to outer radii of the joint $r_i / r = 0.8, 0.9$ and 1.0 curves linking $\max \sigma_j / \sigma_{jo}$ and $\frac{\max \sigma_j / \sigma_{jo}}{z/r}$ are shown in Figure 3-35.

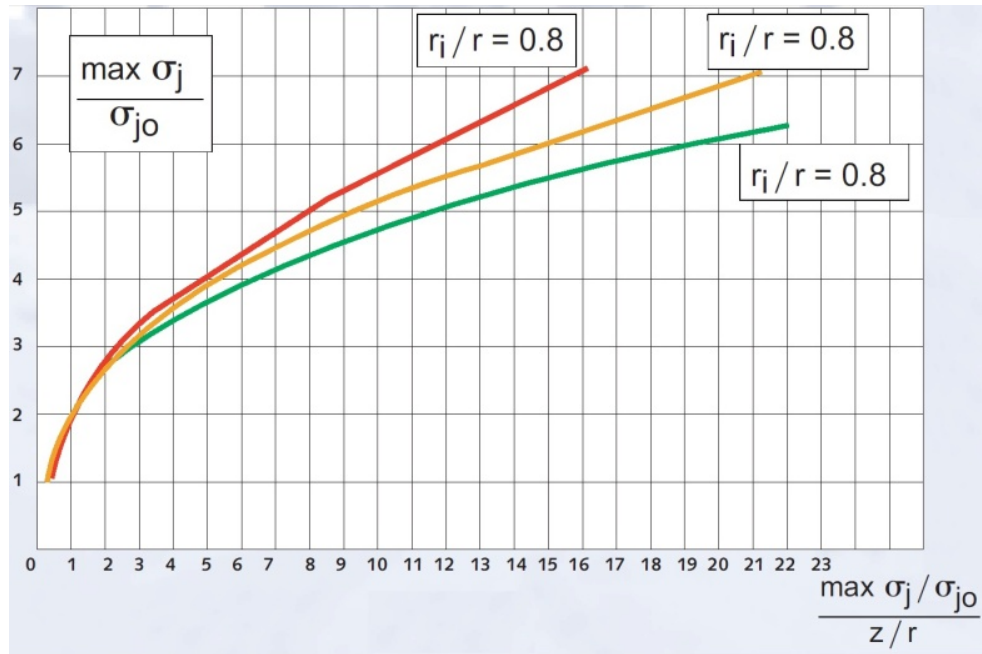


Figure 3-35 Linking Curve Uniform Stress to Maximum Stress
(CPAA, 2008)

In these equations, σ_{jo} is the joint stress for uniform load. From Figure 3-36:

$$\varphi \cong \tan \varphi = \frac{\Delta a}{z} \quad (3.60)$$

where:

$$\Delta a = \frac{a \max \sigma_j}{E_j} \quad (3.61)$$

Hence:

$$\varphi = \frac{a \max \sigma_j}{E_j z} \quad (3.62)$$

or in radians:

$$\varphi = \frac{a \sigma_{jo} \max \sigma_j / \sigma_{jo}}{E_j r \frac{z}{r}} \quad (3.63)$$

and degrees:

$$\varphi = \frac{180}{\pi} \frac{a}{E_j} \frac{\sigma_{jo}}{r} \frac{\max \sigma_j / \sigma_{jo}}{z/r} \quad (3.64)$$

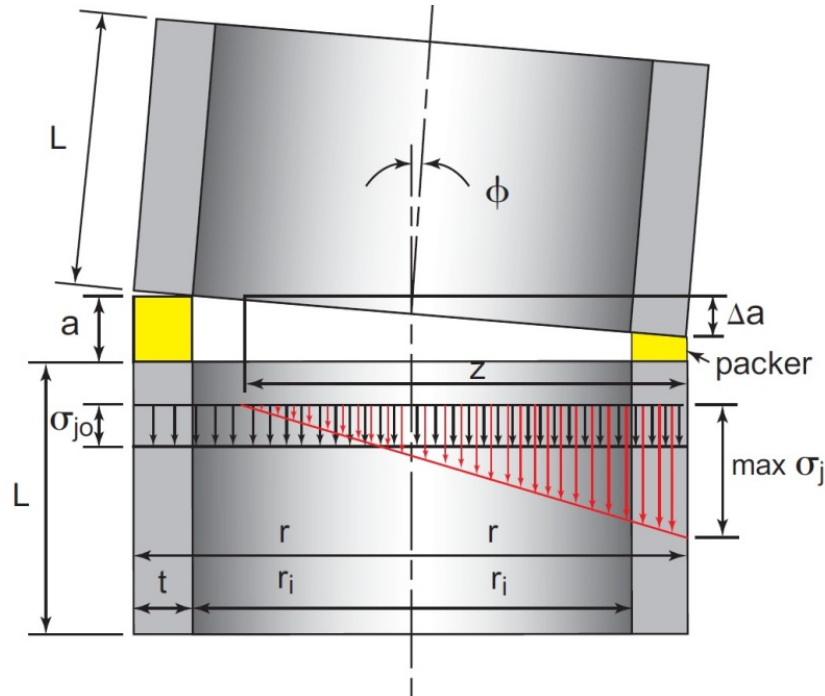


Figure 3-36 Schematic Stress Distribution in Pipe Joint Cased by Misalignment (CPAA, 2008)

This equation allows us to estimate the safe deflection for any pipe-joint configuration. It must be noted that this deflection is the combined pipe-packer deflection and is larger than what would be measured at the joint. If the deflection concentrated at the joint only is required the value of E_p should be substituted for E_j in the equation for φ .

The above considerations are based on the simple elasticity theory assuming that E is constant and independent of the stress. This assumption is not valid for concrete, but it is on the safe side. This explains why actual lines have been deflected in excess of the safe angles predicted by above considerations without causing any damage to the joints (CPAA, 2008).

3.3.6 Najafi et al. (2005)

Najafi's (2005) is one of the best resources for practical purposes in trenchless technology. Thus, a summary of this book as it relates to jacking load is presented here. A rule of thumb is introduced to calculate required jacking force according to Equation 3.65 and Figure 3-37.

$$JF = FP + \sum FR \quad (3.65)$$

where

JF= total jacking (thrusting) force, [kN]

FP= resistance of the tunnel boring machines (TBM) (penetration resistance), [kN]

FR= frictional resistance, [kN]

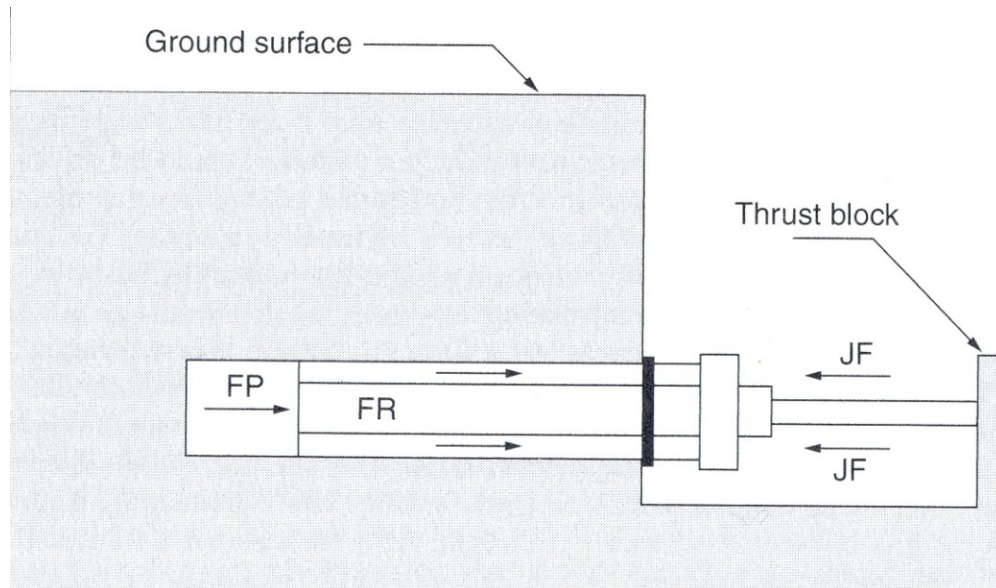


Figure 3-37 Frictional and Face (Penetration) Resistance During Pipe Jacking (Najafi, 2005)

FR, the frictional resistance can be expressed as:

$$FR = R \times S \times L \quad (3.66)$$

where

R= circumferential frictional resistance (skin friction), [kN/m²]

S= perimeter of pipe cross section = (outside diameter of pipe) x π , [m]

L= jacking (thrusting) distance, [m]

Typical numerical values for circumferential resistance (R) are shown in Table 3-6. The penetration resistance varies depending on the soil type and shape and steering action of the

boring head. For slurry shield microtunneling equipment, the value of the resistance of the leading pipe (FP) is usually calculated through the following equation:

$$FP = (P_e + P_w) \times \left(\frac{B_c}{2}\right)^2 \times \pi \quad (3.67)$$

where

P_e = contact (point) pressure of the cutting head, [kN/m²]

P_w = slurry pressure, [kN/m²]

B_c = outside diameter of the shield (boring) machine, [m]

Table 3-6 Typical Values of Circumferential Frictional Resistance for Different Types of Soil (Najafi, 2005)

Soil	Clay	Silt	Sand	Clayey gravel	Swelling clay	Sandy gravel	Loamy sand
R (kN/m ² -m)	3.9	3.9	4.8	4.8	19.3	7.6	9.0

The value of contact pressure of the cutting head (P_e) is generally assumed to be 20 psi (138 kN/m²). Najafi found that the amount of required jacking force is governed by the soil type and its characteristics such as soil density and water content (watertable). Other factors affecting jacking force include project characteristics such as height of cover, size of overcut, lubrication, overburden loads, time and jacking distance, as well as pipe characteristics such as pipe size, dimensional consistency, weight, resiliency, absorbency, and smoothness of its outer surface. In fact, the amount of jacking force that can be applied in any project is limited to a great extent by the strength of the pipe material, the area of the jacking pipe at the smallest cross section, extent of eccentricity of the resultant jacking force, capacity of jacking equipment, and load-bearing capacity of the thrust block.

Design parameters such as the jacking rate and the interaction of the soil or water pressure or both at the face are also important in determining the design jacking force. Oversize cut or use of lubricants may decrease the jacking force up to 30 percent or more (50% maximum) for clayey soils and about 20 percent for sandy soil. On the other hand, occurrence of any unexpected obstructions, such as existence of boulders or restraint of pipes cause of steering errors, can bring about sudden increase in the jacking force.

3.4 Chapter Summary

In this chapter, detailed information on different jacking load predictive models was provided. Factors in reducing jacking loads, as well as pipe and thrust block considerations were presented. The standard guidelines by ASCE 27 were compared with similar guidelines from Germany, U.K., and Australia.

CHAPTER 4

ANALYSIS

4.1 Introduction

In chapter three, pipe jacking guidelines from the U.S. and several other countries were presented. In this chapter, several hypothetical cases and actual project data are provided to clarify the concepts. The conclusion and comparison are illustrated in graphs and tables. In conclusions, design recommendations are provided.

4.2 Discussion

In the previous chapter, it was explained that the main cause of jacking force discrepancy is related to how mobilized friction around the pipe is determined. This is according to normal effective theory in Terzaghi's Trap Door Model (Equation 3.3). Staheli (2006) and Bennett (1998) have considered the simplified equation (Equation 3.4) to calculate normal force. Equations 3.3 and 3.4 were provided in Chapter 3, and are repeated here for reader's convenience.

$$P_{Ev} = \frac{B \left(\gamma - \frac{c}{B} \right)}{K \times \tan \delta} \left(1 - e^{\frac{-K \times \tan(\delta) \times h}{B}} \right) \quad (3.3)$$

$$P_{Ev} = \frac{B \gamma}{K \times \tan \delta} \quad (3.4)$$

Equation (3.4) is used for average of both horizontal and vertical pressure.

Considering the type of soil (cohesionless soil, $c = 0$) and depth of pipe (5 to 7 meters or 15 to 20-ft), which is recommended to be approximately three times the diameter of pipe (Najafi, 2010), the ratio of $\frac{B}{H}$ and the ratio of $\frac{c}{B}$ and exponential part $e^{\frac{-K \times \tan(\delta) \times h}{B}}$ are negligible (equal to zero), thus Staheli and Bennett's assumptions are correct.

Based on his experimental tests, Bennett also introduced some coefficients for calculating jacking force in cohesive soil (Table 3-3). The Bennett and AVT DWA A-125E guidelines present the only models introducing a coefficient to consider misalignment. The coefficients to calculate the frictional jacking force in ASCE 27, CPAA guidelines and Najafi model are constant for different types of soils and are based on experimental data. While in AVT DWA A-125E (German code), the average of vertical and horizontal related forces is dependent on depth.

In the Pipe Jacking Association Guidelines (British code), for cohesionless soil, the concept of calculating normal forces is the same as ATV DWA A-125 E, but a separate equation is also introduced for the cohesive soil (equation 3.39). Among the above guidelines, which employ Terzaghi Trap Door Model, the width of unstable soil is still a controversial issue.

It can be concluded that when the trapezoidal region above the boring is totally unstable (movement happened), the Terzaghi model is valid. In such a case, no cohesion force exists, [$c = 0$]), but for the other conditions, where the boring is stable, using Terzaghi model will result in higher jacking load than required.

The British guidelines introduced a criterion for the stability control of the tunnel bore (section 3.3.3.2). Also, it introduces the model for required jacking load in cohesive soil with a stable bore, which is presented in section 3.3.3.3.1 of this research.

This chapter presents two hypothetical 100-m (300-ft) long projects with different depths to compare the calculated jacking force from each model. For a better comparison and correct assessment of jacking force based on different models; only frictional part of above models is considered.

4.2.1 Hypothetical Projects to Investigate the Effects of Depth and Tunnel Boreing Stability

Two hypothetical projects with the same conditions other than the pipe depth are considered to compare the resulting jacking force predicted by each model. Ranges of friction angle, soil unit weight, Poisson ratio, and modulus of elasticity of different soils can be selected

from Tables 4-3, 4-4, 4-5 and 4-6 respectively. Medium stiff clay is selected for the subsoil and an unstable boring condition is considered to make comparison.

Assumptions for *Shallow Installation* (project (I)) are as follow. Please note that weight of the pipe must be considered in jacking load calculations.

Depth(m)	Pipe diameter(m)	Friction angle (°)	Su(c) (kN/m ²)	Unit Weight (kN/m ³)	Poisson's Ratio	Modulus of elasticity (MPa)
1.0	0.9	15	30	20	0.3	20

Assumptions for *Medium Deep Installation* (project (II)) are as follow:

Depth(m)	Pipe diameter(m)	Friction angle (°)	Su(c) (kN/m ²)	Unit Weight (kN/m ³)	Poisson's Ratio	Modulus of elasticity (MPa)
10.0	0.9	15	30	20	0.3	20

Required jacking loads (tons) based on different predicting models for project (I) (Table 4-1) are illustrated in Figure 4-1, and for project (II) (Table 4-2) are presented in Figure 4-2.

Table 4-1 Predicting Mobilized Friction Jacking Loads (tons) in a Hypotetical Project (I)

Length	ASCE 27& CPAA		British	Bennett			Najafi	Germany
	Firm Clay (5-20)kPa/m			Stiff to Hard Clay			Clay & Silt	
	Lower Bound	Upper Bound		Lower Bound	Best Fit	Upper Bound		
0	0	0	0	0	0	0	0	0
10	14	57	11	7	9	14	13	12
20	28	113	22	14	18	27	25	25
30	42	170	33	21	27	41	38	37
40	56	226	44	27	36	55	51	49
50	70	283	56	34	45	68	64	62
60	85	339	67	41	54	82	76	74
70	99	396	78	48	63	96	89	87
80	113	452	89	55	72	109	102	99
90	127	509	100	62	81	123	115	111
100	141	566	111	68	90	136	127	124

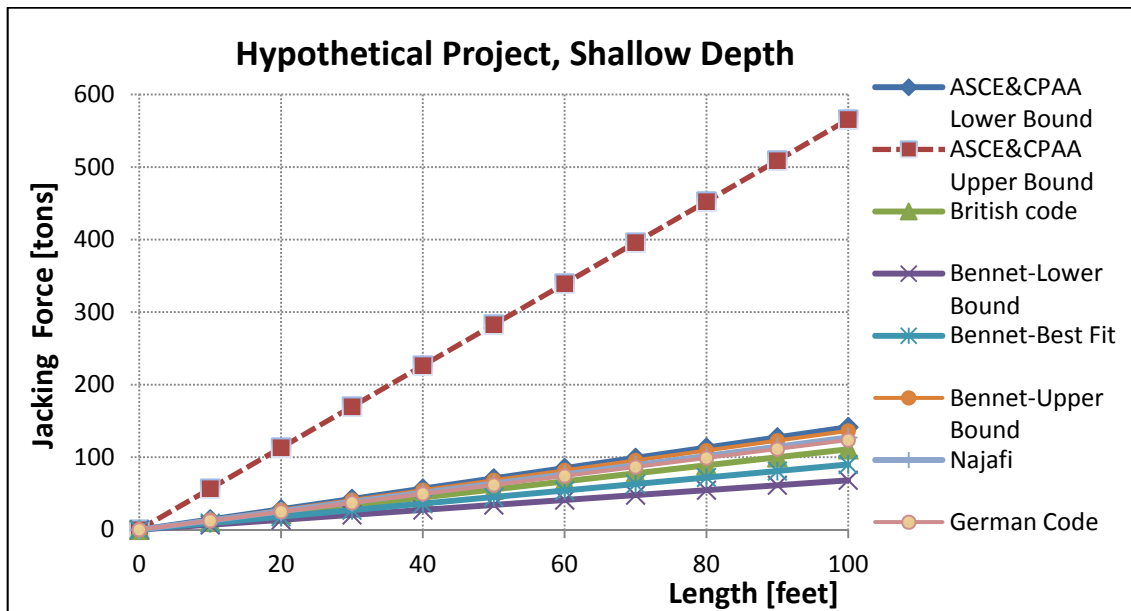


Figure 4-1 Graphical Presentation of Calculated Jacking Friction Forces from Different Predicting Models in a Hypothetical Project (I)

Table 4-2 Predicting Mobilized Friction Jacking Loads (tons) in a Hypothetical Project (II)

Length	ASCE 27& CPAA		British	Bennett			Najafi	Germany
	Firm Clay (5-20) kPa/m			Stiff to Hard Clay			Clay & Silt	
	Lower Bound	Upper Bound		Lower Bound	Best Fit	Upper Bound		
0	0	0	0	0	0	0	0	0
10	14	57	50	7	9	14	13	57
20	28	113	99	14	18	27	25	114
30	42	170	148	21	27	41	38	171
40	56	226	198	27	36	55	51	228
50	70	283	247	34	45	68	64	285
60	85	339	296	41	54	82	76	342
70	99	396	346	48	63	96	89	399
80	113	452	395	55	72	109	102	456
90	127	509	445	62	81	123	115	513
100	141	566	494	68	90	136	127	570

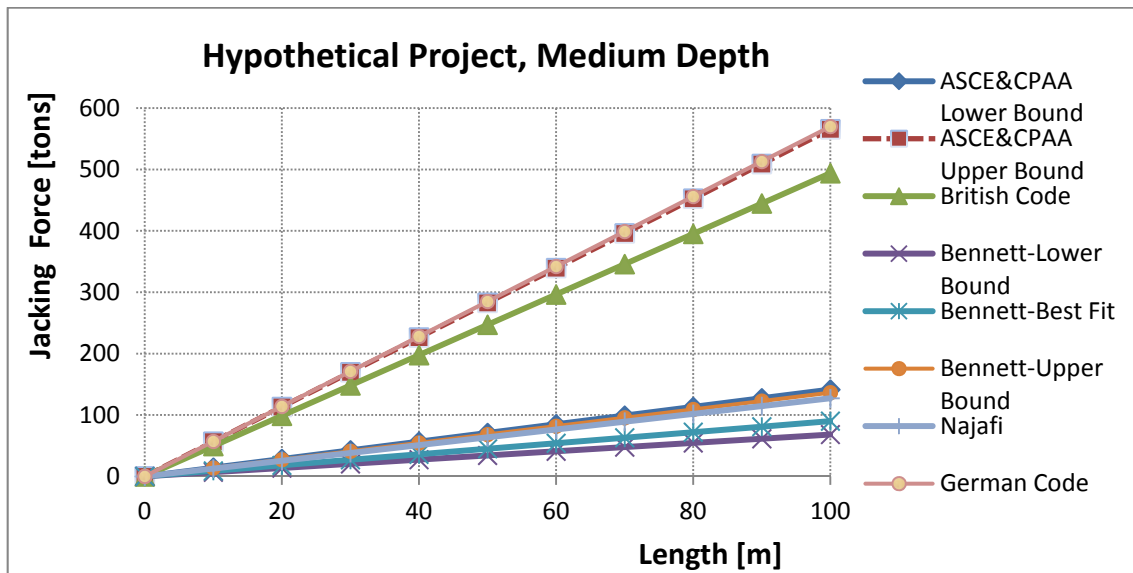


Figure 4-2 Graphical Presentation of Calculated Jacking Friction Forces from Different Predicting Models in a Hypothetical Project (II)

Table 4-3 Ranges of Friction Angles for Soils (Budhu, 2010)

Soil type	ϕ (degrees)
Gravel	30–35
Mixtures of gravel and sand with fine-grained soils	28–33
Sand	27–37
Silt or silty sand	24–32
Clays	15–30

Table 4-4 Typical Values of Unit Weight for Soils (Budhu, 2010)

Soil type	γ_{sat} (kN/m ³)	γ_d (kN/m ³)
Gravel	20–22	15–17
Sand	18–20	13–16
Silt	18–20	14–18
Clay	16–22	14–21

Table 4-5 Typical Values of Poisson's Ratio (Budhu, 2010)

Soil type	Description	ν'
Clay	Soft	0.35–0.4
	Medium	0.3–0.35
	Stiff	0.2–0.3
Sand	Loose	0.15–0.25
	Medium	0.25–0.3
	Dense	0.25–0.35

These values are effective values ν'

Table 4-6 Typical Values of E and G (Budhu, 2010)

Soil type	Description	E^* (MPa)	G^* (MPa)
Clay	Soft	1–15	0.5–
	Medium	15–30	5–15
	Stiff	30–100	15–40
Sand	Loose	10–20	5–10
	Medium	20–40	10–15
	Dense	40–80	15–35

*These are average secant elastic moduli for drained condition

4.2.1.1 Discussion of Results

In above projects, the calculated values based on Najafi, Bennett, ASCE 27, and CPAA models are constant and they are not dependent on the depth at which the pipes are laid. In addition, the ASCE 27 upper bound is approximately four times of other modeling results.

Furthermore, the British and German codes yield approximately four times of the rest of the modeling values in this medium depth project.

The values calculated by experimental models are based on several tests and projects, and are, therefore, valid results, but they should be used in the same site conditions (depth, water content, pipe diameter, and soil type). It should be noted that most of the experimental predicting models were conducted at medium depth (2 to 3 times D). Therefore, the values predicted by ASCE 27 for the upper bound projects (I) and (II) and the experimental models results for project (I) are unrealistic (in this project, weight of concrete pipe causes for experimental and theoretical results to be exceptionally similar). It seems selecting the Terzaghi's model, to calculate normal pressure is not correct, because the instability assumption over the tunnel bore may not happen.

The British code (section 3.3.3.2), is the only one that has a criterion regarding tunnel stability and the cohesive soil is considered the only soil condition for a tunnel to be stable without any support pressure. By checking the requirement of stability (Figure 3-29), it can be concluded that the tunnels are stable in both projects. Using equation 3.39, the required force to overcome friction is estimated to be 70 tons for both projects, which is near to experimental model prediction values.

The stability of the tunnel has a great effect on the required jacking force. Now that, the importance of the stability of boring tunnel was cleared, it is the time to investigate the soil stability boring criteria.

4.2.2 Calculating Unstable Region over the Pipe in Pipe Jacking Method

By calculating the unstable region above tunnel boring, normal pressure can be calculated, which can suggest an estimation of jacking forces. To calculate the unstable region, the stress distribution in the soil must be considered first and then, by selecting suitable failure criteria, the unstable region can be calculated. Later, the weight of unstable region would be considered for estimating normal pressure to calculate required jacking force. In the following

paragraphs, calculation of stress distribution over the tunnel, selected Mohr-coulomb failure criterion and the involved parameters in calculating the unstable region above the tunnel are explained in more details.

4.2.2.1 Stress Distribution in Soil Around a Tunnel Bore

To check the stability of soil above the pipe crown, it is required to investigate the stress distribution on the soil. An example of the stress distribution in a circular hole is shown in Figure 4-3. To simplify the condition, the following assumption is made: The vertical soil depth over the pipe which causes the vertical stress is constant, $\sigma_{zz} = \gamma \cdot h_p = q = \text{const.}$, and for the horizontal stress: $\sigma_{xx} = K_o \cdot \sigma_{zz} = K_o \cdot q = \text{const.}$ h_p is the depth of the pipe below the ground surface, soil density is γ , and the lateral pressure coefficient is K_o .

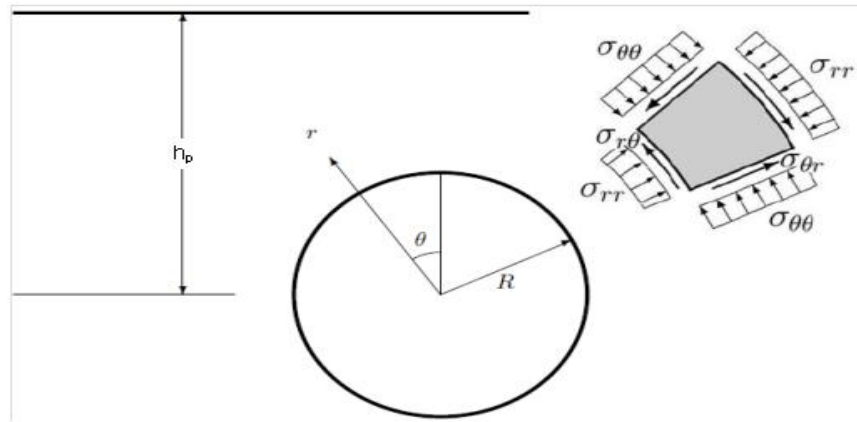


Figure 4-3 Circular Opening in the Half Space
(Schad et.al, 2008)

For normally consolidated soil, the relation for K_o is (Das, 2007):

$$K_o \approx 1 - \sin \varphi' \quad (4.1)$$

For over-consolidated soil, the at-rest earth pressure coefficient may be expressed as (Das, 2007):

$$K_o = (1 - \sin \varphi') \text{OCR}^{\sin \varphi'} \quad (4.2)$$

Under the assumption that Hooke's Law applies between lateral pressure coefficient and Poisson's ratio ν , the relation is:

$$K_0 = \frac{\nu}{1-\nu} \quad \text{or} \quad \nu = \frac{K_0}{1+K_0} \quad (4.3)$$

On the basis of equilibrium conditions in linear elastic material behavior and applying Mindlin (see Schad et.al, 2008) relation for the homogeneous and isotropic half-space, the following equations can be extracted for stress calculation around tunnel boring (in polar coordinates of Figure 4-4):

$$\sigma_{rr} = q \cdot \left\{ \frac{1+K_0}{2} \left[1 - \left(\frac{R}{r} \right)^2 \right] + \frac{1-K_0}{2} \left[1 + 3 \left(\frac{R}{r} \right)^4 - 4 \left(\frac{R}{r} \right)^2 \right] \cos 2\theta \right\} \quad (4.4)$$

$$\sigma_{\theta\theta} = q \cdot \left\{ \frac{1+K_0}{2} \left[1 + \left(\frac{R}{r} \right)^2 \right] - \frac{1-K_0}{2} \left[1 + 3 \left(\frac{R}{r} \right)^4 \right] \cos 2\theta \right\} \quad (4.5)$$

$$\sigma_{r\theta} = q \cdot \frac{1-K_0}{2} \left[1 - 3 \left(\frac{R}{r} \right)^4 + 2 \left(\frac{R}{r} \right)^2 \right] \sin 2\theta \quad (4.6)$$

Note: Mindlin procedure (see Schad et.al, 2008) is based on Hooke's law, $\nu \leq 0.5$, thus extracted equation is valid for $K_0 \leq 1$. But in practice, mainly in rock or heavily over-consolidated soil, lateral pressure coefficients are more than one. Also, compressive stresses are positive (+).

Thus, the above equations for the edge of the opening ($r = R$) are:

$$\begin{aligned} \sigma_{rr} &= 0 \\ \sigma_{\theta\theta} &= q \cdot [1 + K_0 - 2(1 - K_0) \cos 2\theta] \\ \sigma_{r\theta} &= 0 \end{aligned}$$

The extreme values of $\sigma_{\theta\theta}$ would be in crown and invert: $\theta = 0^\circ$, $\theta = 180^\circ$ minimum pressure, maximum tension can be calculated by $\sigma_{\theta\theta} = q(3K_0 - 1)$, and in the spring position, $\theta = 90^\circ$ largest compressive stress can be calculated by: $\sigma_{\theta\theta} = q(3 - K_0)$

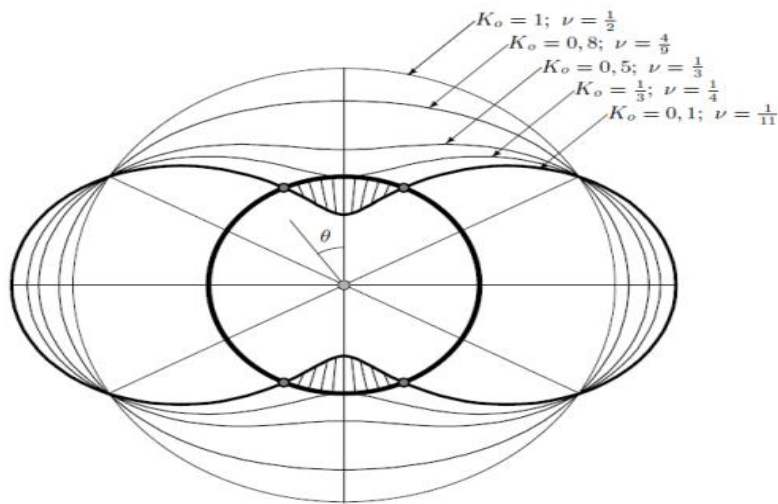


Figure 4-4 Tangential Stresses at the Edge of a Circular Opening (Schad et.al, 2008)

4.2.2.2 Failure Criteria

Usually three regions of soil states are defined, as illustrated in Figure 4-5, and considered for practical implications (Budhu, 2010), as follow:

- **Region I.** Impossible soil states. A soil cannot have soil states above the boundary AEFB.
- **Region II.** Impending instability. Soil states within the region AEFA (Figure 4-5.a) or 1-2-3 (Figure 4-5.b) are characteristic of dilating soils that show peak shear strength.
- **Region III.** Stable soil states (safe design).

Soil states that are below the failure line or failure envelope AB (Figure 4-5.a) or 0-1-3 (Figure 4-5.b) would lead to safe design. Soil states on AB are failure (critical) states.

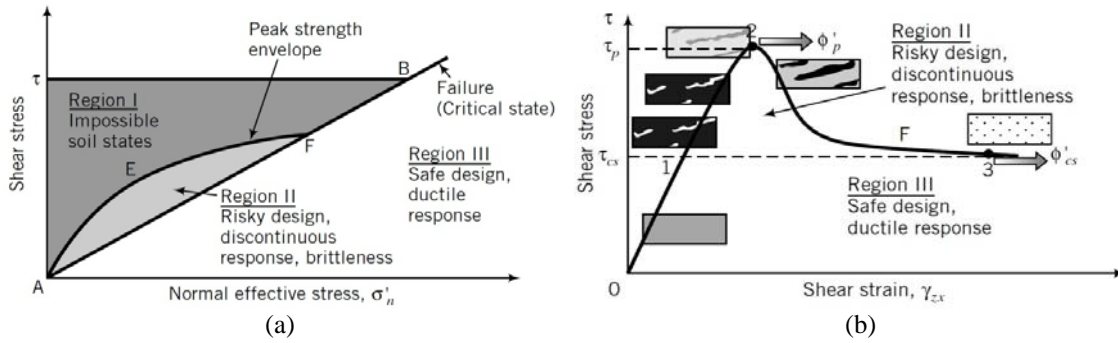


Figure 4-5 Interpretation of Soil States (a) Schematic, (b) Real Condition (Budhu, 2010)

There are several failure criteria such as Coulomb, Mohr- coulomb, Tresca, and Taylor, but in this research Mohr-coulomb (granular type) and Tresca failure criteria (clayey type) have been considered. Mohr's circle can be used to determine the stress state within a soil mass. By combining Mohr's circle for finding stress states with Coulomb's frictional law, generalized failure criterion was developed (Figure 4-6). In Mohr-coulomb, we have following relations:

$$\sin \varphi' = \frac{(\sigma'_1)_f - (\sigma'_3)_f}{(\sigma'_1)_f + (\sigma'_3)_f} \quad (4.7)$$

$$\theta = \frac{\pi}{4} + \frac{\varphi'}{2} \quad (4.8)$$

$$(\sigma'_n) = \frac{\sigma'_1 + \sigma'_3}{2} - \frac{\sigma'_1 - \sigma'_3}{2} \sin \varphi' \quad (4.9)$$

$$\tau_f = \frac{\sigma'_1 - \sigma'_3}{2} \cos \varphi' \quad (4.10)$$

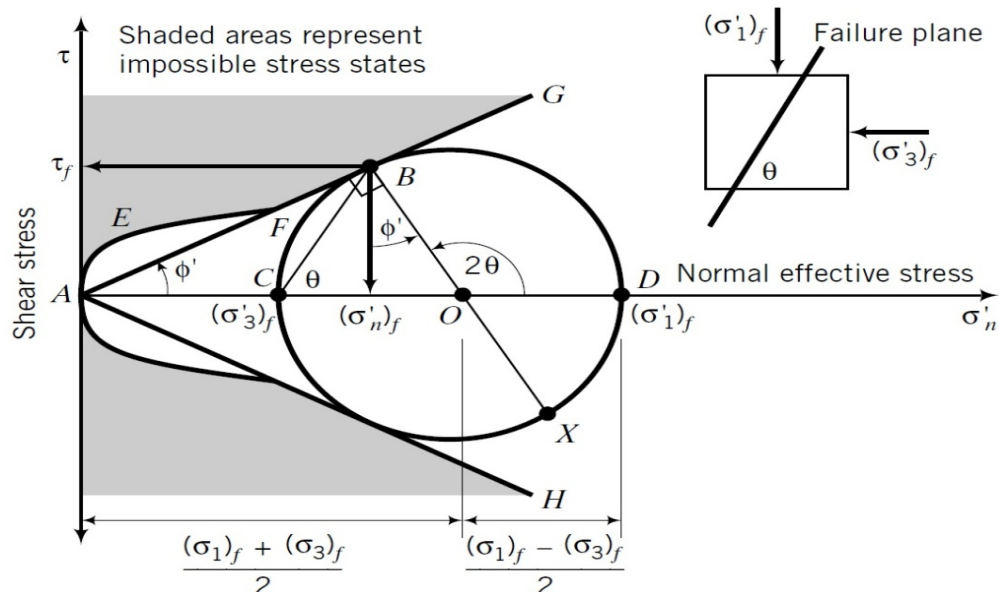


Figure 4-6 The Mohr-Coulomb Failure Envelope
(Budhu, 2010)

The failure criterion in a fine-grained soil under undrained conditions is evaluated by Tresca Criterion. Shear stresses at failure are one-half the principal stresses (to interpret the undrained shear strength). The undrained shear strength, S_u , is the radius of the Mohr total stress circle (Figure 4-7); that is,

$$S_u = \frac{(\sigma_1)_f - (\sigma_3)_f}{2} = \frac{(\sigma'_1)_f - (\sigma'_3)_f}{2} \quad (4.11)$$

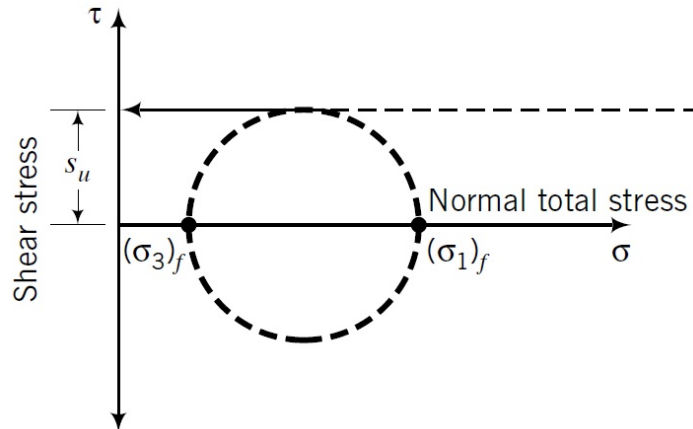


Figure 4-7 Mohre Circle for Undrained Condition
(Budhu, 2010)

In the above equations, parameters are:

σ'_1, σ'_2 = Principal effective stress

σ'_n = Effective normal stress

φ = Shear strength parameter of soil (where φ in the above equation is drained strengths ($\varphi = \varphi'$) for long-term analysis and undrained ($\varphi = 0$, $c = S_u$) for short-term analysis of cohesive materials)

θ = The failure plane from the plane of the major principal stress

4.2.2.3 Failure Senario

Although Mindlin (Schad et.al, 2008) relation is valid under very limited conditions (linear elastic, Isotropic, homogenous, and $K < 1$), it is possible to get some interpretations about the behavior of bored tunnel state in different soil types, such as the following:

- 1- For any type of soil with $K \leq 1/3$, the tunnel is not stable.
- 2- For fine drained grain soil, and for all angles of friction, the boring is not stable.
- 3- For the undrained situation for the condition of $S_u \geq \frac{\sigma_{\theta\theta}}{2}$ the boring is stable.
- 4- For the normally or lightly consolidated drained soil, as the cohesion and angle of friction are not as high as the loading caused by the soil's weight, the Mohr circle will pass the Coulomb criterion and the tunnel will not be stable.
- 5- For the heavy consolidated soil as the shear capacity of soil increases, it is possible to introduce the unstable region over the boring. The philosophy of calculating unstable region is the same as shear crack propagation model in concrete beam shear failure.

So far, some clarifications have been carried out about the stability of tunnel boring and its effect on estimating the required jacking forces. However, the following questions have not been answered yet:

- Why is there a difference between the experimental models values (ASCE 27, CPAA, Najafi, and Bennett) and the model which used Terzaghi's Trap Door (British and German Code)?, and

- Why are the experimental values less than calculated ones in medium depth installation?

By introducing smaller width for unstable region (considering better failure criteria) in predicting the jacking load, Staheli (2006) answered these questions for sandy soil type. The same concept can be used for clayey type soil. Also, some other soil capacity may exist that were not considered. The following sections provide more information.

4.2.2.4 Effects of Cohesion

A close examination of the Terzaghi Trap Door Model (equation 3.3) reveals that another way to decrease the normal pressure over the pipe is considering soil cohesion. The following paragraphs will provide some more information about this phenomenon.

$$P_{Ev} = \frac{B \left(\gamma - \frac{c}{B} \right)}{K \times \tan \delta} \left(1 - e^{\frac{-K \times \tan(\delta) \times h}{B}} \right) \quad (3.3)$$

The term cohesion, C , as used conventionally in geotechnical engineering, is an apparent shear strength that captures the effects of intermolecular forces (c_o), soil tension (c_t), and cementation (c_{cm}) on the shear strength of soils (Figure 4-8).

4.2.2.4.1 Effects of Soil Tension⁶

Soil tension is the result of surface tension of water on soil particles in unsaturated soils. A suction pressure (negative pore water pressure from capillary stresses) can be created that pulls the soil particles together. Since the effective stress is equal to total stress minus pore water pressure, thus, if the pore water pressure is negative, the normal effective stress increases. Since soil is a frictional material, this normal effective stress increase leads to a gain in shearing resistance.

In such case, the intergranular friction angle or critical state friction angle does not change (Figure 4-9). If the soil becomes saturated, the soil tension reduces to zero. Thus, any gain in shear strength from soil tension is only temporary. It can be described as an apparent shear strength c_t . In practice, it will not rely on this gain in shear strength, especially for long-

⁶ Sections 4.2.2.4.1 and 4.2.2.4.2 are excerpted from Budhu, 2010.

term loading. There are some situations, such as shallow excavations in fine-grained soils that will be opened for a very short time, in which you can use the additional shear strength (apparent shear strength) to your advantage.

4.2.2.4.2 Effects of Cementation

Nearly all natural soils have some degree of cementation, wherein the soil particles are chemically bonded. Salts such as calcium carbonate (CaCO_3) are the main natural compounds for cementing soil particles. The degree of cementation can vary widely, from very weak bond strength (soil crumbles under finger pressure) to the bond strength of weak rocks (Budhu, 2010).

The shear strength from cementation is mobilized at small shear strain levels (0.001%). In most geotechnical structures, the soil mass is subjected to much larger shear strains. At large shear strains, any shear strength due to cementation in the soil will be destroyed. Also, the cementation of natural soils is generally non-uniform.

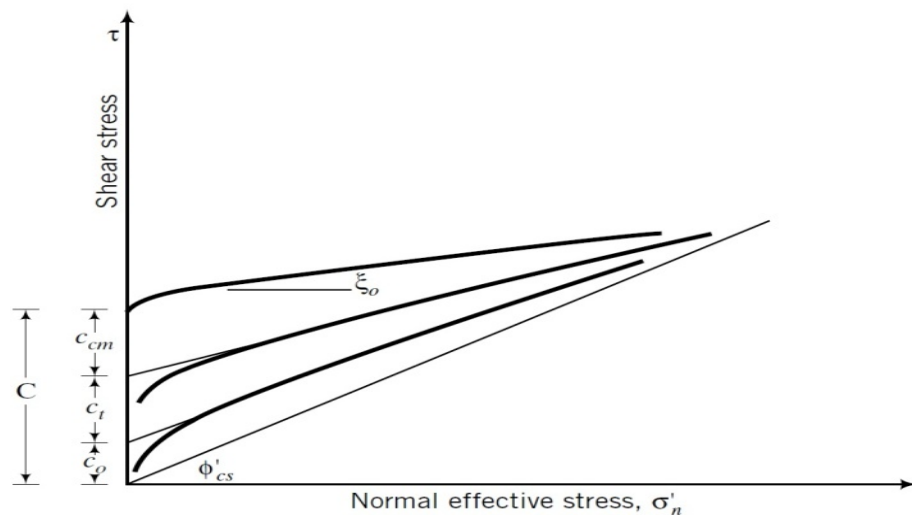


Figure 4-8 Peak Shear Stress Envelope for Soils Resulting from Cohesion, Soil Tension and Cementation (Budhu, 2010)

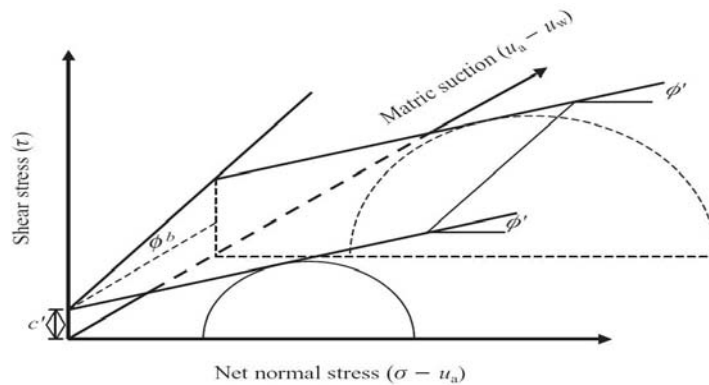


Figure 4-9 Failure Envelope for Unsaturated Soils
(Budhu, 2010)

Considering the information provided above, it can be concluded that some types of soil have some hidden shear capacity, which are not relied on in engineering calculations. This extra capacity may cause a self-stabilizing tunnel when it is not classically expected to happen. This may cause the difference between true engineering models and experimental results for calculating required jacking force. This hidden capacity may change drastically by changing the moisture of the soil. Thus, due to change in moisture conditions, different jacking loads for the same soil type can be expected.

4.3 Pipe Jacking Survey

As explained previously, the best way to predict the required jacking force is using the recorded values from the previous project with the same specifics. Consequently, a series of survey investigation were carried out to gather the recorded information from previous projects (Survey forms are provided in appendix A). Around 50 professionals were contacted, but most of the respondents did not provide all the requested information. Therefore, the survey results were not used in this thesis.

Due to lack of complete survey data, and for comparison purposes, data from two previous projects (one project from Staheli (2006) and another project data collected by Dr. Jason Lueke of Arizona State University), and data from a box culvert project in Dallas, Texas, are used and presented in the following sections.

4.3.1 Sandy Soil Project, East Side Interceptor, Morris Avenue Drive (Staheli, 2006)

For sandy soil type, one of Dr. Staheli's recorded data (2006) is used (Figure 4-10). To continue a comparison of her model with other predicting models and data from the Eastside Interceptor – Morris Avenue Drive is illustrated (Figure 4-11). The first 180-ft (54-m) of this project was not lubricated and the parameters required for jacking pipe prediction models are presented in Table 4-7.

Table 4-7 Parameters Used in Prediction Models Eastside Interceptor, Morris Avenue Drive (Staheli, 2006)

Pipe Material	Packer head Concrete	Soil Type	Loose Sands	Depth to Crown	17-ft (5.1-m)
Pipe Diameter	87.5-in. (2,222-mm)	Residual Friction Angle	32 degrees	Depth to Water	6-ft (1.8-m)
Pipe Weight	2,138 lbs/ft (31.2 kN/m)	Soil Unit Weight	110 pcf (18 kN/m ³)	Interface Friction Coefficient (Scherle)	0.25
Interface Friction Coefficient (Staheli)	0.58	Bennett Arching Factor, C_a (unlubricated)	0.75 (Lower) 1.0 (Best) 1.5 (Upper)	Bennett Friction Factor, C_f (unlubricated)	1.0

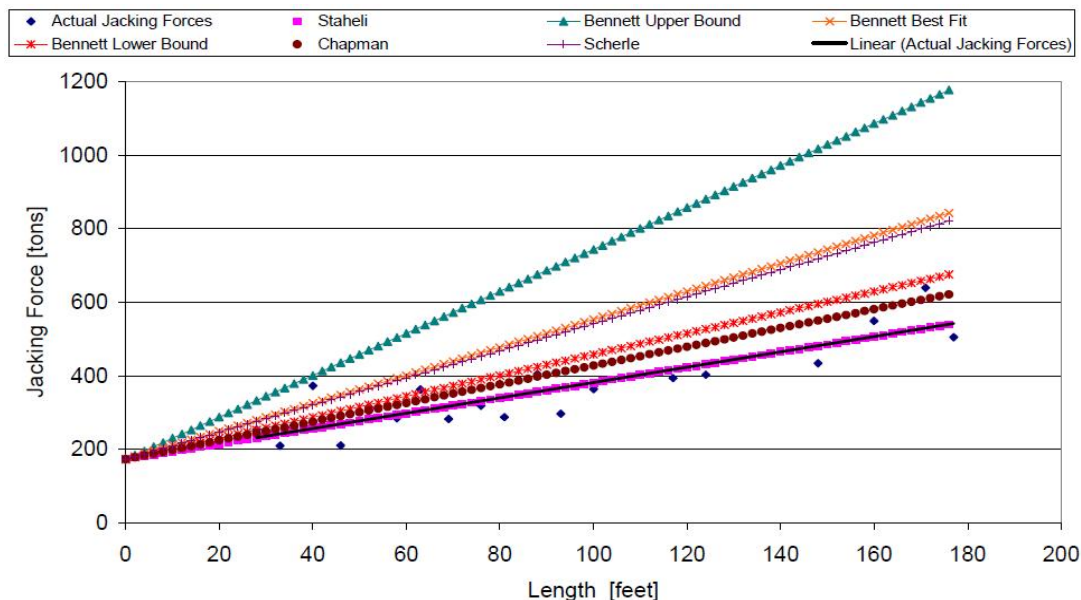


Figure 4-10 Length versus Actual and Predicted Jacking Forces with a Variety of Predictive Models for the Eastside Interceptor – Morris Avenue Drive (Staheli, 2006)

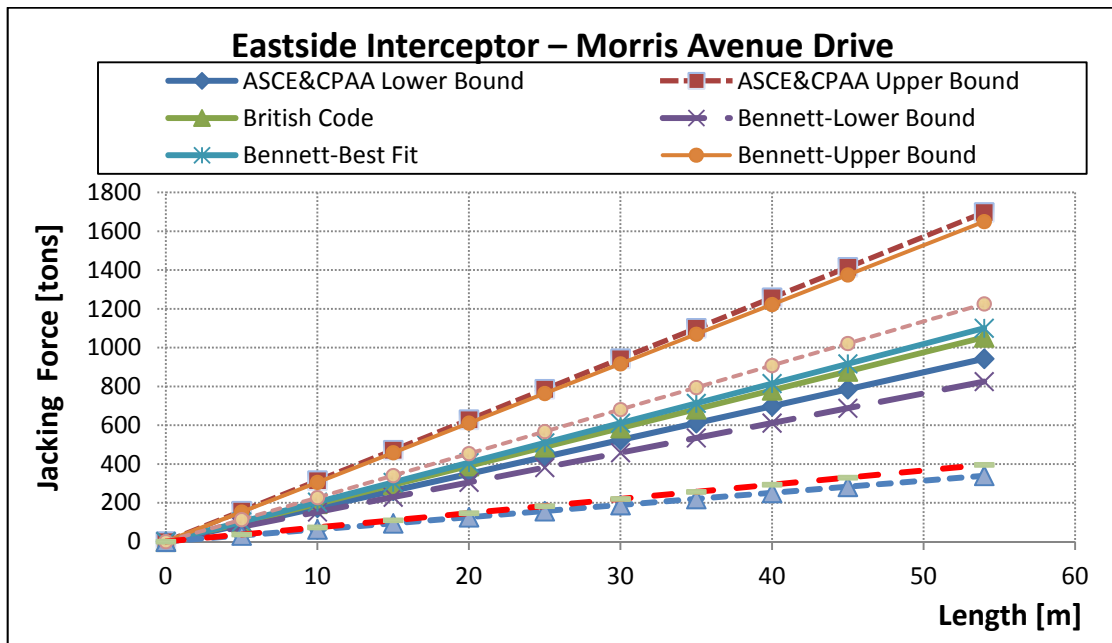


Figure 4-11 Comparison Among the Selected Jacking Force Models in Sandy Soil Type

4.3.2 Clayey Soil Project, Reid Drive, Appleton, Wisconsin (Lueke, 2012)

The information for this project is generously provided by Dr. Lueke from Arizona State University. This project is located at Reid Drive, Appleton, Wisconsin (Figure 4-12) and included the three steps of pilot tube microtunneling (Figure 4-13) which was explained in section 2.2.2.

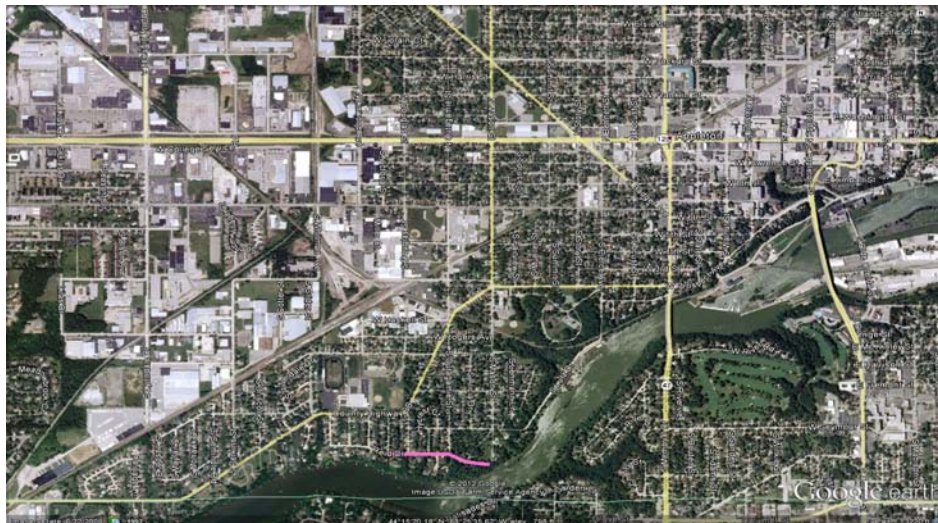


Figure 4-12 Reid Drive, Appleton, Wisconsin, Project Location
(Source: Dr. Jason Lueke, Arizona State University, 2012)

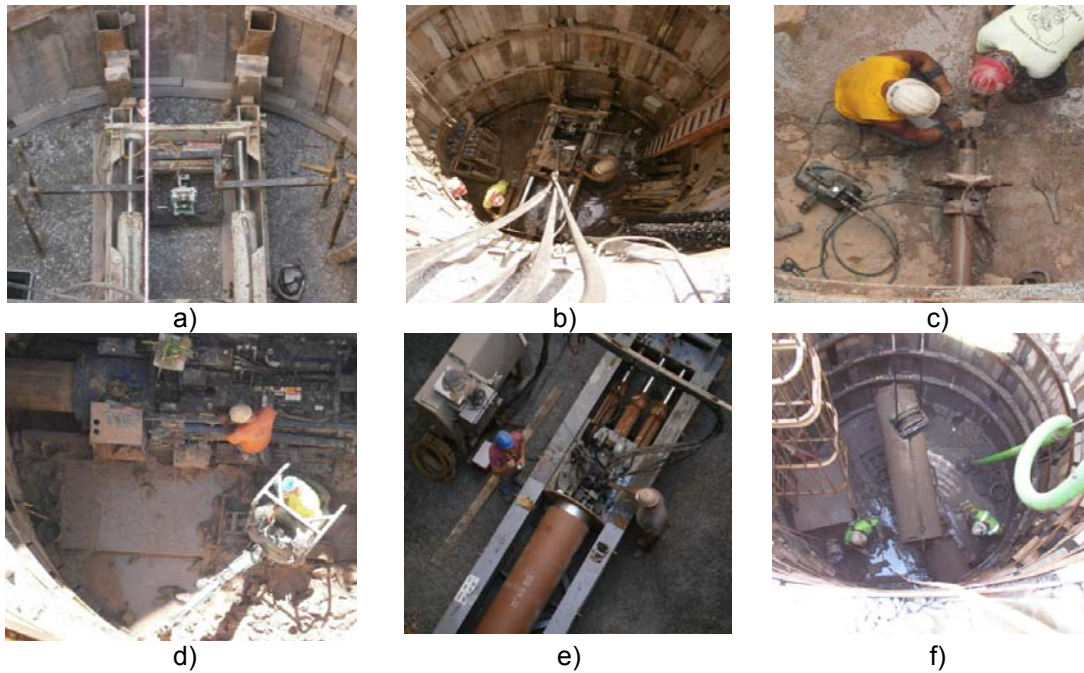


Figure 4-13 Pilot Tub Microtunneling Reid Drive, Appleton, Wisconsin, Construction Procedure:
a) Jacking system installation, b) Pilot tube installation, c) Pilot tube, receiving shaft,
d) Casing installation, e) VCP installation, f) Casing removal
(Source: Dr. Jason Lueke, Arizona State University, 2012)

The specifications of this project are presented in Table 4-8

Table 4-8 Parameters Used in Prediction Models Reid Drive, Appleton, Wisconsin
(Source: Dr. Jason Lueke, Arizona State University, 2012)

Total project length	385-ft (115-m)	Pipe Material	Clay pipe	Soil Unit Weight	107 pcf (16.8 kN/m ³)
Project grade	0.2%	Pipe Compressive Strength	7000 psi (48,263 kN/m ²)	Soil Moisture	23%
Depth to Crown	40-ft (12-m)	Carrier Pipe Diameter	24-in. (610-mm) OD 20.4-in. (517-mm) ID	Soil Type	ML. Red-brown-gray silty clay, trace fine sand-moist
Depth to Water	20-ft (6-m)	Casing Pipe Diameter	25.5-in. (648-mm) OD 23.5-in. (597-mm) ID	CU test UU test	C= 3 psi, ϕ = 31° Su= 0.3-0.5 tsf
Lubrication	Yes	Type of Lubrication	13 lb bentonite with 100 lb water Just during casing installation	Size of Overcut	1.5-in. (38.1-mm)

Figure 4-14 presents the data recording procedure employed to record the jacking load forces data. In this method, two pressure transducers connected to a data logger were attached to the Akkerman hydraulic unit on the hydraulic lines directly behind the gauges.

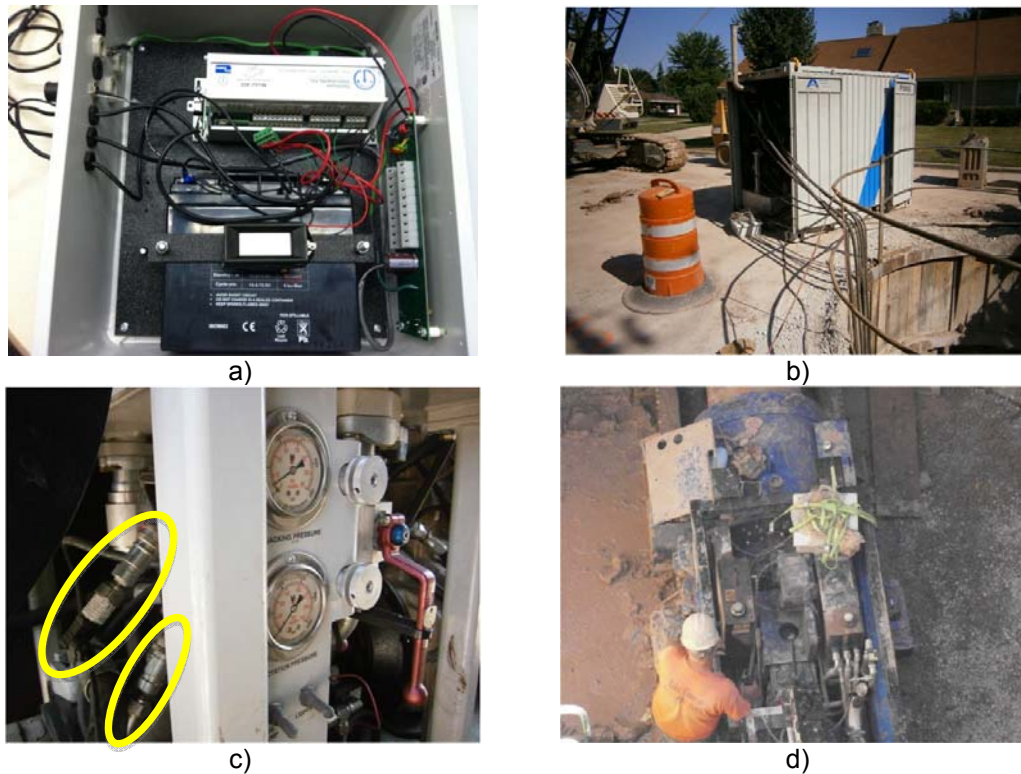


Figure 4-14 Detail of Recording Data System Procedure, a) Data logger with battery, b) Power unit on surface beside shaft, c) Pressure transducers circled with yellow, d) Recording system was attached to jacking system during operation (Source: Dr. Jason Lueke, Arizona State University, 2012)

As the boring operation was below watertable level, it was initially expected that the tunnel will not be stable (or partially stable). The following coefficients are selected for calculations (The reduction coefficients in parenthesis are multiplied to consider lubrication effect):

Interface Friction Coefficient (Staheli)	0.54 x (0.1-0.5)
Bennett Arching Factor, C_a (unlubricated)	0.5 (Lower), 0.5 (Best), 0.66 (Upper)
Bennett Friction Factor, C_f (unlubricated)	0.5 (Lower), 0.5 (Best), 0.66 (Upper)
Najafi Friction Factor	4.5 x (0.5-0.7)
ASCE 27	5-20 x (0.5-0.7)
British and German Codes	Calculated Value x (0.5-0.7)

Considering selected jacking load models and instable boring tunnel for casing pipe (Figure 4-15) and carrier pipe (Figure 4-16), it was expected that the required jacking loads will be in the range of 30-60 tons. In the worst scenario, it was expected that the required jacking

loads should not increase more than 300 tons. Thus, it was recommended to design the shaft, thrust block, and jacking system for a 300-ton capacity.

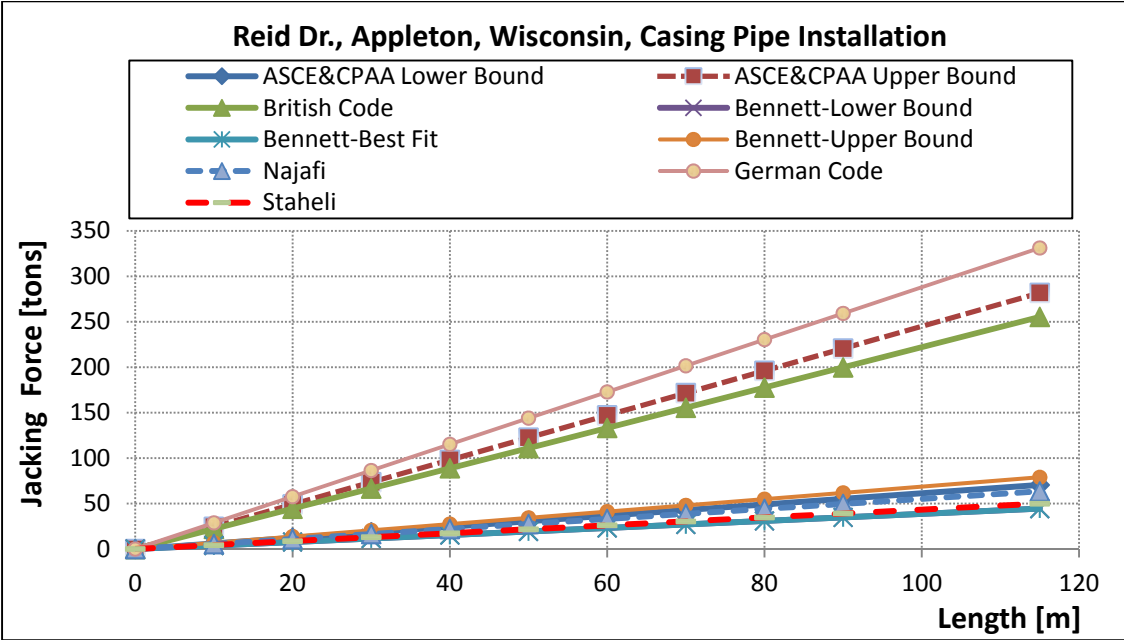


Figure 4-15 Predicting Required Jacking Loads to Install Casing in Reid Drive, Appleton Project

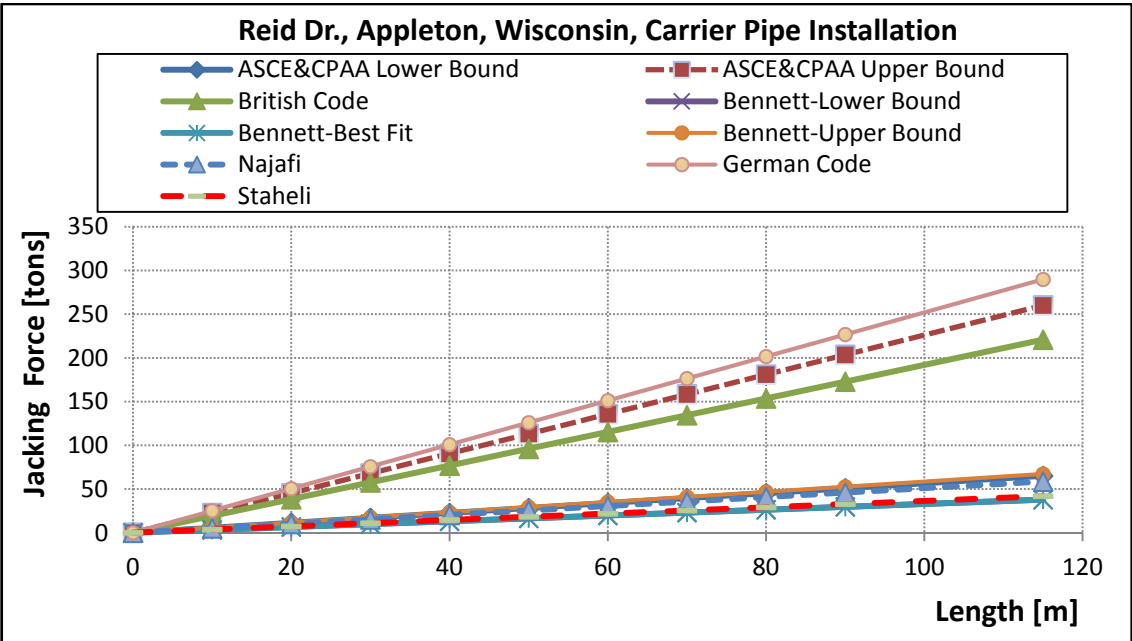


Figure 4-16 Predicting Required Jacking Loads to Install Carrier Pipe in Reid Drive, Appleton Project

The results of recorded jacking pipe in the three steps of PTMT pipe installation are presented in Figure 4-17. Some high points on the graph are observed, which can be related to differences in static and dynamic frictional coefficients and one work stoppage at 100-ft during the weekend (Figure 4-17 (b)).

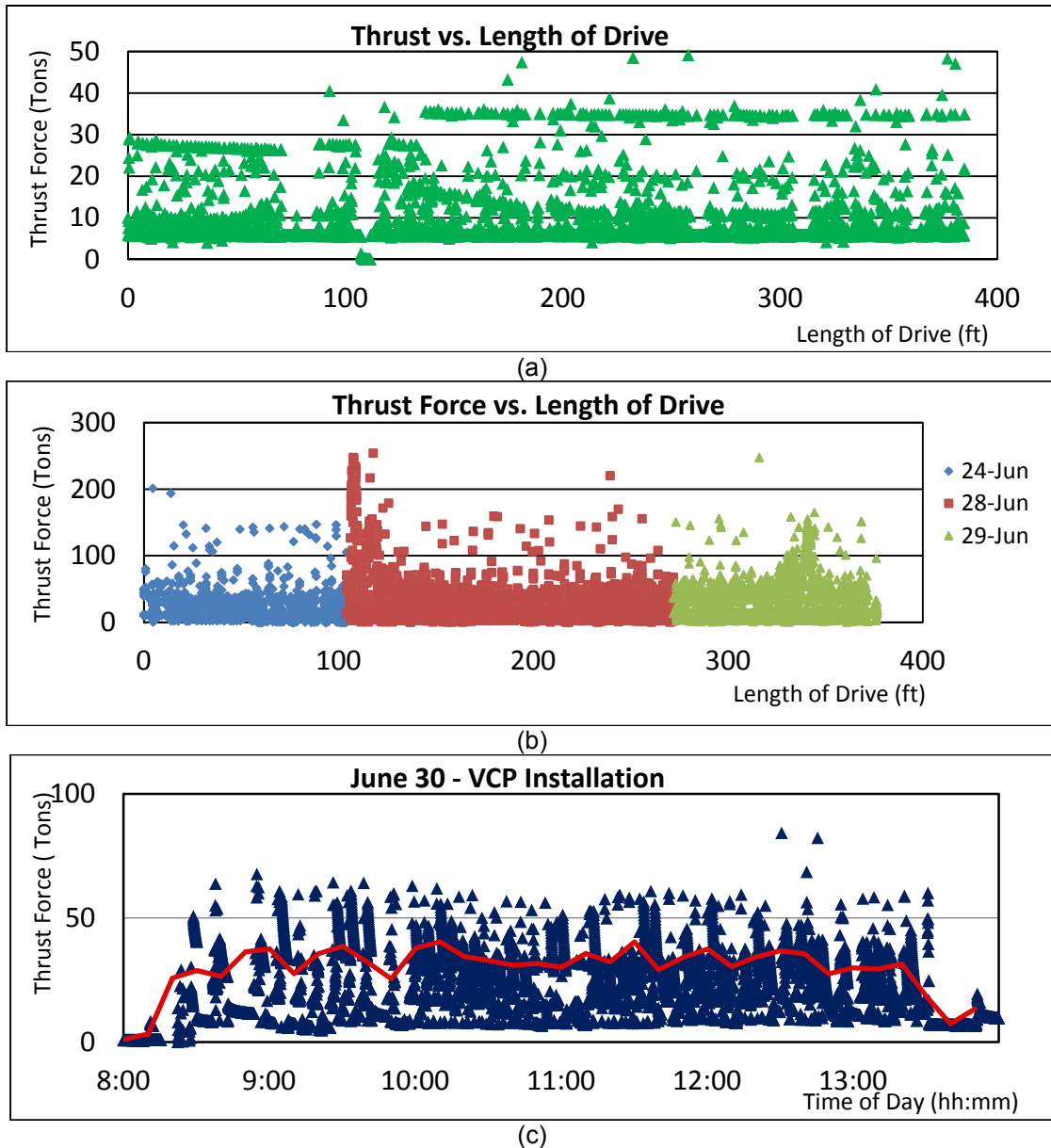


Figure 4-17 Jacking Load during PTMT Installation of Reid Drive, Appleton Project, (a) Step1- Pilot Tube Installation, (b) Step 2- Casing Installation, (c) Step 3- Clay Pipe Installation (Source: Dr. Jason Lueke, Arizona State University, 2012)

4.3.3 Two Barrel Drainage Hand Mining Culvert Installation, located at the intersection of IH 635 and Josey lane, Dallas, Texas (April 2012)

The information for this project is provided by Mr. Art Daniel from AR Daniel Construction Services, Inc (ARDCS). This project was located at the intersection of IH 635 and Josey lane, Dallas, Texas (Figure 4-18). In this project, 130-ft (39-m) of two adjacent (with 3-ft gap) concrete box culvert barrels 8 x 8 x 6-ft (2,400 x 2,400 x 1,800-mm) segments were installed at 32-ft (9.6-m) depth with hand-mining procedure. The soil at the project was mainly soft silty clay. A dewatering system around the driving shaft was used. The contractor installed a concrete railing system under the culvert to keep the grade of installation. For this purpose, two small tunnels of 3 x 3-ft (1,800 x 1,800-mm) at each barrel alignment were excavated initially and a concrete slab was placed ahead of main culvert installation (Figure 4-19). Approximately, for the first 1/3 of total length of each barrel installation, only two-bottom jacks were employed and for the rest of the length four jacks with bentonite injection were engaged. Figure 4-20 illustrates a view of driving jacking shaft and the component of jacking systems.

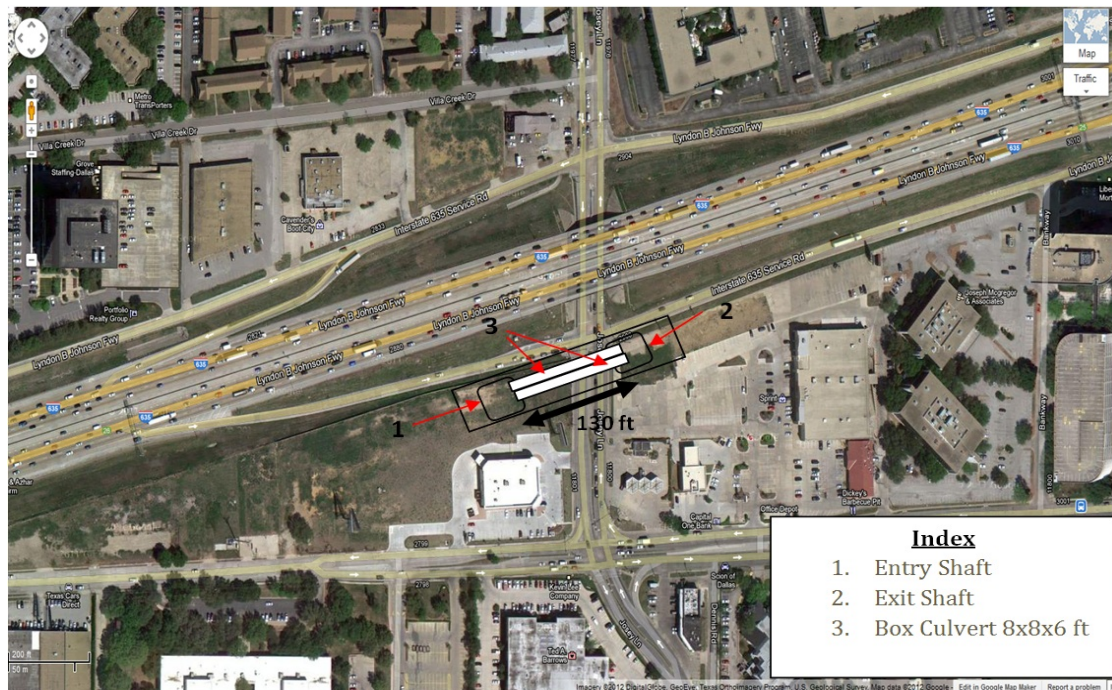


Figure 4-18 Site Layout Josey Lane project, Dallas, Texas
(Source: Google Maps)



Figure 4-19 Guide Railing System Installed at Josey Lane Project.
The Top Board Prevents Face Collapse



Figure 4-20 Driving Shaft Components, Such as Hydraulic Jacks and Ventilation Hose

Recorded jacking loads and line misalignments are presented in Figures 4-21, 4-22, 4-23, and 4-24. In the misalignment Figures, the positive values are high and right and the negative are low and left.

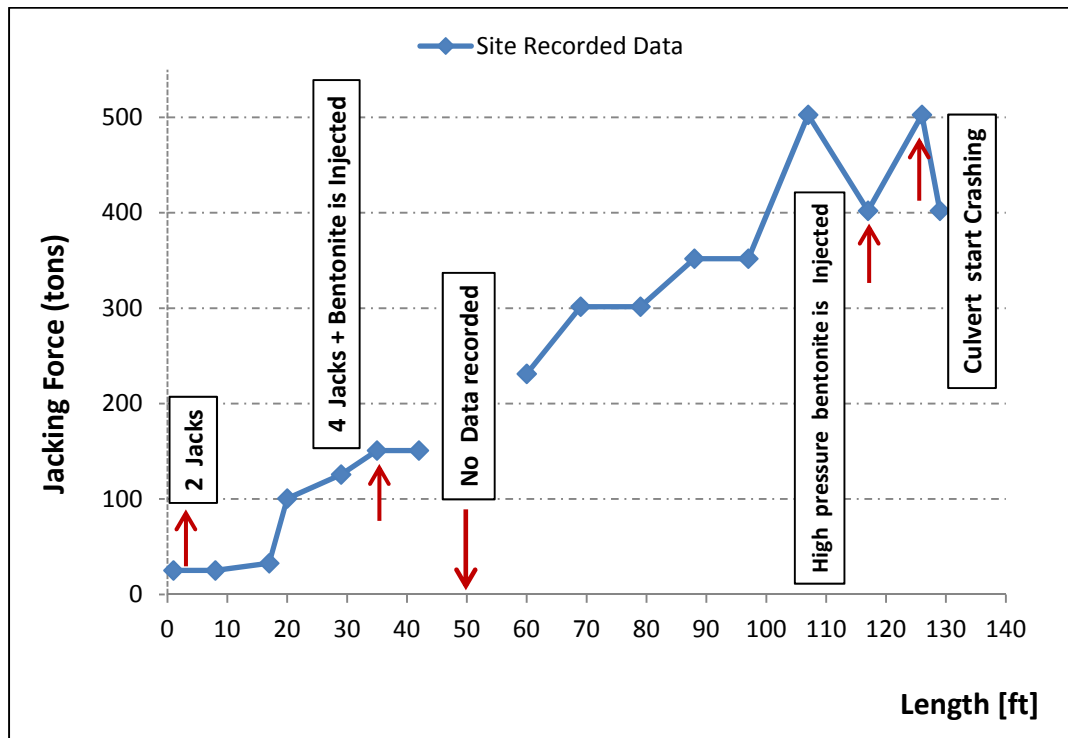


Figure 4-21 Left Barrel Installation-Josey Lane Project Trinity Infrastructure
(Source: Mr. Art Daniel, AR Daniel Construction Services, 2012)

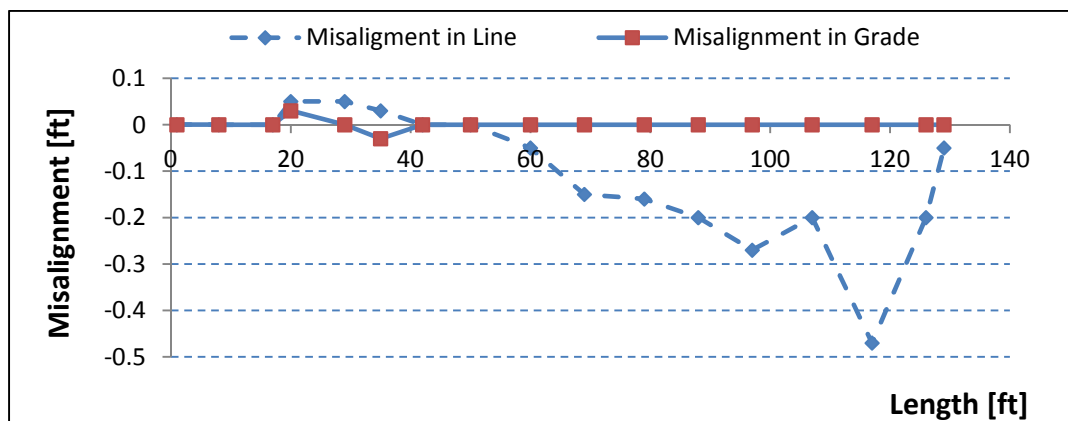


Figure 4-22 Misalignment in Left Barrel Installation-Josey Lane Project
(Source: Mr. Art Daniel, AR Daniel Construction Services, 2012)

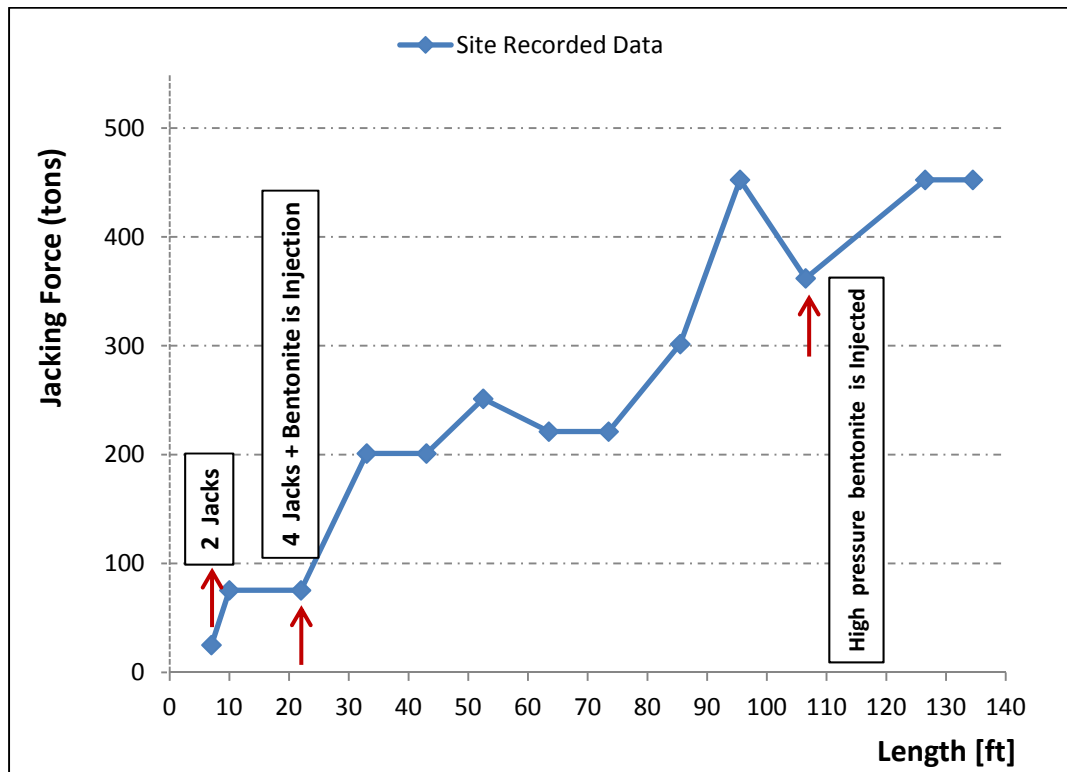


Figure 4-23 Right Barrel Installation-Josey Lane Project Trinity Infrastructure
(Source: Mr. Art Daniel, AR Daniel Construction Services, 2012)

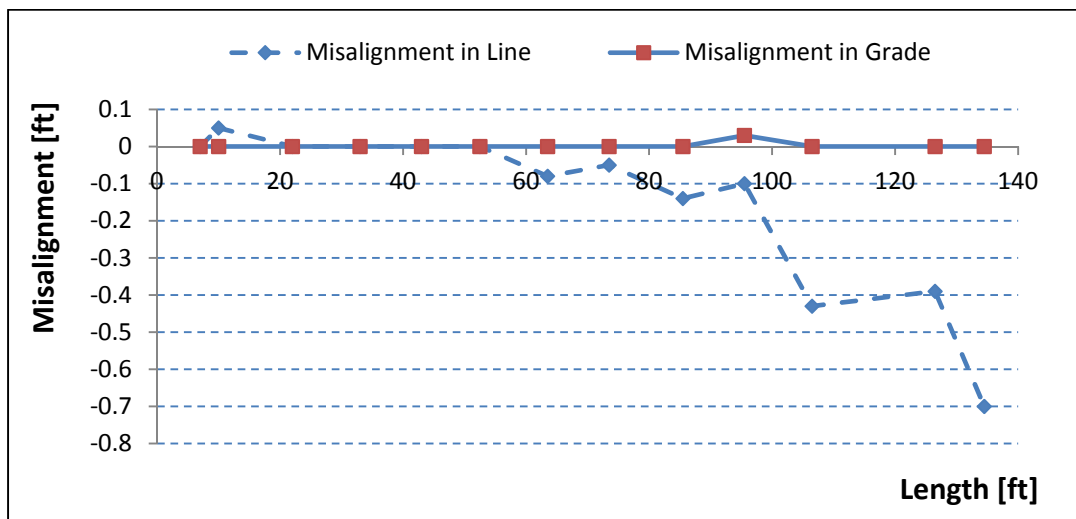


Figure 4-24 Misalignment in Right Barrel Installation-Josey Lane Project
(Source: Mr. Art Daniel, AR Daniel Construction Services, 2012)

ASCE 28 is selected to predict the required jacking loads, as it is the only known guidelines for box culvert jacking. The required jacking load coefficient is the same as circular culvert (ASCE 27, Table 3-5). Based on the soil type of Josey Lane project (silty clay), the jacking loads without lubrication is predicted at about 217-868 tons (which a large range); however, by considering lubrication effects, this load can be reduced by 40% to 130-520 tons. The contractor estimated a maximum jacking load of 122-482 tons for installation 8 x 8-ft by 6-ft long concrete box culvert for the total 130-ft drive. This jacking load was achieved using lubrication and additional measures such as spraying top of box culvert with epoxy paint.

Lessoned Learned:

1. By installing concrete railing, the grade misalignments is reduced to the minimum, but 0.7-ft line misalignment, which could not be corrected at the right barrel, still exists. To correct the line misalignment, in the hand mining culvert installation, the misaligned side can be corrected by additional excavating on the other side. In the right barrel, in Figure 4-24, it seems the soil between the two barrels was not stable and the contractor could not use the support of the soil to correct the direction.

2. Outside dimensions of the culvert segments should have required maximum tolerance to prevent misalignments (different dimensions from one segment to another may create misalignment) which may also increase jacking loads.

3. Thickness of the culvert segments should be limited to an acceptable tolerance to prevent one segment with more thickness to push the soil above it and carry for the length of the bore, resulting an increase in the jacking loads. For this purpose, the contractor used a steel plate to help ease any thickness differences.

4. Painting the surface of culverts can prevent water absorption, and allow the lubrication work effectively.

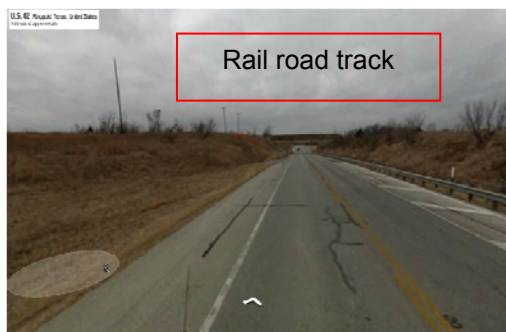
4.4 Case Study Projects

4.4.1 Ringgold Project

The purpose of this project, located near Wichita Falls, Texas, was to improve drainage system in this region. Currently, a 4 x 4-ft (1,220 x 1,220-mm) drainage box culvert exists, however, three rows of new 36-in. (900-mm) pipe culverts must be installed by pipe jacking method parallel to existing one to facilitate the flow of basin water in Area 1 (see Figure 4-25). The pipe jacking alignment is located under a two-lane highway (US 82).



Figure 4-25 Ringgold Project Location (Source: Google Map, 2011)

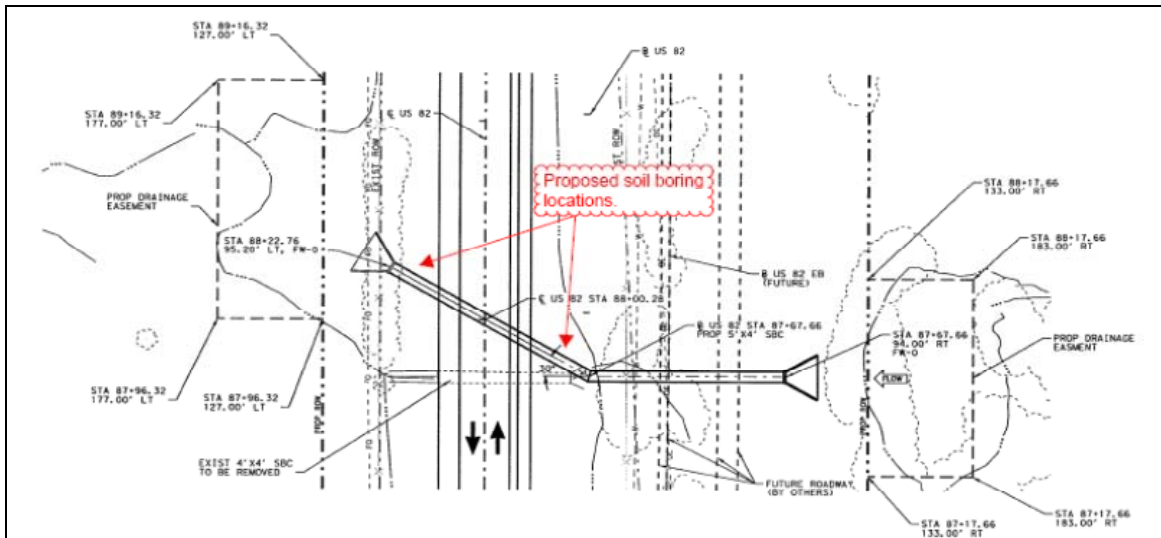


(a)

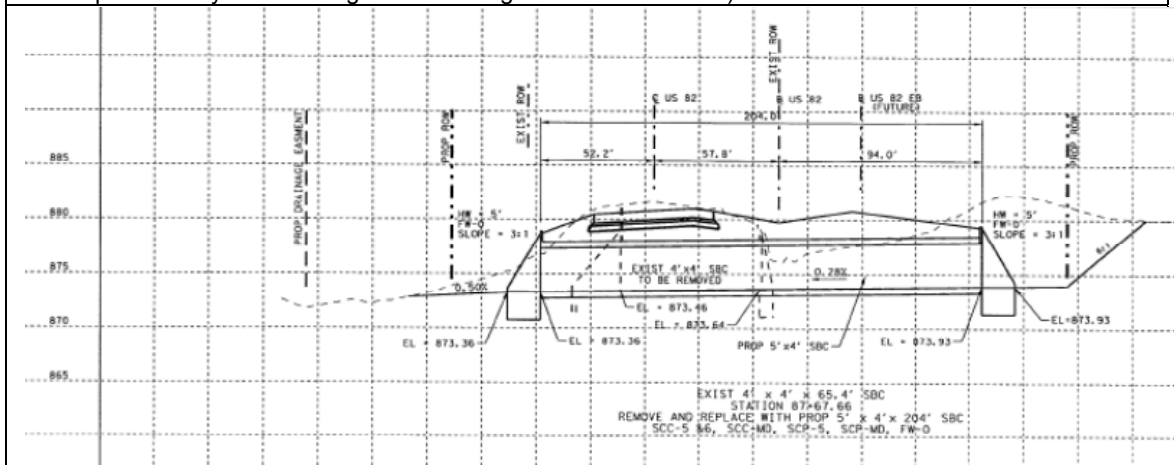


(b)

Figure 4-26 (a) West view of US 81 at Concrete Culvert position (b) East view of US81 at Culvert Position (Source: Google Map, 2011)



Remediation and Extension plan (Existing 4 x 4ft Culvert will be filed, three concrete pipelines estimated to be 36" diameters, will be constructed under pavement by pipe jacking with 30° direction with existing culvert and the extension parts will be constructed with open cut system along the existing culvert direction)



Section of new design

Figure 4-27 Design Documents for Ringgold Project
(Source: TxDOT Wichita Fall District)

Useful Extracted Information from drawings:

1. Existing Culvert information 4 x 4-ft (1,200 x1,200-mm)
2. Culvert Slope -0.28%
3. Diagonal trenchless length= 110-ft (33-m)
4. Straight open cut length=94-ft (31-m)

Depth(m)	Pipe diameter(m)	Friction angle (°)	Su(c) (kN/m ²)	Unit Weight (kN/m ³)	Poisson's Ratio	Modulus of elasticity (MPa)	Length(m)
1.0	0.9	30	30	20	0.3	20	33

By checking the tunnel stability (section 3.3.3.2), the tunnel is expected to be stable, but due to the truck loads (HS20), the tunnel will not be stable and the calculation should be based on unstable conditions. Based on calculation provided in Table 4-9, most predicting models resulted in jacking loads of 35 to 60 tons, not considering a safety factor (please note that weight of the pipe is considered in calculations).

Table 4-9 Predicting Mobilized Friction Jacking Load (Tons) in Ringgold Project

Length	ASCE 27& CPAA		British	Bennett			Najafi	Germany
	Firm Clay (5-20)kPa/m			Stiff to Hard Clay			Clay & Silt	
	Lower Bound	Upper Bound		Lower Bound	Best Fit	Upper Bound		
0	0	0	0	0	0	0	0	0
3	5	19	4	2	3	5	4	4
7	9	37	7	5	6	9	8	8
10	14	56	11	7	9	14	13	12
13	19	75	15	9	12	18	17	16
17	23	93	18	11	15	23	21	20
20	28	112	22	14	18	27	25	24
23	33	131	26	16	21	32	29	29
26	37	149	29	18	24	36	34	33
30	42	168	33	20	27	41	38	37
33	47	187	37	23	30	45	42	41

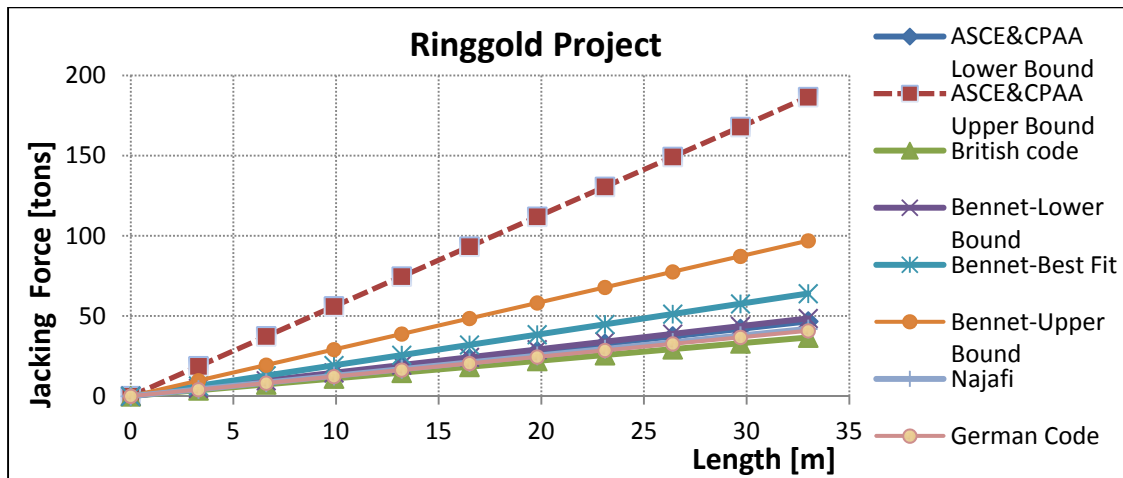


Figure 4-28 Graphical Presentation of Calculated Jacking Friction Force from Different Predicting Model in Ringgold Project

4.4.2 Vernon Project

The purpose of this project, near Wichita Falls, Texas, was to alleviate a flood problem. Three rows of 6 x 4-ft (1,800 x 1,200-mm) drainage box culverts were constructed and another 6 x 4-ft (1,800 x 1,200-mm) box culvert was proposed to be installed by box jacking method. The box jacking alignment was under a four-lane highway and two service roads (see Figure 4-29).



Figure 4-29 Alignment of Proposed Culvert
(Source: Google Map, 2011)

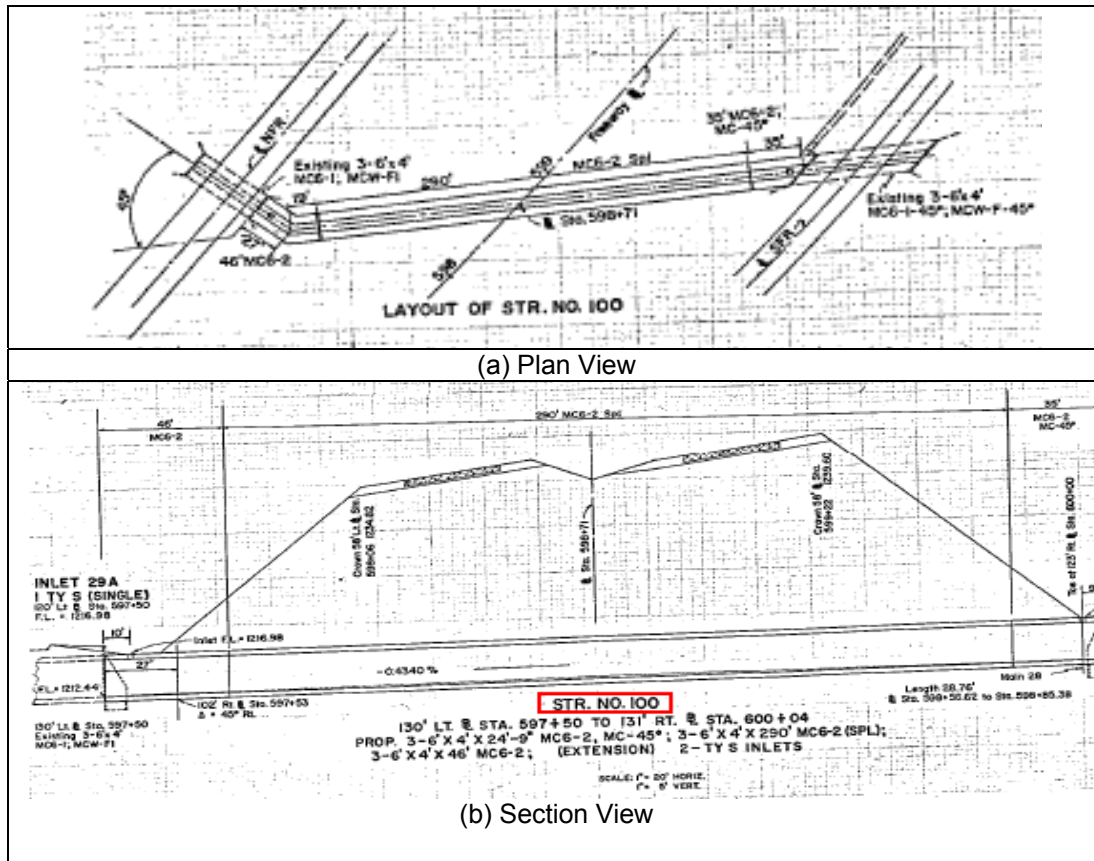


Figure 4-30 As Built Documents for Vernon Project
(Source: TxDOT Wichita Fall District)

Useful Extracted Information:

- 1- Culvert information 3- 6 x 4-ft (1,800 x1,200-mm)
- 2- Culvert Slope -0.434%
- 3- Total length= 46-ft +19-ft +290-ft +35-ft +46-ft= 436-ft (125-m)
- 4- Angle at Conjunction box = 45°

Depth(m)	Pipe Dimention(m)	Friction angle (°)	Su(c) (kN/m ²)	Unit Weight (kN/m ³)	Poisson's Ratio	Modulus of elasticity (MPa)	Length(m)
15.0	1.8 x 1.2	30	30	20	0.3	20	125

As mentioned previously, to calculate jacking loads for box culvert pipe installation, the only reference is ASCE 28. For the silty- clay soil type the frictional jacking resistance ranges from 5 to 20 KPa (Table 3-5). The maximum estimated jacking loads without lubrication is expected to 375 to 1,500 tons. By considering 40% reduction coefficient for lubricated conditions, the expected range of jacking loads is expected to be roughly 225 to 900 tons.

4.4.3 *Design Recommendation for TxDOT Projects*

Ringgold Project: As the tunnel is instable, and the pipe must be laid in a shallow depth, it seems the best estimate should be based on British and German codes (25 tons). The experimental models are not reasonable to be used because most of them are related to deep to medium depth projects. By considering a safety factor of two for operational stoppage, 50-tons capacity for the pipe, abutment, jacking frame, and jacking cylinder and 75 tons for designing pipe joints (Safety factor of 3 to consider misalignment based on CPAA standard) would be a rough estimation. If bentonite lubrication is used a 30%-50% decrease on (based on ASCE, CPAA, German standard, and Najafi, Bennett models) above-mentioned values should be considered.

Vernon Project: The jacking loads are expected to be in the range of 375 to 1,500 tons at un-lubricated and 225 to 900 tons at lubricated conditions. By considering a safety factor of two for operation stoppages, etc., a 1,800-ton capacity is predicted for the pipe, abutment, jacking frame, and jacking cylinder.

4.5 Chapter Summary

In this chapter, several examples and case studies to calculate frictional force in pipe and box jacking method were presented. For comparison purposes, a rough estimate of jacking loads for two hypothetical projects, three past projects and two future TxDOT projects was provided.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION FOR FUTURE RESEARCH

5.1 Conclusions

This study presented an analysis and synthesis of pipe jacking literature, and provided a comparison of available standard guidelines for pipe jacking operations. Several hypothetical and case study projects were analyzed and discussed to clarify the concepts. The following is a summary of this research:

- In ASCE 27 and CPAA standards, the higher and lower limits have more than 400% difference in some types of soil (firm clay). The same is observed in Bennett's model; however, in this model, a best-fit value is introduced. Bennett's best fit and Najafi's coefficients provide more realistic values.
- Staheli's model is for granular soil type and it seems a suitable failure mode is selected to model the normal pressure over the pipe. Staheli's model was compatible with data obtained from case study projects, considering a safety factor for design purposes.
- Only German code considers misalignment in calculating jacking forces. The effects of misalignment require more research.
- Only British guidelines consider the stability of tunnel. Stability of bored tunnel has a significant effect on the jacking loads. It is recommended to consider the experimental models as the upper bond, since the lubrication or watertable may cause some instability and increase the required loads.
- ASCE 28 is the only known guideline for calculating box culvert jacking loads.
- As part of this research study, concentration stress on pipe due to misalignment are calculated and added to ASCE 27 Section (Section 3.3.1).

- Acceptable dimensional tolerances of concrete box segments significantly reduce jacking loads.
- Painting the outside surface of the concrete box culvert helps the efficiency of the lubrication considerably.

5.2 Recommendations for Future Research

To have more reliable models, it is recommended to prepare a recoded data bank from previous projects with different site conditions. By using statistical analysis of project data and comparison with theoretical models, reliable conclusions on jacking load estimation can be made.

For future research, the following topics are recommended:

- 1- Live load effects on pipe jacking technique with shallow cover depth, which is not covered in the existing code (ASCE 27-00).
- 2- Thrust block capacity calculation and its rotational effects on pipe jacking operations.
- 3- Shield, TBM and cutterhead types and their effects on required jacking loads.
- 4- Settlement control calculations to prevent sudden sinkholes.
- 5- Allowable space between driven pipes in pipe and box jacking methods.
- 6- Curved pipeline jacking and misalignment calculation methods.
- 7- Stability of face and tunnel boring in different soil conditions.

APPENDIX A

PIPE JACKING AND MICROTUNNELING QUESTIONNAIRE



The University of Texas at Arlington

Center for Underground Infrastructure Research and Education (CUIRE)

Pipe Jacking and Microtunneling- Designer Questionnaire

TxDOT

Richard Williammee, P.E.
Phone: 817-370-6675
Email: Richard.Willammee@txdot.gov

Texas Department of Transportation,
Forth Worth District,
Texas 76115-0868

CUIRE

Dr. Mohammad Najafi, P.E.
Phone: (817) 272-0507
E-mail: najafi@uta.edu

The University of Texas at Arlington
428 Nedderman Hall
Arlington, TX 76019-0308

Saeed Rahjoo
Phone: (817) 823-9775
E-mail: saeed.rahjoo@mavs.uta.edu

Instructions:

The University of Texas at Arlington and the Center for Underground Infrastructure Research and Education (CUIRE) are working on a major research project to provide realistic estimations of jacking loads. The primary objective of this project is to gain an understanding of pipe jacking under different site and project conditions. The collected responses will be kept confidential by the researcher to the maximum extent allowable by law. There are no risks or individual benefits associated with taking this survey. Your time and effort in completing this survey will be greatly appreciated and will be acknowledged in the report.

To show our appreciation for your time and effort, we will send you a copy of the research findings upon project completion scheduled for Summer 2012.

If you have any questions or concerns, please feel free to contact Richard Williammee, TxDOT, Forth Worth District at (817)370-6675 E-mail: Richard.Willammee@txdot.gov or CUIRE office at (817)272-9177 or Saeed Rahjoo, Graduate Research Assistant, Phone: (817) 823-9775, E-mail: saeed.rahjoo@mavs.uta.edu or Dr. Mohammad Najafi the Principal Investigator of this project, directly at 817-272-0507, E-mail: najafi@uta.edu

Contact Person Information:

Name			
Company and Position			
Email		Phone	

Project Information (Please use new form for additional projects):

Project Name			
Project Location			
Project Total Cost		Project Duration	

I. Project Information:

1-Type of Project

2- What type of method was used?

If you have the exact value, please type it down on "Others" option.

II. Site Conditions:

1- What was the total drive length of the project (ft)?

2- What was the pipe outside diameter (in.)?

3- What was the average pipe crown depth ranges from the surface¹ (ft)?

4- What was the soil type?

5- What were the soil design parameters? (psi) Liquid(LL), Plastic(PL)

TCP c (cohesion) Phi (friction angle) Atterberg limits

6- What was the average depth of the watertable from the surface (ft) during construction?

¹ Top of the pipe from Surface

* Definitions are provided on page 5

III. Pipe Conditions:

- 1- What was the pipe type?
- 2- What was the pipe wall thickness (in.)?
- 3- What was the size of the overcut* (in.)?
- 4- What was allowable pipe compression stress (psi)?

For the following questions if you cannot provide exact value, rough estimation also, can help.

IV. Additional Information:

- 1- Did you consider lubrication?
- 2- Thrust block design capacity (tons)?
- 3- Capacity of the hydraulics jack (tons)?
- 4- Expected max. jacking force (tons)
 - a) Main (Driving Shaft)
 - b) Intermediate jack station (if any)
- 5- Maximum expected settlement at the surface (in.)

* Definitions are provided on page 5

V. Predicting Model:

- 1- What guideline, standard or famous predicting model do you usually use to calculate required jacking load? (It would be highly appreciated if you could send us a sample of your calculation.)

****IMPORTANT NOTE****

If you have Acrobat Writer please type your answers and then press Submit button. Otherwise, if you have Acrobat Reader, please type your answers and make print of it then scan and send it to us by email.

Submit

* Definitions are provided on page 5

GLOSSARY

Auger boring: Also horizontal auger boring, a technique for forming a bore from a drive pit to a reception pit, by means of a rotating cutting head. Spoil is removed back to the drive shaft by helically wound auger flights rotating in a steel casing. The equipment may have limited steering capability.

Closed face: The ability of a tunnel-boring machine to close or seal the facial opening of the machine to prevent, control, or slow the entering of soils into the machine. Also may be the bulk heading of a hand-dug tunnel to slow or stop the inflow of material.

Earth pressure balance shield: Mechanical tunneling shield that uses a full face to support the ground in front of the shield and usually employs an auger flight to extract the material in a controlled manner.

Microtunneling: A trenchless construction method for installing pipelines. Microtunneling uses all of the following features during construction:

(1) Remote controlled—The microtunneling-boring machine (MTBM) is operated from a control panel, normally located on the surface. The system simultaneously installs pipe as spoil is excavated and removed.

Personnel entry is not required for routine operation.

(2) Guided—The guidance system usually references a laser beam projected onto a target in the MTBM, capable of installing gravity sewers or other types of pipelines to the required tolerance, for line and grade.

(3) Pipe jacked—The process of constructing a pipeline by consecutively pushing pipes and MTBM through the ground using a jacking system for thrust.

(4) Continuously supported—Continuous pressure is provided to the face of the excavation to balance groundwater and earth pressures.

Open-face shield: Shield in which manual excavation is carried out from within a steel tube at the front of a pipe jacking operation.

Overcut: The annular space between the excavated borehole and the outside diameter of the pipe.

Pilot tube microtunneling: Pilot Tube Microtunneling (PTMT), also referred to as guided auger boring, guided boring method, and auger drilling, was introduced in the 1990s. According to the North American Society for Trenchless Technology (NASTT), guided auger boring is defined as "Auger boring systems which are similar to microtunneling, but with the guidance mechanism actuator sited in the drive shaft (e.g. a hydraulic wrench which turns a steel casing with a symmetric face at the cutting head). The term may also be applied to those auger boring systems with rudimentary articulation of the casing near the head activated by rods from the drive pit (NASTT 2002)."

Pipe jacking: A system of directly installing pipes behind a shield machine by hydraulic jacking from a drive shaft, such that the pipes form a continuous string in the ground.

Shield (shield system): means a structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields can be permanent structures or can be designed to be portable and moved along as work progresses.

Slurry: A fluid, mainly water mixed with bentonite and sometimes polymers, used in a closed loop system for the removal of spoil and for the balance of groundwater pressure during tunneling and microtunneling operations.

TCP: Texas Cone Penetration

Tunnel boring machine (TBM): (1) A full-face circular mechanized shield machine, usually of worker-entry diameter, steerable, and with a rotary cutting head. For pipe jacking installation, it leads a string of pipes. It may be controlled from within the shield or remotely such as in microtunneling.

(2) A mechanical excavator used in a tunnel to excavate the front face of the tunnel (mole, tunneling head).

Utility tunneling: It is general approach of constructing underground utility line by removing the excavated soil from the front of cutting face and installing liner segments to form continuous ground support structures. The product pipe is then transported and installed inside the tunnel. The annular space between the liner and the pipe is usually filled with grout.



The University of Texas at Arlington

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Pipe Jacking and Microtunneling- Contractor Questionnaire

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*The University of Texas at Arlington
428 Nedderman Hall
Arlington, TX 76019-0308*

Saeed Rahjoo
Phone: (817) 823-9775
E-mail: saeed.rahjoo@mavs.uta.edu

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Contact Person Information:

Name			
Company and Position			
Email		Phone	

Project Information (Please use new form for additional projects):

Project Name			
Project Location			
Project Total Cost		Project Duration	

I. Project Information:

1-Type of Project

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If you have the exact value, please type it down on "Others" option.

II. Site Conditions:

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3- What was the average pipe crown depth ranges from the surface¹ (ft)?

4- What was the soil type?

5- What were the soil design parameters? (psi) Liquid(LL), Plastic(PL)

TCP c (cohesion) Phi (friction angle) Atterberg limits

6- What was the *average* depth of the watertable from the surface (ft) during construction?

¹ Top of the pipe from Surface

* Definitions are provided on page 5

III. Pipe Conditions:

- 1- What was the pipe type?
- 2- What was the pipe wall thickness (in.)?
- 3- What was the size of the overcut* (in.)?

IV. Trenchless Machine and Equipment Type:

- 1- What type of shield*/TBM* was employed?
- 2- What type of spoil removal system did you use?

V. Additional Information:

- 1- Did you consider lubrication?
- 2- Thrust block design capacity (tons)?
- 3- Capacity of the hydraulics jack (tons)?

For the following questions if you cannot provide exact value, rough estimation also, can help.

VI. Observation Data

- 1- Maximum observed jacking force (tons)
- a) Main (Driving Shaft)

* Definitions are provided on page 5

b) Intermediate jack station (if any)

2- Average thrust rate (in./s)?

3- Average cutting torque (lb. ft)?

4- Maximum observed settlement at the surface (in.)

5- Max. observed deviation from target (mm)

a)Line

b)Grade

****IMPORTANT NOTE****

If you have Acrobat Writer please type your answers and then press Submit button. Otherwise, if you have Acrobat Reader, please type your answers and make print of it then scan and send it to us by email.

Submit

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Open-face shield: Shield in which manual excavation is carried out from within a steel tube at the front of a pipe jacking operation.

Overcut: The annular space between the excavated borehole and the outside diameter of the pipe.

Pilot tube microtunneling: Pilot Tube Microtunneling (PTMT), also referred to as guided auger boring, guided boring method, and auger drilling, was introduced in the 1990s. According to the North American Society for Trenchless Technology (NASTT), guided auger boring is defined as "Auger boring systems which are similar to microtunneling, but with the guidance mechanism actuator sited in the drive shaft (e.g. a hydraulic wrench which turns a steel casing with a symmetric face at the cutting head). The term may also be applied to those auger boring systems with rudimentary articulation of the casing near the head activated by rods from the drive pit (NASTT 2002)."

Pipe jacking: A system of directly installing pipes behind a shield machine by hydraulic jacking from a drive shaft, such that the pipes form a continuous string in the ground.

Shield (shield system): means a structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields can be permanent structures or can be designed to be portable and moved along as work progresses.

Slurry: A fluid, mainly water mixed with bentonite and sometimes polymers, used in a closed loop system for the removal of spoil and for the balance of groundwater pressure during tunneling and microtunneling operations.

TCP: Texas Cone Penetration Test

Tunnel boring machine (TBM): (1) A full-face circular mechanized shield machine, usually of worker-entry diameter, steerable, and with a rotary cutting head. For pipe jacking installation, it leads a string of pipes. It may be controlled from within the shield or remotely such as in microtunneling.

(2) A mechanical excavator used in a tunnel to excavate the front face of the tunnel (mole, tunneling head).

Utility tunneling: It is general approach of constructing underground utility line by removing the excavated soil from the front of cutting face and installing liner segments to form continuous ground support structures. The product pipe is then transported and installed inside the tunnel. The annular space between the liner and the pipe is usually filled with grout.

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