

EFFECT OF BENTONITE SLURRY INJECTION ON PIPE JACKING FORCES

by

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ABSTRACT

EFFECT OF BENTONITE SLURRY INJECTION ON PIPE JACKING FORCES

Pipe jacking is a technique used to form tunnels by pushing prefabricated pipes through the ground from an entrance shaft to an exit shaft. The main limitation for pipe jacking method is limitation of jacking distance. Main aim of this study is trying to advance both the knowledge of the lubricity mechanism at the pipe-soil interface and the technology for lubrication and overcut stability in pipe jacking, in order to upgrade jacking distance by ensuring that lubricants are properly and effectively used, in other words optimization of bentonite injections that is applied for lubrication. For this purpose a research program is started which have laboratory and field stages. Jacking force data of three cases are recorded at the field stage, and at Bogazici University Geotechnical Laboratory, two new laboratory test setups are prepared. At the first laboratory test set up, by applying a constant pressure on 5 cm height bentonite slurry which was poured on sand layer, bentonite slurry injection in pipe jacking operations at the jobsite is tried to be simulated. At the second laboratory test set up by pulling out a concrete pipe sample from sand in condition with and without bentonite slurry around the concrete pipe sample, bentonite slurry injection methods which are commonly used in practice for lubrication are tried to be modeled in laboratory. Interface coefficient of friction results which are obtained from both laboratory tests and field jacking data are evaluated together. As an important result it was determined that with an efficient lubrication, 87% reduction of interface coefficient of friction can be achieved which means 20% reduction of total cost. For this bentonite slurry injection should be done under a constant injection pressure with continuous feeding of bentonite slurry in a pressure closed system. At the end of this study a renovation of bentonite injection system for pipe jacking is recommended.

ÖZET

BENTONİT ENJEKSİYONUNUN BORU İTME KUVVETLERİNE ETKİSİ

Boru itme yöntemi bir itme (giriş) bacasından prefabrike olarak üretilmiş olan boruların hidrolik pistonlarla bir çıkış bacasına kadar sürülmesi yöntemidir. Bu yöntemin en önemli kısıtlayıcı unsuru itme mesafesinin sınırlı olmasıdır. Bu çalışmanın ana amacı boru itme yöntemindeki itme mesafesinin arttırılabilmesi için yağlanma etkisinin oluşturabilmesi amacıyla kullanılan malzeme ve yöntemlerin yerli yerinde uygulanması diğer bir ifadeyle bentonit enjeksiyonun optimizasyonun sağlanması ve mevcut bilgi ve teknolojilerin daha ileri bir aşamaya taşınmasıdır. Sahada üç adet boru itme uygulamasının itme kuvveti değerleri kayıt altına alınırken Boğaziçi Üniversitesi Geoteknik Laboratuvarında da iki yeni test düzeneği kurulmuştur. İlk test düzeneğinde, kum zeminde bentonit enjeksiyonunun temsil edilmesi için pleksiglas bir hücre içinde hazırlanmış olan kum zemin üzerine 5 cm yüksekliğinde dökülen bentonit çamurunun sabit basınç altında kum zemine penetrasyonu gözlemlenmiştir. İkinci test düzeneğinde ise yine pleksiglas bir cam hücre içerisinde hazırlanmış olan kum numune içerisinden beton boruyu temsil eden beton çubuk numunesi, etrafında bentonitin var olduğu ve var olmadığı durumlarda çekilerek, arazide uygulanmakta olan bentonit enjeksiyon yöntemleri laboratuvarında modellenmeye çalışılmıştır. Bu deneyler sonucunda ölçülmüş olan kum zemin ile beton arasındaki ara yüzey sürtünme katsayısı değerleri, arazi uygulamalarında kaydı tutulmuş olan değerlerle birlikte yorumlanmıştır. Bu çalışmanın en önemli sonucu efektif bir bentonit enjeksiyonu ile zemin ile beton boru ara yüzeyindeki sürtünme katsayısının 87% oranında azaltılabileceği ve toplam maliyette en az 20% oranında bir tasarruf sağlanabileceğidir. Bunun için, bentonit enjeksiyonunun, bentonit miktarı ve basıncının sabit tutularak kesintisiz bir şekilde uygulanması gerekmektedir. Bu çalışmanın sonunda ise boru itme yöntemi için yeni bir enjeksiyon sistemi önerilmiştir.

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LIST OF SYMBOLS

B	Half width of the trap door
b	Ideal silo width
c	Cohesion
C	Cover of depth from surface to pipe
C_u	Coefficient of uniformity
C_c	Coefficient of curvature
d	Distance
d_{10}	Effective (characteristic) particle size
D_{50}	Mean sand diameter
De	Pipe external diameter
Db	Excavated bore diameter
D_c	Diameter of concrete sample
F_T	Total jacking force
F_{Fr}	Frictional jacking force
F_{Face}	Penetration jacking force (face force)
F_{net}	Net frictional part of jacking force
G_s	Specific gravity of soil solid
h	Depth of cover
K	Coefficient of soil pressure above the pipe
K_o	Coefficient of lateral earth pressure at rest
L	Length of drive
L_c	Length of concrete sample
LL	Liquid limit
PI	Plasticity index
M	Mean interface shear stress
$N.$	Normal Force
R	Over break ratio
R_n	Normalized roughness
R_{max}	Maximum roughness

T_γ	Tunnel stability number for tunnels in cohesionless soils without surcharge loading
T_s	Tunnel stability number for tunnels for shallow depths
T_c	Tunnel stability number for tunnels in clay
w_n	Natural water content
W	Weight of concrete pipe sample
κ	Coefficient of reduction
δ	Angle of wall friction in plane of shear
μ	Coefficient of viscosity
μ_{int}	Interface friction coefficient
σ_v	Normal stress
σ_H	Horizontal stress
τ	Shear stress
τ_s	Shear resistance of the slurry
Δp	Difference between the slurry pressure p and the ground water pressure u
σ_T	Required support pressure for tunnels in cohesionless soils without surcharge loading
γ	Total unit weight of the soil
τ	Shear stress
ϕ	Angle of internal friction
ψ	Dilation angle of the soil

LIST OF ABBREVIATIONS

<i>IJS</i>	Inter Jacking Station
<i>ASTM</i>	American Society for Testing and Materials

1. INTRODUCTION

1.1. Pipe Jacking Method

Pipe jacking is a technique used to form tunnels by pushing prefabricated pipes through the ground from a starting shaft to an exit shaft. In this method, excavation, which was performed with conventional techniques in the past, is performed by using slurry type machines, so that the environmental damages such as lowering of the underground water level are prevented. In highly populated cities which have intensive infrastructure and heavy traffic flow, construction works can be carried out by this method without disruption.

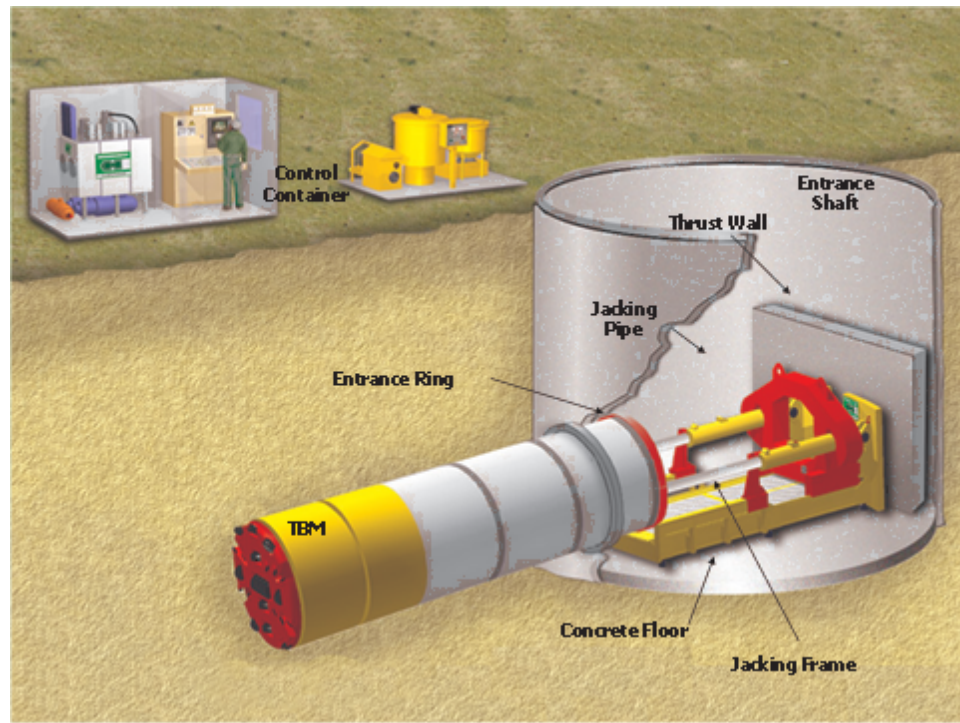


Figure 1.1. Pipe jacking techniques.

The application areas of pipe jacking method can be summarized as below.

- (i) Installation of pressure pipe lines and gravity sewers: The installation of sewers by pipe jacking and microtunnelling can compete directly on price with the

alternatives of deep open cut and traditional tunnelling. Pipe jacking can offer a viable alternative at depths greater than 4 meter or where the installation is under busy roads or close to existing buried service lines.

- (ii) Installation of ducts: Pipe jacking can not only be used for laying deeper the old ducts but also to make a multifunctional duct system which can collect all infrastructure service lines inside, in order to be accessible permanently without any need to make open cut excavation.
- (iii) Installation of crossings lines: Pipe jacking offers possibility to direct install concrete pipes at crossings to provide a permanent line, not only for sewers and culverts, but also for larger structures such as pedestrian underpasses.
- (iv) Replacement of the existing pipe with the new one: This system is named as pipe eating. When the hydraulic diameter of the existing lines becomes inadequate or the line reaches the end of its service life it can be replaced by installing a new or larger diameter pipe.

1.1.1. Advantages of Pipe Jacking Method for Urbanized Areas

- Minimizes risk of settlement,
- Minimizes disruption to the surface structures, traffic, existing sewers, utilities and services during construction,
- Reduces the social cost and easement requirements
- When working in contaminated ground, the only soil excavated is that of the pipe bore.
- Considerably reduces the amount of excavated material to be removed compared to an open cut excavation method, because only the pipe bore is excavated.
- Avoids the need to dewater the pipe trench, thereby significantly reducing the risk of collapse/settlement.
- Prevention of ground water ingress by use of pipes with sealed flexible joints.
- Minimizes reconstruction work to road surfaces and restoration work to buildings.
- Applicable to most soil types and conditions.
- Applicable to any depth.
- Speedier construction

- Reduces the potential of injuries due to open excavations.
- Reduces need for man-entry and improves operator/worker safety, because of remote control capability.
- Accurate installation with line and grade tolerances of better than $+/- 2.5$ cm over even the longest drive lengths.
- Strength of pipe is typically greater than open cut pipe.
- No requirement for secondary lining.
- Provides a cost effective modern solution.

1.1.2. Disadvantages and Limitations of Pipe Jacking Method

- When work is in expanding clay, high frictional loads can develop which limit jacked drive length. The segmental lining is more suitable in such conditions.
- Stoppage during pipe jacking can cause blockage of the installation.
- It is not possible to make a rapid change of line, level, or introduce a bend in to the line. Curve capability is very limited without special measures.
- Especially for microtunneling in non-person entry diameters, it is not possible to add an interjacking station, span length is controlled by the maximum safe end load that can be applied to the pipe. This means that drive length is limited by the pipe diameter and safe end load.

In comparison with other methods, the following issues typically apply:

- (i) As difficulty increase with increasing depth in cut and cover constructions, pipe jacking has greater advantages over this traditional method. Pipe jacking method becomes a cheaper solution, especially for depths greater than 4 m and in unstable ground conditions which requires sheet pile installation to make a safe excavation. Therefore pipe jacking provides a cost-effective solution specially at big depths.
- (ii) Pipe jacking is designed to provide a continuous lining as the operation advances, without significant overcut of the soil, so that movements is greatly reduced or avoided altogether. Hence, a major advantage of pipe jacking is that work can be carried out with minimum soil cover such as 1 meter or less. In traditional

tunneling, the segmental lining is placed within the tail of the shield. This leaves an annular gap between the excavated tunnel diameter and the outside diameter of the lining. Where the ground is stable this void can be filled with stabilizing grout. However, in unstable ground such as loose sand, it is difficult, if not impossible to avoid ground moving into the space before the grouting is done and thereby causing settlement. To cause minimal damage to property, existing utilities and adjacent structures on the surface pipe jacking is very advantageous.

- (iii) If the location of jacking shafts is carefully designed and planned the disruption of pipe jacking to the public and social life will be limited. It has become a giant environmental problem to make storage of excavated material at the end of open trench excavation. Trenchless methods have a big advantage because of creating less amount of excavated material when compared to open trench excavations.
- (iv) It is remotely controlled and during installation, fewer places are kept open so that it is very safe not only for workers but also for public.
- (v) Traditional tunneling installs a segmental liner which is not suitable for use as a finished pipeline; therefore it is generally necessary to introduce a secondary liner. But pipe jacking does not require this kind of a secondary liner.

1.2. Understanding Mechanisms that Govern Jacking Forces

The total jacking load is the force required to overcome frictional resistance of the pipe string and face resistance at the shield. Accurate prediction is important to the design of the jacking pit and thrust wall; selection of jacking equipment (jacking frame and main rams) and jacking pipe wall thickness; the need for and placement of intermediate jacking stations; and lubrication requirements. Penetration or face resistance opposes the advance of the shield and vanes according to its shape and boring action. For open face operations (hand excavation and auger or cutter heads) the resisting force is primarily related to the diameter of the cutting edge of the shield. To avoid settlement or heave of the ground the boring head contact pressure should be equal to the lateral earth pressure at rest- hydraulic pressure should be set to match the hydrostatic pressure of the groundwater (Marshall, 1998).

1.3. Importance of Predicting Jacking Forces

Predicting jacking forces is important for planning, design, and construction phases of microtunneling projects. The construction of the microtunneling shafts has a significant economic impact on the total cost of the pipeline installation project. For pipes with diameters less than 900 millimeters the maximum length of the tunneling drive is determined by the maximum jacking capacity and this, in turn, dictates shaft spacing. The engineer must predict the jacking forces to determine the maximum spacing of the jacking shafts during the design process. Often the microtunneling alternative will be compared to the open-cut alternative. The microtunneling alternative may be eliminated as a feasible construction alternative from a cost perspective if the number of shafts cannot be minimized (Staheli, 2006).

In pipe diameters greater than 900 millimeters, intermediate jacking stations (IJS) can be used, as shown in Figure 1.2. An IJS is a series of hydraulic jacks that can be incorporated into the pipeline at a distance behind the microtunneling machine. The hydraulic jacks in the IJS propel the machine and the pipes between the machine and the IJS forward. The main jacks located in the jacking shaft are then activated to propel the pipes that have been installed after the IJS. In this fashion, the IJS serves to inchworm the pipe-string forward, effectively isolating the frictional component of the jacking force in particular sections of the pipeline. IJS shells are fabricated to the exact outer diameter of the jacking pipe and are machined to exert the thrust from the hydraulic jacks to the load-bearing end of the jacking pipe. IJS's are long lead-time items and must be ordered well in advance of construction. The contractor must predict the jacking forces to determine the number of IJS's needed on any particular tunnel drive (Staheli, 2006).



Figure 1.2. Interjack station.

Microtunneling jacking shafts must also be structurally designed to withstand the jacking forces. As the main jacks push the microtunneling machine and pipeline into the ground, a thrust wall, located behind the main jacking system, distributes the reactionary force into the walls of the shaft and the ground surrounding the shaft. If the shaft is not designed adequately to support the jacking forces, shaft failure can occur. Jacking force predictions are therefore crucial to adequately design thrust blocks and jacking shafts to avoid shaft failures (Staheli, 2006).

1.4. Importance of Reducing Jacking Forces

As mentioned above, pipe jacking is a technique used to install underground pipelines through a borehole excavated by a shield-type boring machine, which operates hydraulically from a launching shaft. Slurry pipe jacking is firmly established as a special method for non-disruptive construction of underground pipelines. During the pushing processes, the mud slurry is injected into the face and over-cutting area surrounding the pipes. After completion of the pipeline, the mortar is injected into the

over-cutting area in order to provide permanent stability of the surrounding soil.

During the jacking process, the jacking load is an important parameter, controlling the pipe wall thickness, location of intermediate jacking stations, selection of the jacking frame and lubrication requirements. The total jacking force required to propel the tunneling machine and pipe sections forward must overcome forces associated with face pressure on the machine and friction on the machine and pipeline. The face pressure force acts at the front of the machine and originates from ground water and earth pressures. The frictional force develops between the surrounding soil and the exposed outer surface area of the tunneling machine and installed pipe sections. The face pressure component relates to the depth of burial and is estimated based on the soil and groundwater conditions at the site. The face pressure component of the jacking force remains theoretically constant if the depth of soil over the pipeline is constant. However, the frictional force increases as the drive length increases. As a result, longer drives require greater jacking forces.

Microtunneling is a method of tunneling and is not limited by size. Typical microtunneling diameters range from 600 to 3600 millimeters. The microtunnel machine excavates a slightly larger diameter hole than the diameter of the installed pipe sections. The distance between the maximum excavated diameter and the outer diameter of the installed pipe sections is referred to as the over cut or annular space. Over cuts of between 20 and 50 millimeters on the diameter are typical. The over cut is necessary to reduce frictional forces, facilitate steering of the tunneling machine, and to allow injection of lubrication into the annular space if required.

Lubrication can effectively reduce the needed thrust if a discrete layer of the lubricant is maintained between the pipe and the excavated soil. Lower frictional force allows larger jacking lengths to be achieved. Reduction of frictional stress around the pipes is closely related to the efficiency of lubrication injection. The lubricant must be designed so that it can form a layer in the surrounding soil, be pressurized to overcome ground water pressure and stabilize the over-cutting area. Lubricant should fill the complete over-break void to minimize surface settlement (Staheli, 2006).

1.5. Scope of Dissertation

The benefits of bentonite lubricants in maintaining tunnel bores and reducing frictional resistance have been recognized but there is still little attempt to standardize their use. Many of authors, by examination of total jacking loads only, have demonstrated that lubrication is beneficial and can reduce frictional resistance. To better understand the function of lubrication further work was required to investigate their effects on pipe-soil interaction. One of the main aim of this thesis is improving the knowledge of the lubricity mechanism at the pipe-soil interface in order to ensure that lubricants are properly and effectively used.

The research that reported in this thesis aims to advance both the knowledge of the lubricity mechanism at the pipe-soil interface and the technology for lubrication and overcut stability in pipe jacking and slurry machines.

This thesis presents the results of a study focused on the identification of the lubrication mechanisms that control interface shearing between jacking pipes and sand. Bentonite injection methods which are commonly used in practice for lubrication are modeled in laboratory conditions. Coefficient of friction results which are obtained from laboratory test are compared with jobsite results. At the end of this study, a renovation of bentonite injection system for appropriate lubrication during pipe jacking is recommended.

Prior to performing the research, a review of previous work was conducted. This work, including not only studies on interface friction, normal stress distributions around tunnels, and the prediction and control of jacking forces but also rheological studies about slurries which are summarized in Chapter 2.

Methodology of the laboratory tests are mentonined in Chapter 3. Observation of bentonite slurry injection in sand and measurement of interface friction coefficients between soil and concrete sample with and without bentonite are mentioned. Especially, the bentonite slurry injection cases in the jobsite applications are tried to be

modeled.

In Chapter 4 three jobsite cases are mentioned and critised from viewpoint of effect of bentonite slurry injection.

In Chapter 5 both coefficient of friction at the jobsite and at laboratory are calculated. Also, three representative laboratory tests are compared with three different jobsite case data.

In Chapter 6 not only results about importance and efficiency of bentonite injection are summarized, but also a renovation for bentonite injection during pipe jacking operation is recommended.

2. LITERATURE REVIEW

2.1. Introduction

The development of pipe jacking technology has been evolutionary and recent, dating from its inception in Japan in the mid 1970s to its further development and refinement in Germany, beginning in the early 1980s and in the mid-to-late 1980s in the UK. Although pipe jacking method was applied with open face shields in mid 1990s, it has been started to be applied with machine after 2000 in Turkey.

The trenchless technique of pipe jacking allows installing underground ducts and pipelines with diameters ranging from 0.5 to approximately 3 meters without causing excessive surface disruption. As illustrated in Figure 2.1, the technique involves pushing a string of pipes through the ground from a launch shaft towards a reception shaft, while excavating the soil at the front end using either a mechanized or manually-operated excavation system. As the excavation progresses, pipes are placed one by one into the launch shaft, added at the rear end of the pipe string and pushed forward a distance allowing insertion of the next pipe. This cycle is repeated until the pipe string breaks through at the reception shaft, which may lie a distance of 50 to 500 m from the launch shaft (François, 2006).

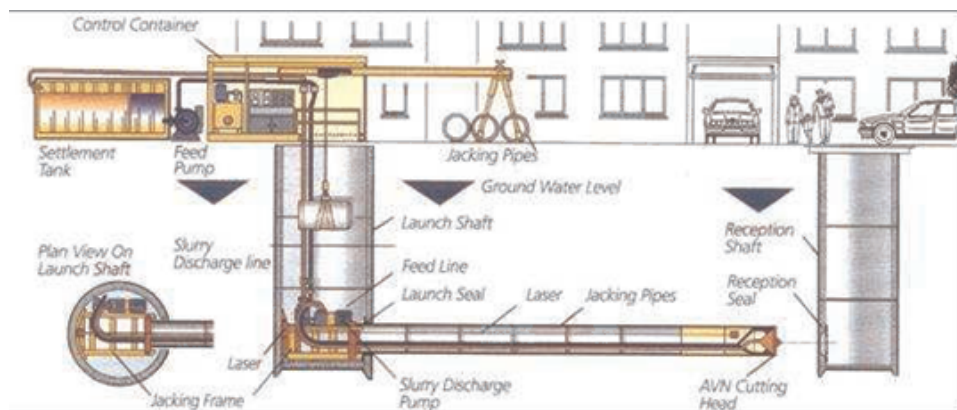


Figure 2.1. Pipe jacking method profile (François, 2006).

Also pipe jacking method can be used instead of shield method for large diameter tunnels up to 3,5 meters internal diameter. If pipe jacking is used instead of shield method for a large diameter tunneling, it can use how to deal with the lubricant at over-cutting area and what kind of the monitoring system for references. Under this situation, it is necessary to use the theory of the stability of surrounding soil, the thrust prediction and the seepage behavior of the mud slurry. During pipe jacking and micro-tunneling process, jacking load is an important parameter, as well as the controlling of the pipe wall thickness, the need for and location of an intermediate jacking station, the selection of jacking frame and lubrication requirements (François, 2006).

2.2. Jacking Forces

Jacking forces are comprised of two components: the face pressure force and the frictional force, as schematically shown in Figure 2.2. The face pressure force is made up of the earth and fluid pressure acting on the face of the machine. Microtunneling machines are designed with the intent to be operated in a pressure-balance fashion. The operator pumps pressurized slurry to the face of the machine to counterbalance the groundwater pressure. Further, the rate of advance of the machine theoretically balances the excavation rate so that neither settlement nor heave of the ground surface occurs. For these conditions to be met, the face pressure force acting on the machine remains between the active and passive earth pressure acting over the area of the face of the machine. However, in practice, the face pressure force is highly dependent on the operation of the machine. Operators tend to find a balance between maximizing production rates without expending all of the available torque required to turn the cutting wheel at the face of the machine (Staheli, 2006).

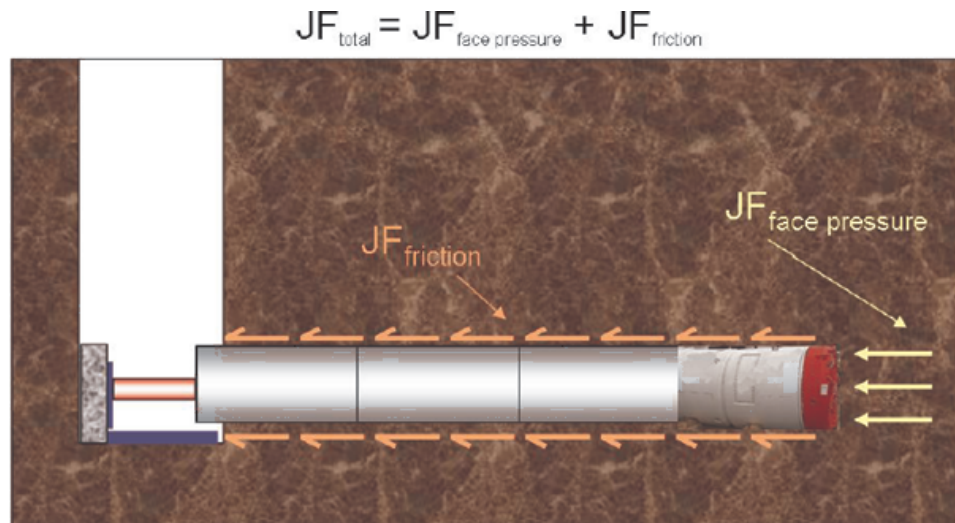


Figure 2.2. Components of jacking force (Staheli, 2006).

If the operational speed is too fast, face pressure forces will increase, resulting in increased torque on the head causing the machine to stall. If the operational speed is too slow, torque readings will be low and material will tend to slough into the machine resulting in over-excavation. This has the potential to manifest as settlement at the ground surface, depending on ground conditions and depth of cover (Staheli, 2006).

A great amount of effort has been spent trying to predict face pressure forces. In general, field experience shows that face pressure forces represent a very low percentage of the overall jacking force. They are also the primary indicator used by the machine operator to determine advance rates. As such, the operator has the ability, by adjusting the machine speed, to control the face pressure force. Therefore, it may not be worthwhile to further refine these predictions. Therefore, most operators tend to drive the machine as fast as possible without pushing the machine beyond the reasonable limits of the torque capacity, causing the machine to stall. As a result, the face pressure acting on the face of the machine is closer to the active than the passive earth pressure (Staheli, 2006).

Frictional forces on the microtunneling machine and pipeline have been much less understood than face pressure forces, although there is general agreement that the

frictional component of the jacking forces should follow the general formula,

$$F_{Friction} = \mu_{int}N \quad (2.1)$$

where μ_{int} is the coefficient of friction and N is the normal force. There is debate over the appropriate values for the coefficient of friction and the method used to calculate the normal force. The vast majority of the work that has been performed on microtunneling jacking forces has been at a macro-scale and has focused on evaluation of field data, taking an empirical approach to developing models to determine jacking forces (Staheli, 2006).

To estimate the friction coefficients, the vast majority of the models focus on the friction angle of the soil (φ'), and use some type of reducing factor on the soil internal friction angle to develop the friction coefficient (from $1/4\varphi'$ to $3/4\varphi'$). Others have published values for interface friction coefficients between specific pipe materials and specific soils. However, these published coefficients of friction are based on a limited number of field case history evaluations and do not identify the mechanism of shearing at the interface. Since soil internal friction angles have a relatively small range, values chosen in most of the models do not have a significant impact on the overall calculation of jacking force compared to the accuracy of the calculation method (on the order of 15 to 25%) (Staheli, 2006).

Significant variation in jacking force calculation models have existed in the calculation of the normal forces. Very little is understood about the mechanisms that control the arching in the soil when pipe jacking is occurring. As a result, there is a wide range of approaches used to calculate the normal force in jacking force models that have been developed to date. Most of the models have some base in Terzaghi's Arching Theory; however, some do not allow for soil arching at all, and base the normal force on the soil unit weight and the depth of cover.

Since the mechanisms surrounding both the friction and the normal force are not well understood, until one of these variables is better understood, the relative influence

of the other will remain unknown.

2.2.1. Frictional Resistance

Frictional resistance results from the integration of skin friction acting on areas of the external surface of pipe string and shield. In the U.K. The mean interface shear stresses assembled by (Craig, 1983), Table 2.2, have typically been used to predict jacking forces, the experience of German practice (Stein et al, loc cit) is also reproduced in the table. The values contained in Table 2.1 are of too broad a range to be of practical use and should not be used with any degree of confidence for design purposes. Also, frictional resistance should be quoted in terms of force per metre because mean shear stress implies uniform contact around the pipeline and contact stress data from Marshall's research generally show non-uniform contact around the instrumented pipe (Marshall, 1998).

Table 2.1. Typical values of mean interface shear stresses (Marshall, 1998).

Ground Type	Mean interface shear stress (kPa)			
	UK (Craig, 1983)	Germany (Stein etal, 1989)	France (Craig, 1983)	Australia (Craig, 1983)
Rock	2-3			1
Boulder clay	5-18	3-18		
Firm clay	5-20	5-9	8 - 10	5-8
Wet sand	10-15	2-16		13
Silt	5-20	5-9	17	
Dry dense sand		1-7		
Dry loose sand	25 -45		20-30	
Fill	up to 45			
Dense gravel		2-6	50	

By using the mean interface shear stress (M) Stein offers to calculate frictional

resistance force below;

$$F_{fr} = M\pi D_e L \quad (2.2)$$

In the equation above D_e is pipe external diameter, L is the length of drive, M is mean interface shear stress. The principal parameters identified as affecting frictional resistance may be categorized as site-related and construction-related (Marshall, 1998).

Forces determined by site conditions include:

- Surcharge load and transient live loading
- Type of ground and variation along the pipeline
- Primary load from the ground
- Position of the water table
- Stability of the soil

Construction-related factors include:

- Amount of over break during excavation
- Use of lubricant in the over break void
- Use of intermediate jacking stations
- Pipeline misalignment
- Frequency and duration of stoppages

2.3. Factors Influencing Jacking Loads

2.3.1. Soil Stability

The ability of the soil to support itself is an important factor in frictional resistance. If a soil collapses onto the pipe string, friction loads will increase and could result in failure of the pipe jack. For tunnels in cohesionless soils without surcharge

loading, the required support pressure is given by;

$$\sigma_T = \gamma D_e T_\gamma \quad (2.3)$$

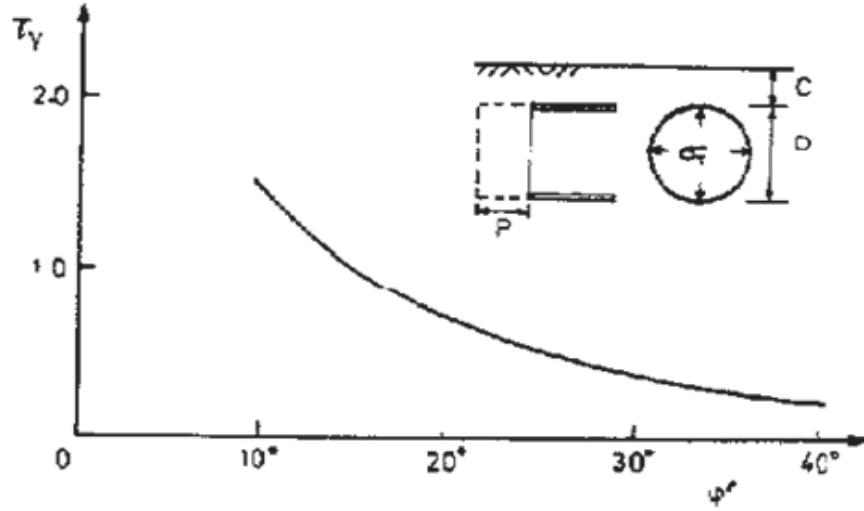


Figure 2.3. Relationship between tunnel stability number T_γ and drained angle of internal friction φ' without surcharge (Marshall, 1998).

where T_γ is the stability number given in Figure 2.3 and is only a function of the angle of friction for the soil if the tunnel is at shallow depth and a large surcharge acts on the surface, the weight of soil may be neglected and the stability number T_s is given in Figure 2.4. Both solutions apply to dry soil, if water pressure is present it is added to σ_T and the buoyant weight, $(\gamma - \gamma_w)$, should be used in Equation 2.3 for soil below the water table (Marshall, 1998).

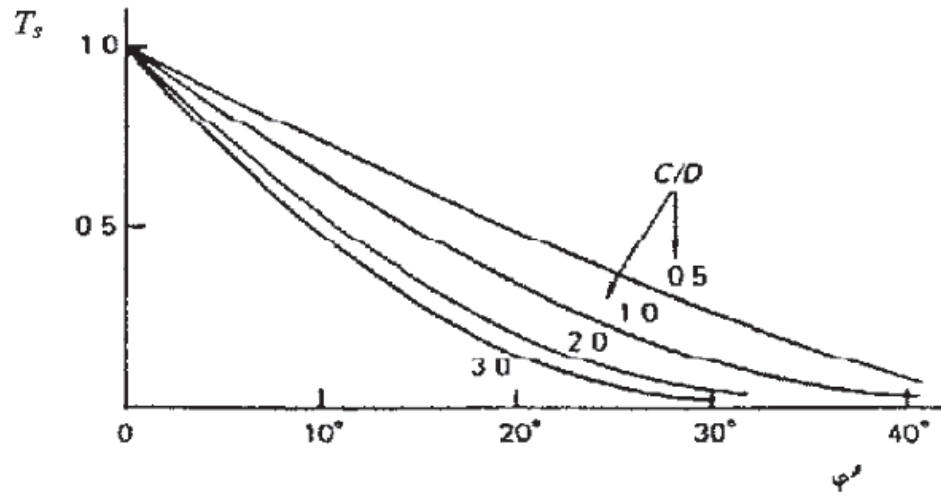


Figure 2.4. Relationship between tunnel stability number T_s and drained angle of internal friction ϕ' - shallow depth with surcharge (Marshall, 1998).

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

$$\sigma_T > \gamma(C + D_e/2) - T_c C u \quad (2.4)$$

where the stability ratio T_c is given in Figure 2.5. In pipe jacking the unsupported length P is usually small or zero, and $P/D = 0$ (Marshall, 1998).

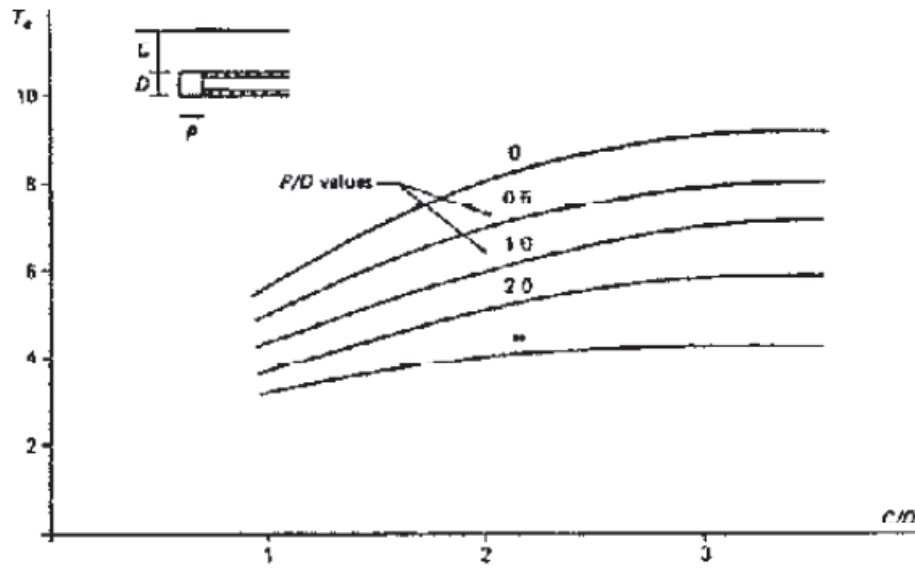


Figure 2.5. Face stability ratio in cohesive soils (Marshall, 1998).

2.3.2. Over Break

The over break is the annular gap between excavated tunnel bore and the outer pipe wall and is typically 10 – 15 mm on radius. The over break ratio is given by ;

$$R = (Db - De)/De \quad (2.5)$$

where Db is the excavated bore diameter and De is the external pipe diameter.

In experimental pipe jacks in sand at Loughborough University, Chapman (loc cit) reports significant draw-along of soil at the pipe-soil interface. (Rogers and Yonan, 1992) suggest a minimum over break of 10 mm to minimize friction caused by the draw along effect in granular soils. The tests at Loughborough used 200 mm diameter pipes installed in dry sands with over break ratios varying between 0 and 0.14. The optimum over break ratio was found to be 0.04, which equates to a relatively large 20 mm over break (on radius) on a 1 m diameter pipe. Forces were reported as remaining low as the ratio was increased, but increased significantly as the ratio reduced to zero (Marshall, 1998).

2.3.3. Lubrication

The technical viability and the cost-effectiveness of pipe jacking predominantly depend on the magnitude of the jacking forces. They determine the required capacity of the jacking rig, the design of the thrust wall reacting the jacking forces and the stresses in the concrete segments. When jacking forces cause the capacity of one of these elements to be exceeded, the distance between the launch and the reception shafts has to be reduced, or intermediate jacking stations have to be inserted at regular intervals in the pipe string to overcome excessive jacking forces. Excavating additional shafts may not only be too expensive but also causes surface disruption, which may not be acceptable or possible in certain environments. Intermediate jacking stations carry operational complications which also result in a significant increase of the overall project cost as well as a significant increase in construction time (François, 2006).

Effective lubrication could greatly reduce jacking forces, allowing significantly longer drives to be achieved. In additions, overcut ratios around the pipes could be decreased, thus reducing the impact of pipe jacking in terms of ground and structure settlements around the pipes.

The use of bentonite suspensions as lubricants in pipe jacking is now common practice. Lubrication can only work effectively if a discrete layer of the lubricant is maintained between the sliding surfaces of the pipe and the excavated soil surface. If the ground collapses onto the pipe, the effect of lubrication will be greatly reduced. The first and most important function of bentonite suspensions or other lubricant is therefore to provide sufficient internal pressure to stabilize the tunnel bore. The slurry must be designed to form a filter cake in the surrounding soil without excessive bleeding of material and be pressurized to the necessary level to overcome ground water pressures and stabilize the tunnel bore; it must fill the complete over break void before this can be achieved. Filling the over break to maintain the bore is also assumed to minimize settlement in the short-term and possibly in the long-term also (Marshall, 1998).

At the time of writing the benefits of bentonite lubricants in maintaining tunnel bores and reducing frictional resistance have been recognized but there is still little attempt to standardize their use. Only by examination of total jacking loads, it is demonstrated that lubrication is beneficial and can reduce frictional resistance. To better understand the function of lubrication further work was required to investigate their effects on pipe-soil interaction. Improving the knowledge of the lubricity mechanism at the pipe-soil interface will ensure that lubricants are properly and effectively used in the future.

2.3.4. Pipeline Misalignment

Inevitably, the excavation of a pipe jacked tunnel will deviate from the intended line and level by some degree. Steering jacks within pipe jacking shields are used to make corrections and with good control line and level can be maintained within typical specified tolerances of 50 or 75 mm. Resulting wiggle of the pipeline is usually a series of shallow oscillations about the true line. A misaligned pipeline will result in increases in total jacking load by introducing a series of radial forces acting on the pipes (Marshall, 1998).

2.3.5. Stoppages

It has been widely reported that stoppages in cohesive soils will create restart jacking loads greater than those needed to maintain steady motion. Interruptions in pipe jacking, such as the introduction of a new pipe, may be accompanied by considerable increases in axial load that must be considered when calculating jacking loads. Few authors have published data quantifying the stoppage phenomenon because records from many schemes plot only jacking forces that have stabilized. Restart jacking loads on a 1,8 m tunnel in alluvium, (Rogers et.al, 1989), were typically 50% greater than the average jacking resistance after relatively short stoppages. After longer weekend breaks, restart forces were as much as 150% higher the peaks usually associated with large restart loads are not evident in the jacking record for this drive (Marshall, 1998). There is also a case observation in chapter 4 about stoppage effect.

2.4. Frictional Studies

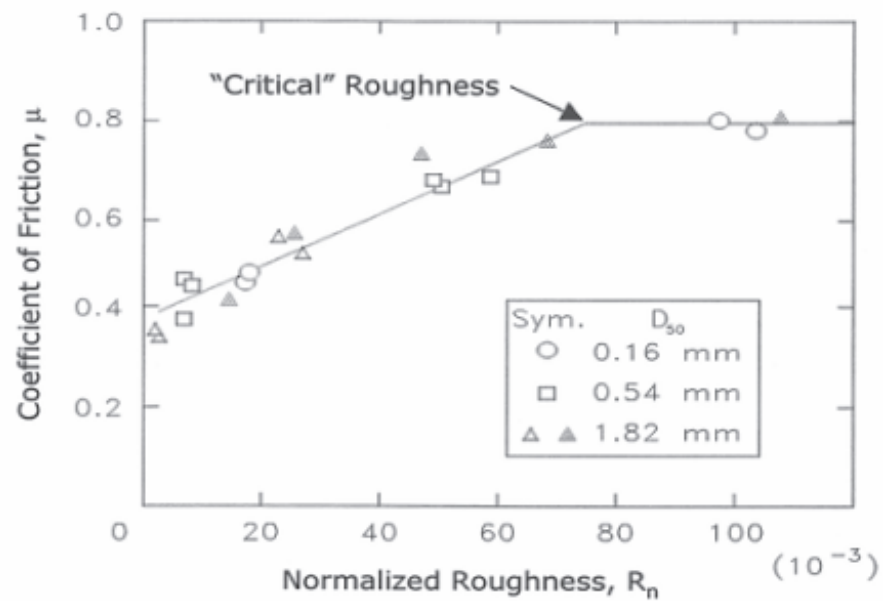
Research on interface strength has been conducted since the early 1940's. Early research conducted by (Potyondy, 1961) concluded that three major factors had an effect on interface strength: the moisture content and composition of the soil, the continuum surface roughness, and the normal load. To analyze interface frictional behavior between sands and a variety of roughened surfaces, (Brumund and Leonards, 1973) developed a cylindrical apparatus that they filled with sand. They then pulled seven different textured rods from the cylinder and measured the interface strength. They were able to conclude that interface strength increased with surface roughness until the strength was equal to the internal friction angle of the sand. By testing two different types of sand, they were also able to conclude that the size, angularity, and surface texture of the sand have an effect on the interface friction angle (Staheli, 2006).

(Uesugi and Kishida, 1986-a) made tremendous advances in relating interface friction to surface roughness when they tested five granular materials against a set of mild steel plates in a simple shear apparatus. Each of the mild steel plates had different surface roughness characteristics. (Uesugi and Kishida, 1986) found that the average soil particle diameter, roughness of the steel plate, density of the material, and the type of sand were significant factors affecting the interface friction. A statistical analysis of their data revealed a relationship between the average particle size of the particulate material and the roughness of the continuum that the particle was contacting, leading them to propose a normalized roughness factor (R_n , 1986-b). When the normalized roughness was analyzed, the relationship between normalized roughness and interface friction proved to be bilinear, revealing a critical roughness as can be seen in Figure 2.6. At values of surface roughness below the critical roughness, the interface friction increased in a linear fashion up to the critical roughness value. However, once the continuum surface reached the critical roughness value, further increases in continuum roughness did not result in higher interface friction. This critical roughness represented an interface friction value that was equal to the internal friction angle of the soil that was being sheared against the interface (Staheli, 2006).

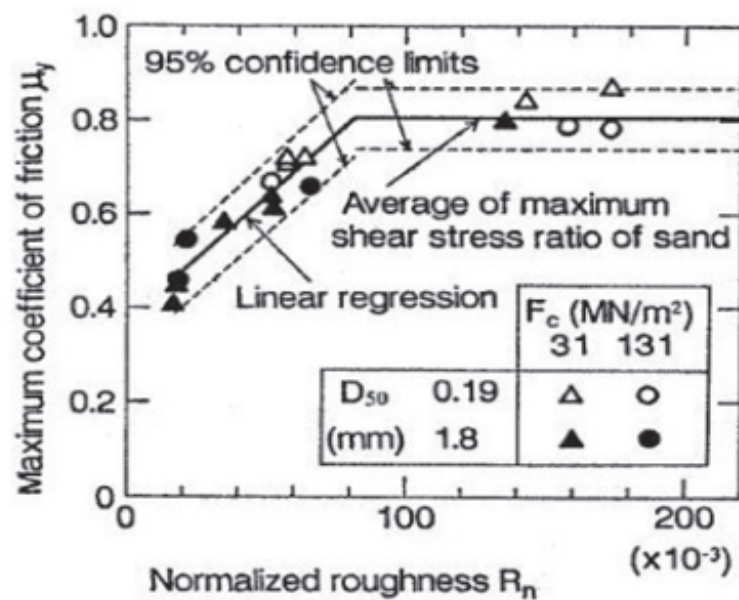
According to Uesugi and Kishiada the influential factors of sand-concrete friction are concrete roughness and mean diameter of sand. neither concrete strength nor sand type affects yielding (peak)friction coefficient. Hence, they defined a normalized roughness R_n which is equal to R_{max} divided by (D_{50}) mean sand diameter. when R_n is rough enough (R_n bigger than 130) peak friction coefficient reaches shear stress ratio of sand and shear yield occurs within the sand mass not at the interface of sand-concrete (Staheli, 2006).

Results of Uesugi

- The characteristics of sand-concrete friction is similar to those of the friction between sand and rough steel surfaces (Figure 2.6).
- The maximum coefficient of friction depends on concrete roughness and mean diameter of sand. the upper limit of maximum coefficient of friction is the maximum shear stress ratio of the sand dependent on the relative density.



(i) Normalized surface roughness vs. friction coefficient for sand-steel interfaces (Staheli, 2006)



(ii) Normalized surface roughness vs. friction coefficient for sand-concrete interfaces

Figure 2.6. Normalized surface roughness vs. friction coefficient for sand-steel interfaces (Staheli, 2006).

2.5. Theories Governing Normal Stresses on the Pipeline

When determining the normal load acting on the pipeline during tunneling, microtunneling, or pipe jacking, the most widely accepted theory of how the soil stresses are distributed on the pipeline is that presented by Karl Terzaghi in his arching theory (Terzaghi, 1943). Terzaghi performed experiments in which he layered sands within a large box that contained a small trap door in the base of the box. He then slowly removed the trap door at the base and measured the stresses when the soil began to yield. Terzaghi found large decreases in the vertical stresses for very small displacements of the trap door. He attributed this phenomenon to arching in the soil mass above the door. The pressure decrease on the trap door was equal to the vertical component of the shearing resistance that acted on the boundaries. Figure 2.7 shows a representation of Terzaghi's Trap Door model (Staheli, 2006).

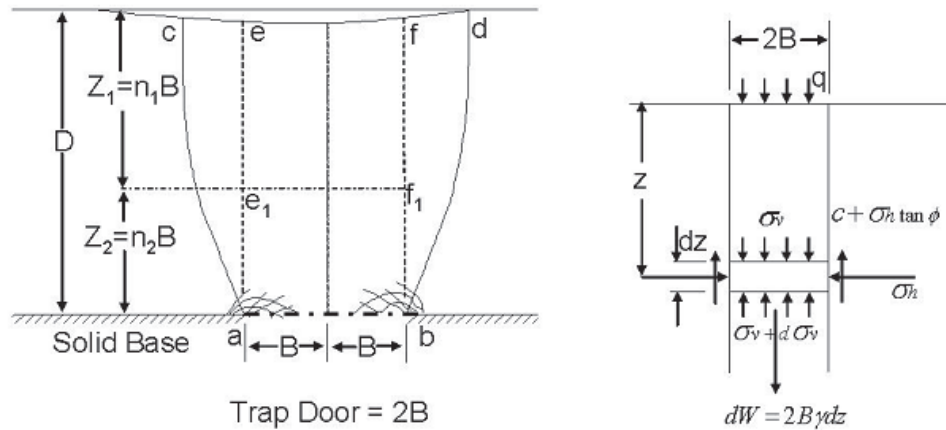


Figure 2.7. Terzaghi's trap door model (Staheli, 2006).

Terzaghi then developed equations for the vertical stress acting on the trap door at a depth of z , based on the width of the trap door, $2B$. For ideal sand (cohesion equal to zero) without any surcharge loading, he found that as the vertical stress to be independent of the depth and derived the following equation:

$$\sigma_v = \sigma_{v\infty} = \gamma B / (K \tan \phi) \quad (2.6)$$

Experimental investigations performed by Terzaghi indicated that $K=1$ above the centerline of the trap door up to a maximum of 1.5 at a distance of $2B$ above the centerline. At a distance of $5B$, lowering the trap door had no effect on the state of stress of the soil.

Many tunneling researchers have used this research performed by Terzaghi and applied it to calculating 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 (Staheli, 2006).

Terzaghi method indicates that the frictional resistance is not dependent on the depth of cover after a certain depth as a result of the soil arching theory. Where the height of the arch is less than the depth of cover, the height of the arch is the effective weight on the pipe. The depth of cover is not a decisive factor when it is more than the height of the arch.

(Auld, 1982) presented a model for calculating normal stresses based on Terzaghi's Trap Door Model. Auld's model represents a pipe driven through cohesionless soils, and was based on the assumption that the soil would collapse onto the pipeline exerting a radial stress around the circumference of the pipeline. Figure 2.8 shows the basis for Auld's model. The model takes into account the arching of the soil above the pipeline and applies the Terzaghi's Arching Theory to a pipe diameter with the following equations (Staheli, 2006):

$$\sigma_\nu = \frac{\gamma B}{K \tan \phi} (1 - e^{-k \tan \phi H / \phi}) \quad (2.7)$$

$$\sigma_H = 0.3\gamma(0.5D + D\sigma_\nu) \quad (2.8)$$

$$P_{Total} = \frac{\pi D}{2} (\sigma_\nu + \sigma_H) \quad (2.9)$$

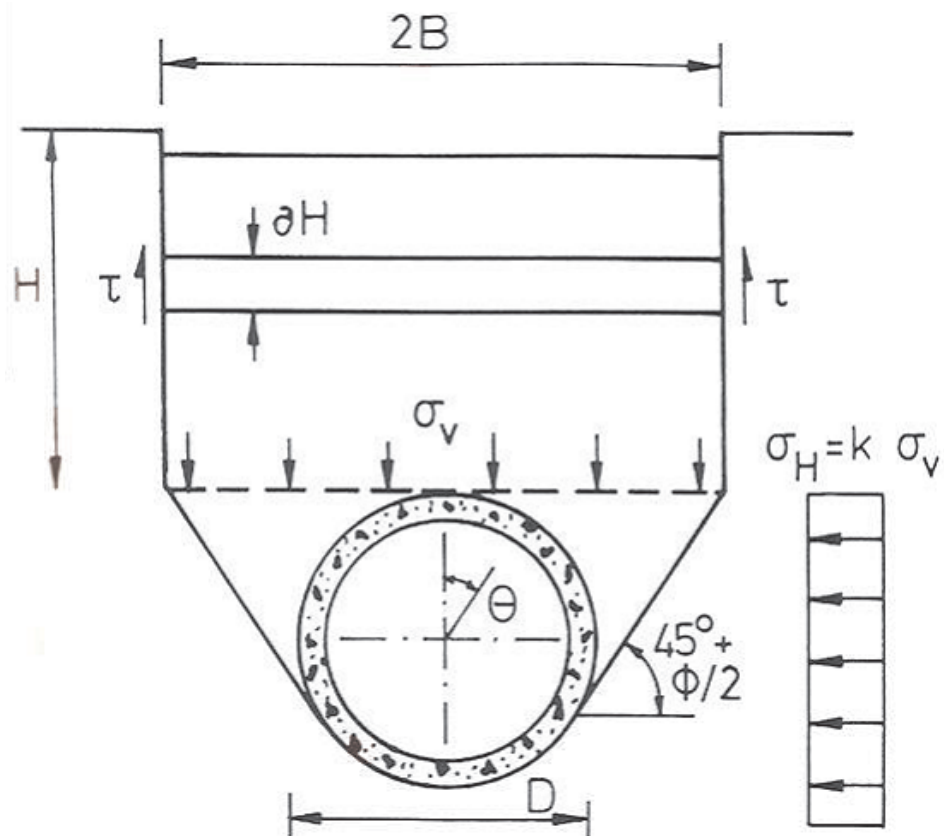


Figure 2.8. Distribution of normal stresses according to auld (Staheli, 2006).

Standards have been established in Germany for calculating the normal load on pipes for microtunneling projects for pipe design purposes. These standards, similar to ASTM standards, are set forth specifically for microtunneling applications. The German standards covering microtunneling pipe design are found in ATV A 161. (Stein, et. al 1989) These standards recommend using a silo width (b) according to Figure 2.9, that is very similar to the model proposed by Auld (Staheli, 2006).

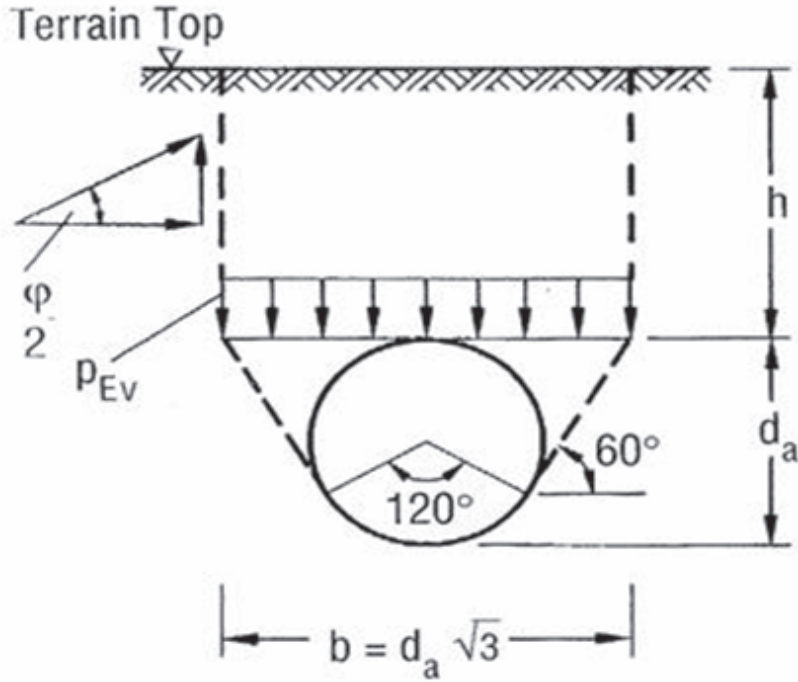


Figure 2.9. Recommended area for calculating normal stresses on microtunneled pipelines according to ATV A161 (Staheli, 2006).

The German standard ATV A161 recommends using the following formula to calculate the vertical pressure P_{EV} acting on the pipeline;

$$P_{EV} = \frac{b(\gamma - \frac{2c}{b})}{2K \tan \delta} (1 - e^{-2K \tan \delta \frac{h}{b}}) \quad (2.10)$$

Where:

d = pipe diameter

b = ideal silo width = $\sqrt{3}d$

h = depth of cover

K = coefficient of soil pressure above the pipe

δ = angle of wall friction in plane of shear

c = cohesion

Figure 2.10 shows a variety of interpretations and applications of Terzaghi's arch-

ing theory as it has been applied by various authors to calculate normal loads acting on microtunneled pipelines

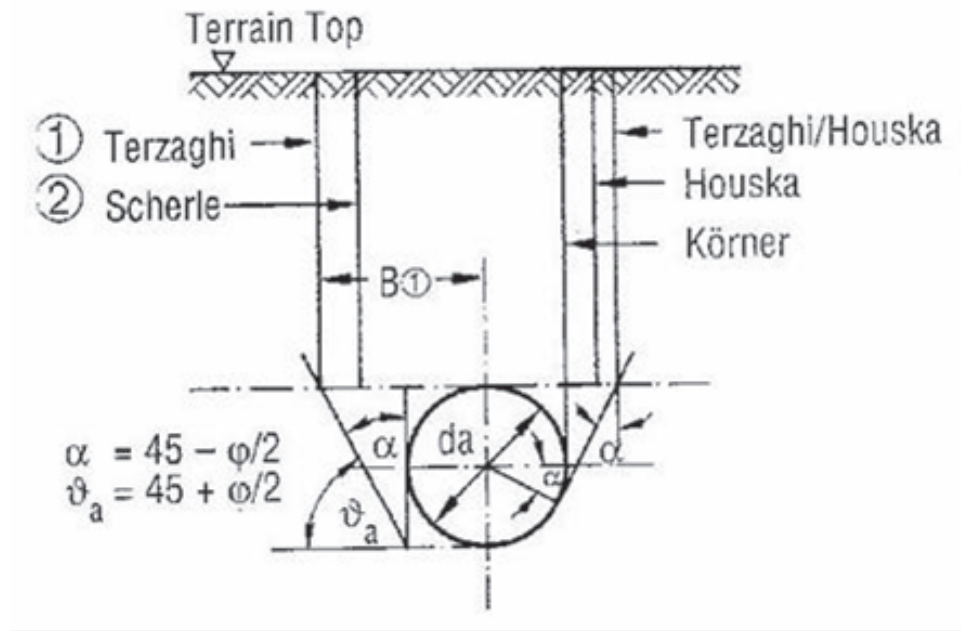


Figure 2.10. Applications of Terzaghi's arching theory by a variety of authors for the calculation of normal stresses (Staheli, 2006).

2.6. Research on Normal Stress Distributions Around Jacking Pipes

A number of practitioners and researchers have conducted studies on normal stress distribution around jacking pipes during pipe jacking operations. These have included field studies, numerical modeling studies, centrifuge modeling studies, and evaluations with critical state soil mechanics using the CAM-CLAY model for pipe jacking in cohesive materials. A significant amount of research was conducted at the University of Oxford under the direction of Dr. George Milligan. Results from three of his students, (Norris, 1992), (Marshall and Zhou, 1998) are discussed herein.

2.6.1. Field Studies Conducted by Norris

Milligan and Norris conducted field studies at the University of Oxford from the late 1980's through the 1990's that involved jacking an instrumented concrete jacking

pipe on five (5) field projects (Norris and Milligan, 1991). Their studies focused on joint deflection, interface friction, normal stresses, the effects of misalignment, and the effects of time delays on jacking forces. The instrumented pipe was fitted with sensors to measure joint deflection, contact stress transducers, pore pressure probes, and extensometers. Figure 2.11 shows the instrumented pipe used in the field tests of the five projects on which the instrumented pipe was jacked, two were in clay soils, one was in weathered mudstone, one was in dense silty sand, and one was in loose sand and gravel (Staheli, 2006).

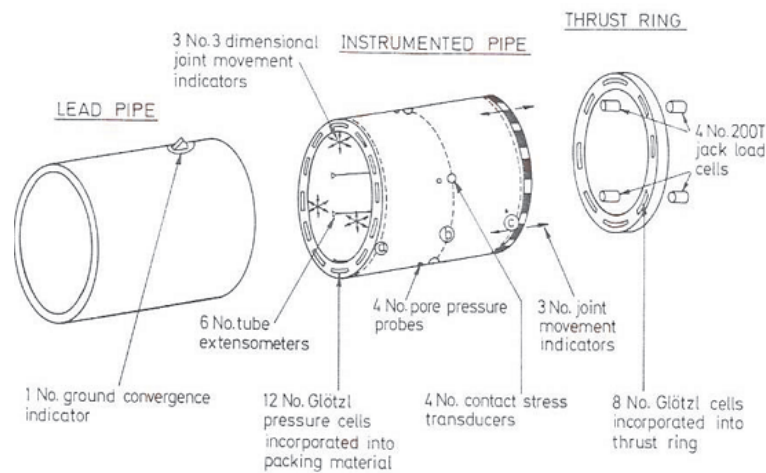


Figure 2.11. Instrumented pipe used in field tests at University of Oxford (Staheli, 2006).

For the case history with silty sand, a 59.5-inch Spun Concrete pipe was jacked 514 feet at a depth ranging from 23 to 32 feet deep. Norris found that load cells measuring normal stresses around the pipe during jacking generally measured values between 20 kPa and 40 kPa on the top and right sensors, as shown in Figure 2.12. Values on the bottom sensor measured higher peak values, although it should be noted that bentonite was pumped thorough a port on the bottom of the pipe inducing high pore pressures on the bottom of the pipe, as shown in Figure 2.12. Normal stress values generally measured below 50kPa for the left sensor. Values for the left sensor were greater than the right sensor due to steering corrections during the drive (Staheli, 2006).

For the field case history in loose sands and gravel, a 47-inch Spun Concrete pipe was jacked 1,260 feet at a depth ranging from 13 to 23 feet. During microtunneling, steering corrections were excessive and even caused pipe joints to break (Norris, 1992). Norris found that normal stresses around the pipe were fairly evenly distributed and averaged approximately 50 kPa at any given time. Norris concluded that bentonite injection into the annular space around the pipe caused the concrete pipe to float, resulting in equal normal stresses on the top and bottom of the pipeline. Figure 2.13 shows the total normal stress distribution around the pipeline during jacking between 91 and 131 meters of the drive at the main jack and interjack location (Staheli, 2006).

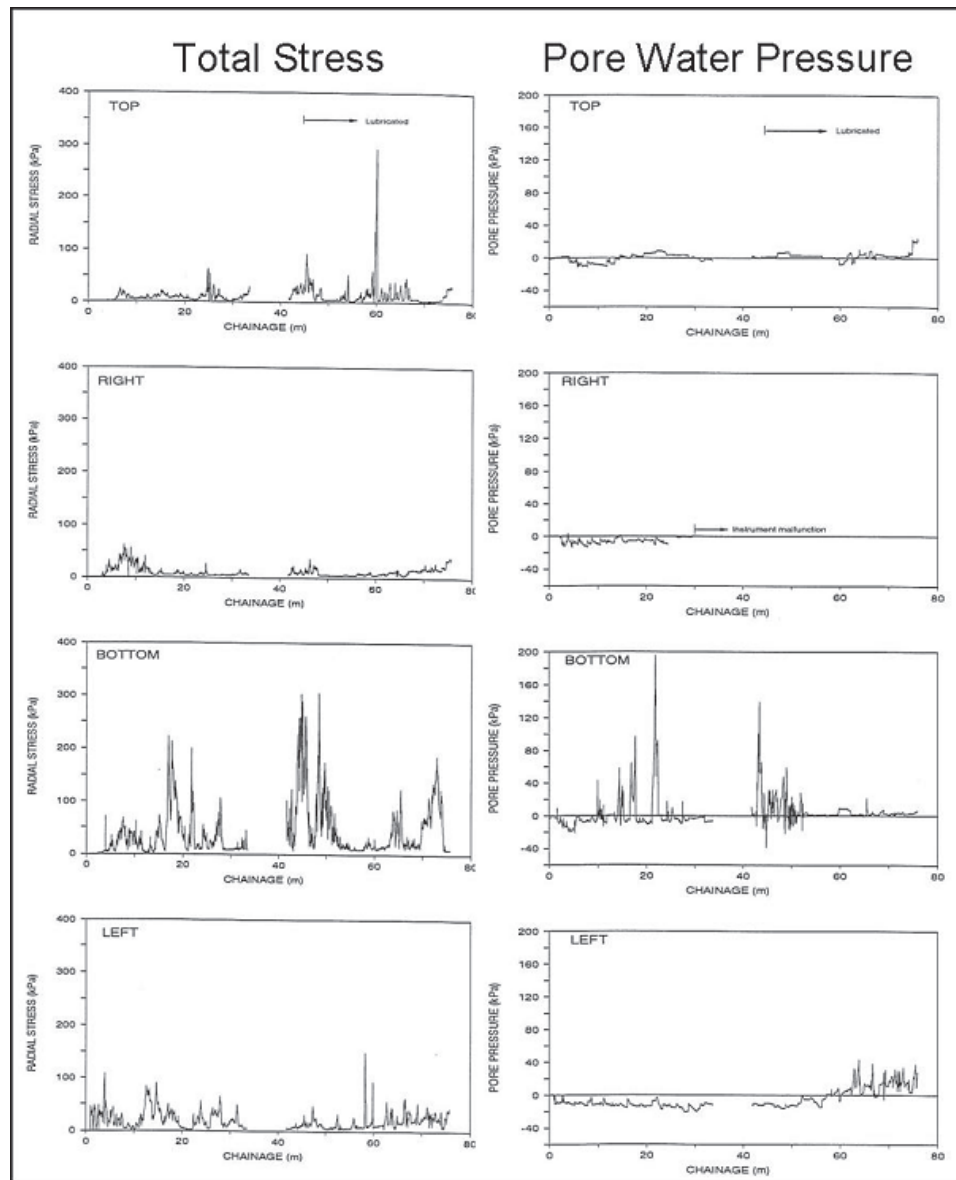


Figure 2.12. Total normal stress and pore water pressure measured by Norris around pipe while jacking in dense silty sand (Staheli, 2006).

2.6.2. Continuation of Field Studies by Marshall

Marshall continued the work by Norris by completing four additional field studies. These included one test in London clay, one in glacial till, one in soft peaty clay, and a fourth test in sandy gravel. The test in sandy gravel consisted on a 39.4-inch Spun Concrete pipe that was jacked 524 feet approximately 18 feet deep in dense silty sand. The measurements of effective radial stress in the main and intermediate jacking

stations are shown in Figure 2.13.

Marshall found that all pore pressure plots revealed excellent agreement with calculated pressures and indicated that hydrostatic pore pressure generally existed during jacking in the fine sand. Pore pressures increased above hydrostatic on the whole, corresponding with the pumping of lubrication. Pore pressure data, considered together with total radial stress data, provided very useful information on effective stress behavior (Marshall, 1998). Table 2.2 shows the sensor positions as well as peak and average measurements during the drive (Staheli, 2006).

Table 2.2. Typical values of mean interface shear stresses (Marshall, 1998).

Sensor	Rear Set of Sensors				Center Set of Sensors				Front Set of Sensor			
Position	σ' [kPa]		u[kPa]		σ' [kPa]		u[kPa]		σ' [kPa]		u[kPa]	
	Peak	Avg	Peak	Avg	Peak	Avg	Peak	Avg	Peak	Avg	Peak	Avg
Bottom	49	4	86	48	41	6	83	50	51	22	89	54
Left	209	49	74	41	171	45	100	46	65	59	74	43
top	122	44	47	38	67	16	70	38	-	-	-	-
Right	26	2	80	44	46	12	-	-	82	55	87	45
Average	101	25	72	43	81	20	84	45	66	45	83	47

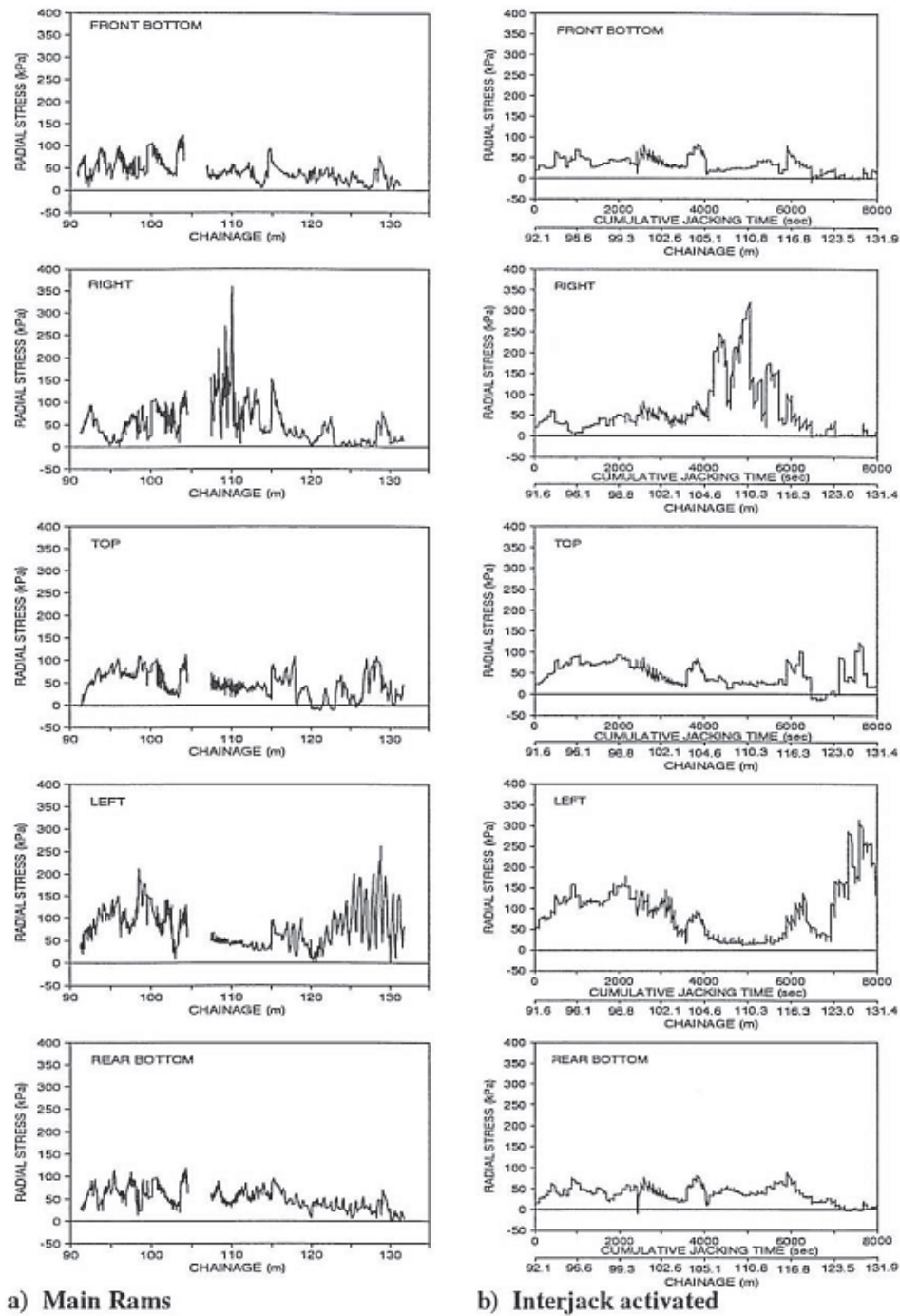


Figure 2.13. Total normal stress measured by Norris while jacking through loose sand with gravel (Staheli, 2006).

Based on these results, Marshall concluded that the pipeline was buoyant in the bentonite lubricant and that the pipe-soil contact was shown to be non-uniform along and around the instrumented pipe. He also concluded that the reason for the

non-uniform contact between the pipe and the soil was due to the arrangement of the lubrication injection sockets and the position of the instrumented pipe relative to the microtunneling machine (inserted as the second pipe behind the shield) (Marshall, 1998).

2.6.3. Numerical Modeling Studies by Zhou

As Marshall was conducting field studies on concrete jacking pipes, Zhou was conducting numerical analysis of concrete jacking pipes with finite elements. Zhou used the mesh generation program DATAIN (Zhou, 1998) for his finite element model and examined stresses within the pipe and stresses at the boundary between the pipe and the soil by modeling interface elements. Zhou modeled normal stresses acting on the pipe due to soils of various stiffness (Staheli, 2006).

In granular materials, Zhou found that the normal stresses between the pipe and the soil were very small. Zhou concluded that it was clear that the pipe separated or almost separated from the surrounding soil over most of the external surface of the pipeline. He further concluded that the effects of the distribution of the stresses from the surrounding soil on the stresses in the pipeline are small, since the magnitudes of the stresses on the interface are small. Zhou further concluded that the Australian model (Based on the work by (Auld, 1982) from the Terzaghi model) gives a somewhat good prediction for the maximum normal stress acting on the pipeline (Staheli, 2006).

2.6.4. Centrifuge Modeling of Stress Changes Above a Tunnel in Sand

At the University of Cambridge, (Jacobsez, Standing and Mair, 2004) investigated the changes in stresses in sand above a tunnel by developing a centrifuge model. They modeled the excavation at the tunnel face with the cavity contraction model after (Atkinson and Potts, 1977) and the radial equilibrium model. They found that during tunneling excavation, a considerable stress distribution occurred so that support pressures of relatively small magnitude were required, following the cavity contraction model. Figure 2.14 shows the failure mechanism proposed by Atkinson and Potts. In

the model developed by Atkinson and Potts, the failure envelope is defined by the angle 2ψ , where ψ is the dilation angle of the soil. The model illustrated that the cavity contraction model is a good representation of actual conditions in granular material as the stress path at the tunnel crown reached failure at the predicted failure envelope. Figure 2.15 shows the vertical stress measurements on the pipe during the test (Staheli, 2006).

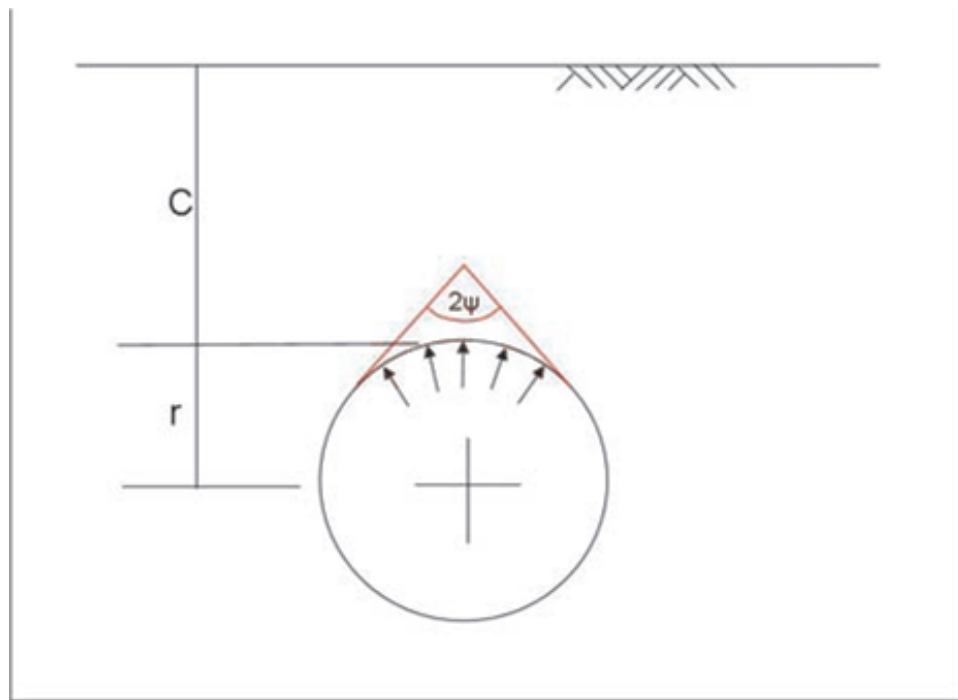


Figure 2.14. Total normal stress measured by Norris while jacking through loose sand with gravel (Staheli, 2006).

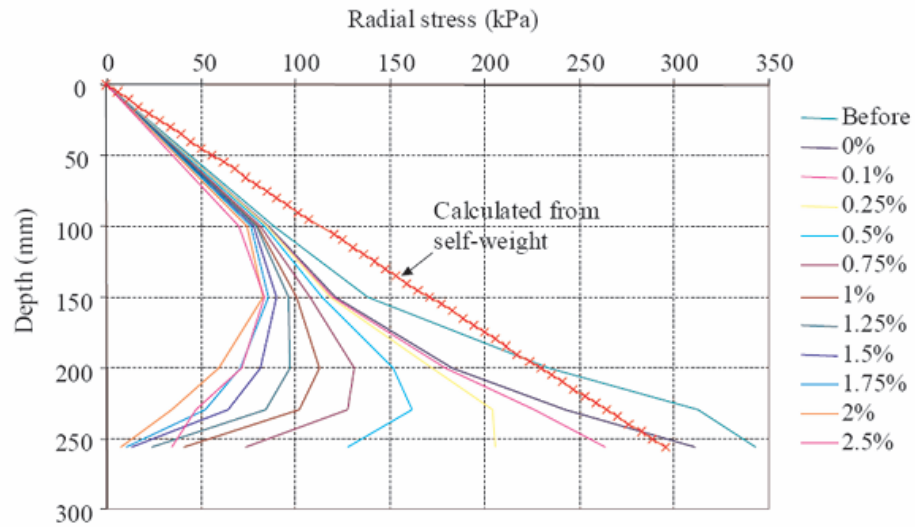


Figure 2.15. Predicted failure envelope based on cavity collapse model (Staheli, 2006).

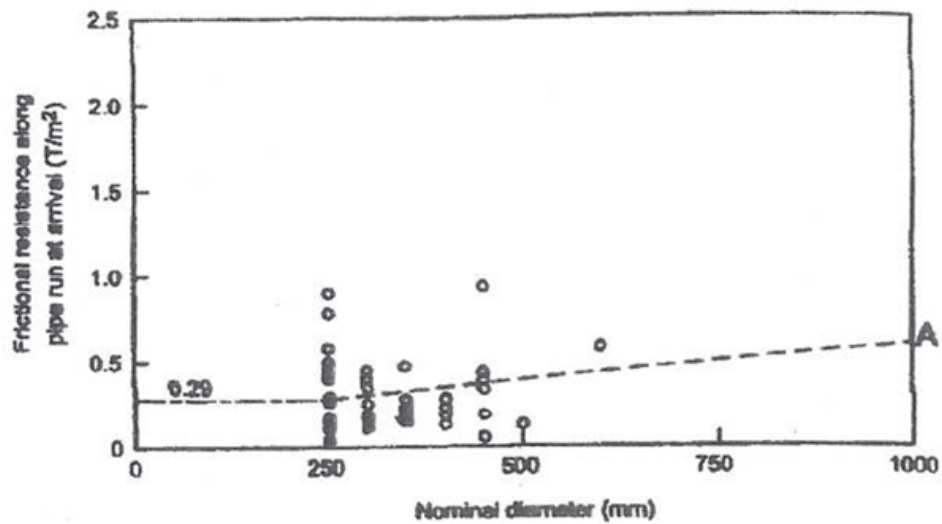


Figure 2.16. Vertical stress measurements during centrifuge tests (Staheli, 2006).

2.7. Research on Microtunneling and Jacking Forces

A number of practitioners and researchers have developed predictive models for the estimation of frictional jacking forces in the late 1970's and early 1980's. These methods are summarized in (Stein, 2005). A summary of the predictive models is presented in Table 2.3. Details of selected models are presented herein (Staheli, 2006).

Table 2.3. Existing predictive models for predicting the frictional component of jacking forces (Modified and Adapted from (Stein, 2005)) (Staheli, 2006).

Author	Predictive Model Frictional Component of Jacking Force	Symbols, Notes, Etc.
Walendky/Möncke (1970)	$\gamma h \sqrt{\frac{K_a^2 + 1}{2}} \tan \delta$	$\delta = \varphi \epsilon / 2$ K_o = Coefficient of lateral earth pressure at rest H = Cover depth
Helm (1964)	Circular Cross Section: $\mu \gamma h \frac{K_a + 1}{2}$ Rectangular Cross Section: $\mu \gamma h \frac{b_a + K_o d_a}{b_a + d_a}$	K_a = Active earth pressure coefficient b_a = External width of the microtunneling shield or machine d_a = External height of diameter of microtunneling shield or machine
Szentandrási (1981)	$\mu (H_w + \frac{d_a}{2} \frac{K_{Sc} h + K_{K1} + K_{K2} + K_{S2}}{4} + \frac{W_s - F_A}{4 d_a})$	H_w = Effective cover depth W_s = Dead weight of pipe F_A = Buoyancy
Scherle (1977)		For a very dense compacted sand.
Soloma (1979)	Circular Cross Section: $\gamma (h + \frac{d_a}{2}) \sqrt{K_m \tan \delta}$ Rectangular Cross Section: $\mu \gamma h (b_a + K_m d_a)$ $1 + \frac{d_a}{2h}$ $\frac{d_a}{b_a + d_a}$	K_m = Effective earth pressure Coefficient
Weber (1981)	Circular Cross Section: $\mu \sqrt{p_o p_h}$ Rectangular Cross Section: $\mu ((\kappa b_a) + (K_o d_a) + K_o \frac{d_a^2 / 2h}{b_a + d_a})$	Slurry Boring Method $\mu = 0.46$ P_o = Vertical earth pressure p_h = Horizontal earth pressure
Weber (1981)	$\mu E_s \frac{\Delta d_a}{d_a}$ With Stiff ness Modulus from Ohde: $E_s = \nu (\frac{\sigma_1}{\sigma_3})^w$	Auger Boring Method with steel Pipes (318.508 and 711 mm diameter): νw = Stiffness coefficients Δd_a = Deformation dimension of the pipe string
Iseki (as Summarized in Stein 2005)	$\mu (q + W_s) + C$	q = Loading vertical to pipe axis $[kN/m^2]$ $\mu = \tan \frac{\phi}{2}$
Ebert (1990)	$0.5 \mu (\alpha \gamma h [\gamma (h + \frac{d_a}{2}) + F_a] [K_e + W_s] \frac{1.0}{L R})$	γ = Soil density a = Active load coefficient P_o = Surface loads K_e = Earth pressure coefficient at rest L_R = Pipe length
Paul (as summarized in Stein 2005)	$\mu [2 (\gamma \frac{b}{d_a} + P_{VR}) (1 + K'_a) + W_s]$	
Chapman (1990)	$a + 3.8 D_a$ Details of Method found in Section 2.3.4	Based on Statical Evaluation of 198 Slurry Microtunneling Projects $a = 1.53$ for clay, $a = 2.43$ for sand $a = 3.43$ for sand / gravel
Bennett (1998)	$F_r = C_a \gamma' d_p \tan (C_f \phi_r) A_P L$ Details of Method and Values for Friction Reduction Factor, C_f and Arching Reduction Factor C_a found in Section 2.4.4.	γ' = Effective soil unit weight. d_p = Pipe diameter; ϕ_r = Residual soil friction angle A_P = Pipe Circumference L = Length of tunnel.
(2000)	$C' =$ Adhesion of Pipe and Earth $(8 kN/m^2$ for $N < 10$ and $5 kN/m^2$ for $N > 10$) Details of Method found in Section 2.4.5	B_c = Diameter of the pipe Q = Normal force W = Pipe weight

2.7.1. Predictive Model Developed by Scherle

(Scherle, 1977) established interface friction coefficients between concrete and asbestos cement pipes and various soil types based on data collected at field sites. He grouped the interface friction coefficients into three categories representing the state of motion of the pipeline: static friction, sliding friction, and fluid friction. The third category, fluid friction, represented the state of motion when bentonite was used as a supporting and lubricating fluid around the pipeline, and Scherle gave a range of friction coefficients for this state depending on the liquid limit of the bentonite suspension. Scherle asserted that the frictional component of the jacking force was a function of the interface friction coefficient, multiplied by the unit weight of the soil, multiplied by the depth of cover to the spring line of the pipe, times a factor that was based on the state of stress in the soil. Values for the interface friction coefficient as established by Scherle are shown in Table 2.4 (Staheli, 2006).

Table 2.4. Interface friction coefficients according to scherle.

Pipe Material and Soil and Interface	Static Friction Interface Friction Coefficient. μ [-]	Sliding Friction Interface Friction Coefficient. μ [-]	Fluid Friction Interface Friction Coefficient. μ [-]
Concrete Pipe on Gravel or Sand	0.5 to 0.6	0.3 to 0.4	0.1 to 0.3
Concrete Pipe on Clay	0.3 to 0.4	0.2 to 0.3	0.1 to 0.3
Asbestos Cement Pipe on Gravel or Sand	0.3 to 0.4	0.2 to 0.3	0.1 to 0.3
Asbestos Cement Pipe on Clay	0.2 to 0.3	0.1 to 0.2	0.1 to 0.3

2.7.2. Predictive Model Developed by Weber

Weber conducted investigation of jacking forces of microtunneling pipes and found no relationship between depth of burial and jacking forces. Weber established a calculation model that provided an upper bound for the interface friction value. Weber used an interface friction value of 0.46, corresponding to an interface friction angle of 24.7 degrees as summarized in (Stein et. al, 1989). Weber uses this value regardless of pipe material or soil type in order to provide a conservative value for predicted jacking forces. Weber also published a table of jacking stresses related to skin friction that were categorized by soil type . These values for jacking stresses were calculated

post-construction from several microtunneling projects as shown in Table 2.5 (Staheli, 2006).

Table 2.5. Frictional jacking stresses for various soil types as reported by (Weberz, 1981).

Soil Type	Jacking Stress due to Skin Friction [kN/m^2]	Jacking Stress due to Skin Friction [$tons/ft^2$]
Gravel, sand	8.4 ± 2	0.087 ± 0.02
Loamy sand	9.3 ± 1	0.097 ± 0.01
Loam	7.3 ± 1	0.076 ± 0.01
Loam, stones	5.7 ± 4	0.060 ± 0.04

2.7.3. Investigations of Pipe-Soil Interactions with an Instrumented Jacking Pipe Conducted by Milligan and Norris

Milligan and Norris determined the soil-pipe interface friction angle at each of the sites by measuring the normal loads on the pipes from the contact stress transducers. They attempted to isolate sections of the bore where lubrication was applied and compared those to sections where lubrication was not applied or poorly applied.

Milligan and Norris studied the pipe-soil interface for cohesive materials and developed a hypothesis regarding the asperities in the concrete pipe at the soil-pipe interface and the contact area with the cohesive soil. They found that much higher values of interface shear strength were measured at low normal stress levels. They hypothesized that the roughness of the concrete, and, more particularly, the soil surface caused the surface area over which contact was made to be small, and that as the normal stress increased, the contact area between the pipe and the soil would increase. They assumed this trend would continue until the normal stress caused intimate contact at the surface, similar to the findings of (Dove and Frost, 1999). Milligan and Norris conducted a laboratory test with a standard direct shear Cassagrande apparatus to model shearing conditions in the field, replacing the upper portion of the shearing device with concrete and filling the lower portion with London Clay. However, difficulties were

encountered in specimen preparation due to the fissured nature of the London Clay and modeling clay was used in its place. Testing with the shearing apparatus was found to be somewhat inconclusive (Staheli, 2006).

2.7.4. Statistical Evaluation of Jacking Force Data by the ISTT Working Group No. 3 and Chapman Leading to Chapman's Predictive Model

The International Society for Trenchless Technology formed a working group in 1992 entitled Working Group No.3, which conducts technical research on microtunneling. The group focused their efforts on the analysis of jacking forces in order to find a formula to calculate jacking forces by type. (ISTT WG No.3 1994) At the time the research statement was developed, the type was undetermined, and a statistical analysis of data from 398 projects was undertaken to find similarities in order to group the data by a common variable. The variables included machine type, soil type, soil removal system type (auger vs. slurry), pipe diameter, earth cover, jacking distance, among others. However, because all of the projects were analyzed together, and none of the records were analyzed for construction problems or case history anomalies, the range of data scatter was extremely large and few conclusions could be drawn from the study (Staheli, 2006).

In addition, the parameters studied, such as the largest force at the completion of the drive, were not necessarily indicative of the jacking behavior throughout the drive. The committee recommended that the data be further analyzed in smaller groups separated by categories such as geotechnical conditions along the tunnel, pipe diameter, etc. However, a follow-up report was not published.

In 1999, (Chapman and Ichioka, 1999) revisited the work done by the ISTT working group and re-evaluated the data. They separated the microtunneling case histories into three categories by soil type: clay, sand, and sand with gravel. For each category they plotted pipe diameter versus frictional resistance along the pipe (in tons per square meter) resulting in the graphs shown in Figure 2.17 through 2.18.

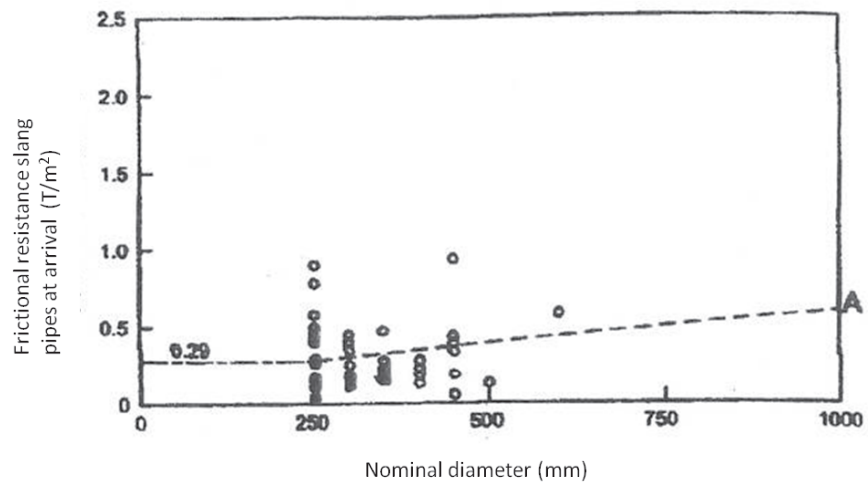


Figure 2.17. Pipe diameter vs. frictional resistance in clay (Staheli, 2006).

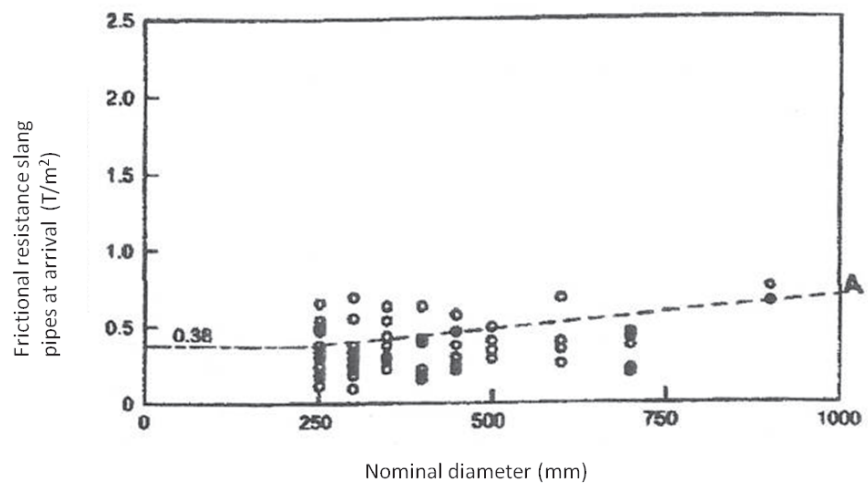


Figure 2.18. Pipe diameter vs. frictional resistance in sand (Staheli, 2006).

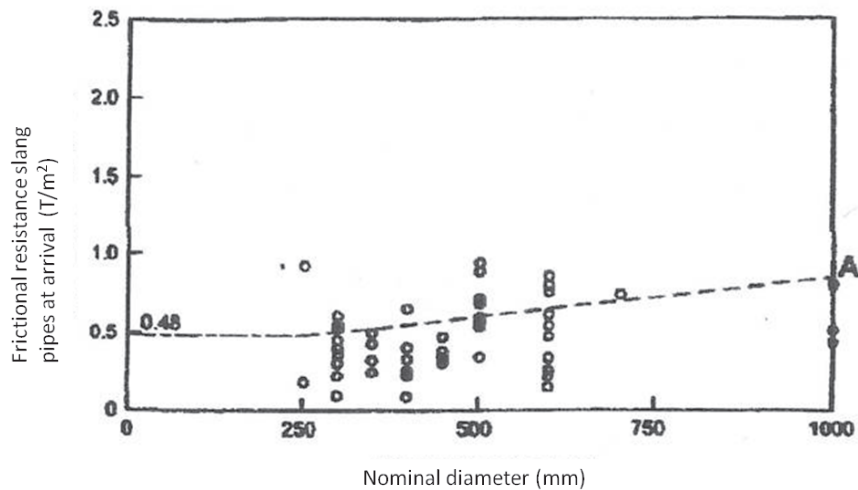


Figure 2.19. Pipe diameter vs. frictional resistance in sand and gravel (Staheli, 2006).

Chapman and Ichioka plotted all of the data contained in the three charts shown in Figure 2.17 through Figure 2.19 on one chart and performed a linear regression on the data. For all of the soil types plotted on a single chart, they found the slope of the regression line to be 0.38 as shown in Figure 2.20, labeled A.

Linear regression lines were also carried out on the data when separated out by soil type in Figure 2.17 through Figure 2.19 and the slope of the linear regression was found to be similar to 0.38 but with a different intercept.

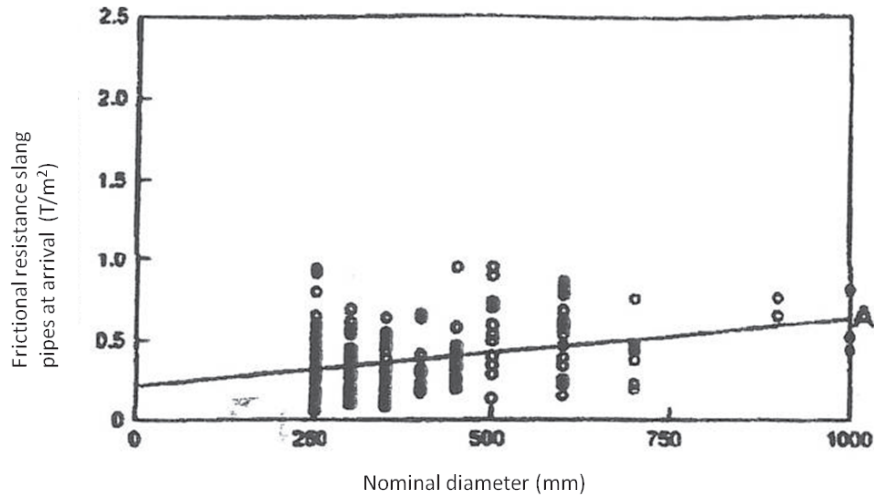


Figure 2.20. Pipe diameter vs. frictional resistance for all soil types (Staheli, 2006).

Chapman and Ichioka then developed an equation for frictional resistance, P , which they related to the diameter of the pipe by the following equation:

$$P = a + 0.38D \quad (2.11)$$

- where P = frictional resistance [$tons/m^2$]
- a = intercept value for each soil type (taken from Figure 2.17 through 2.19)
- D = pipe diameter

The intercept value, a , was found by using the smallest nominal diameter, 250 mm (outer diameter 360 mm). Chapman listed values for a as shown in Table 2.6 (Staheli, 2006).

Table 2.6. Frictional jacking stresses for various soil types as reported by (Weber, 1981).

Soil Type	Intercept Value a for Chapman, Ichioka Predictive model
Clay Soils	0.153
Sand	0.243
Sand and Gravel	0.343

2.7.5. Controlled Field Tests with Microtunneling Leading to Bennett's Predictive Model

A series of field tests were conducted by the US Army Corps of Engineers in which instrumented test beds were constructed of four soil types, and through which two microtunnels were constructed. Figure 2.21 shows test bed through which the microtunnels were driven.

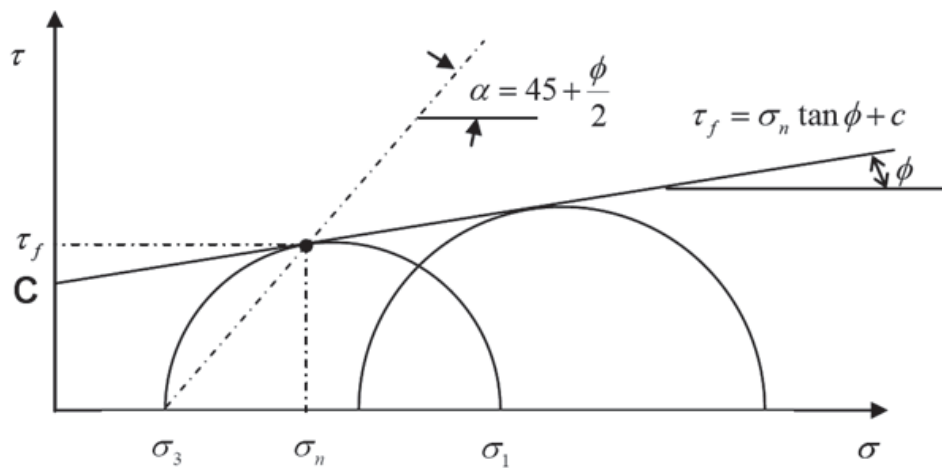


Figure 2.21. Microtunneling test bed for controlled field test (Staheli, 2006).

The drive data from the microtunnels were collected and analyzed by (Bennett, 1998), who then developed a model for predicting jacking forces in both cohesive and granular soils. Bennett also analyzed the effects of steering corrections, delays, face pressures, loss of face stability, over-cut and lubrication. In addition to the field testing performed at the Waterways Experiment Station, Bennett also analyzed case history

data from five (5) projects that included 39 microtunnel drives. Of these 39 microtunnels, 37 of the tunnels ranged between 34 and 36 inches in diameter. The remaining two tunnels had 62.9 and 82.5-inch outer diameters (Staheli, 2006).

Bennett's predictive model is based on the concept that the total jacking force is a function of the surface area of the pipe multiplied by a normal force and a friction coefficient. The normal force, however, is not based on depth, but rather on the diameter of the pipe and the effective unit weight of the soil. Bennett then introduces an arching reduction factor that he terms C_a that is multiplied by the effective unit weight and the diameter (Staheli, 2006).

Bennett's friction factor is based on the friction angle of the soil. Here Bennett introduces the friction reduction factor that he calls C_f that is multiplied by the residual friction angle of the soil. Bennett then establishes upper bound, best-fit, and lower bound values for the arching and friction reduction factors for use in his proposed model. Further, Bennett separates his recommendations for the arching and friction reduction factors based on whether the microtunnel is in sand or clay (Staheli, 2006).

One final distinction that makes Bennett's model unique is that he separates the drive into two individual segments. The first segment of the microtunnel drive Bennett terms the initial dewatered, non-lubricated interval followed by the lubricated non-dewatered interval. Each of these segments has different recommended values for the arching and friction reduction values. Table 2.7 provides the coefficients C_a and C_f for Bennett's predictive model by,

$$F_r = C_a \gamma' d_p \tan(C_f \phi_r) A_p L \quad (2.12)$$

Where:

- F_r = Frictional Jacking Force;
- γ' = Effective soil unit weight,
- d_p = Pipe Diameter;

φ_r = Residual Soil Friction Angle,

A_p = Pipe Circumference, and

L = Length of Tunnel.

Table 2.7. Arching and friction reduction factors from bennett (Staheli, 2006).

	Initial Dewatered, Non-Lubricated 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

2.7.6. Osumi's Predictive Model Using Jacking Force Reduction Factors

Osumi of the Japan Microtunneling Association studied 49 pipe jacking projects and developed a method for calculating the frictional component of jacking forces. Osumi based the interface friction coefficient between the pipe and the soil on the internal friction angle of the soil. He assumed that regardless of pipe material, the interface friction coefficient, μ' , was equal to the tangent of half of the interface friction angle ($\tan(\varphi/2)$). Osumi bases the normal force on the depth of cover and the weight

of the pipe. His jacking force equation also has a component for adhesion between the pipe and the earth which accounts for cohesive soils (Staheli, 2006).

Osumi's equation for the frictional component of the jacking force is as follows:

$$f_o = \beta(\pi B_c q + w)\mu' + \pi B_c C' \quad (2.13)$$

Where: f_o = frictional component of jacking force

β = Jacking Force Reduction Factor

B_c = Outer Diameter of the Pipe

q = Normal Force

W = Pipe Weight

C' = Adhesion of Pipe and Earth

(8 kN/m² for $N < 10$ and 5kN/m² for $n > 10$)

Using the jacking force data from the 49 tunnels, Osumi performed a statistical analysis to determine the Jacking Force Reduction Factor, β . This factor was developed for four types of soil: cohesive, sandy, gravel, and solid. Osumi does not provide any description of the soil type that is used to form the group of soils termed solid. Table 2.8 provides recommended values for the Jacking Force Reduction Factor as provided by Osumi.

Table 2.8. Jacking force reduction factors (Staheli, 2006).

Soil Category	Jacking Force Reduction Factor, $\beta[-]$
Cohesive Soil	0.35
Sandy Soil	0.45
Gravel	0.60
Solid Soil	0.35

2.7.7. Staheli's Predictive Model

A total of thirteen case histories were chosen with unlubricated segments that were suitable for comparison with laboratory data. These case histories included Hobas, Polycrrete, Permalok Steel, Wet Cast Concrete, and Packerhead Concrete ranging in diameter from 25, 8 to 87, 5 inches. Using the values for the interface friction coefficient based on pipe material and residual soil friction angle of the soil at the project site, the estimated normal stress was calculated for each project. The results of this calculation are showed that the estimated normal stress was relatively low, ranging from 118 psf to 399 psf. In addition, the estimated normal stresses indicated that the normal stresses were independent of depth, as suggested by (Terzaghi, 1943) as the depths of the projects ranged from 8 to 85 feet with no correlation between depth and normal stress (Marshall, 1998).

Evaluation of the normal stresses indicated that predictive models previously proposed by (Auld, 1982), ATV A161 (Stein, 1989) (Scherle, 1977), and Körner dramatically over-predict normal stresses acting on the pipeline. Development of a model for calculating the normal stress above the pipeline focused on the redistribution of stresses around the pipeline as the microtunneling machine excavates through the soil. It is important to recognize that the microtunneling process involves excavating with a pressurized slurry and is cutting a hole with a cutter wheel that is slightly larger in diameter than the machine, typically on the order of three quarters to one inch larger. As a result, granular soil above the pipeline is allowed to move toward the machine and the pipeline, allowing redistribution of the stresses around the machine (Staheli, 2006).

In large diameter tunneling with a slurry shield, the cavity contraction model has been used to model the behavior of the soil at the face and above the machine during excavation to estimate surface settlements above the tunnel. The cavity collapse model is also applicable to microtunneling because excavation takes place with a slurry shield. All of the microtunneling case histories were constructed below the water table. In every case, the jacking records reveal that the pressure within the slurry chamber was

maintained below that of the groundwater pressure, indicating that when excavation would begin there was, in fact, an area of lower pressure toward the machine, lending credence to the use of a cavity collapse model. In addition, the low pressure within the tunnel shield would cause granular soils in the immediate vicinity of the machine to drain (Staheli, 2006).

Centrifuge modeling performed by (Jacobsz et. al, 2004) on granular soils above a tunnel, as outlined above, found that the failure envelope agreed well with that defined by (Atkinson and Potts, 1977) based on the cavity collapse model. Atkinson and Potts related this area to the dilation angle of the soil, as shown in Figure 2.14. However, due to the fact that the soil is collapsing onto the pipeline, it appeared that the soil that would likely remain intact above the pipeline would be above the shear plane of failure (Staheli, 2006).

According to the Mohr-Coulomb failure criteria, shear strength on the failure plane (as seen in Figure 2.22) is defined as

$$\tau_f = \sigma_n \tan \phi + c \quad (2.14)$$

where σ_n = normal stress and ϕ, c = shear strength parameters of soil (where ϕ and c in the above equations are drained strengths for long-term analysis and undrained ($\phi = 0$ and $c = S_u$) for short-term analysis of cohesive materials

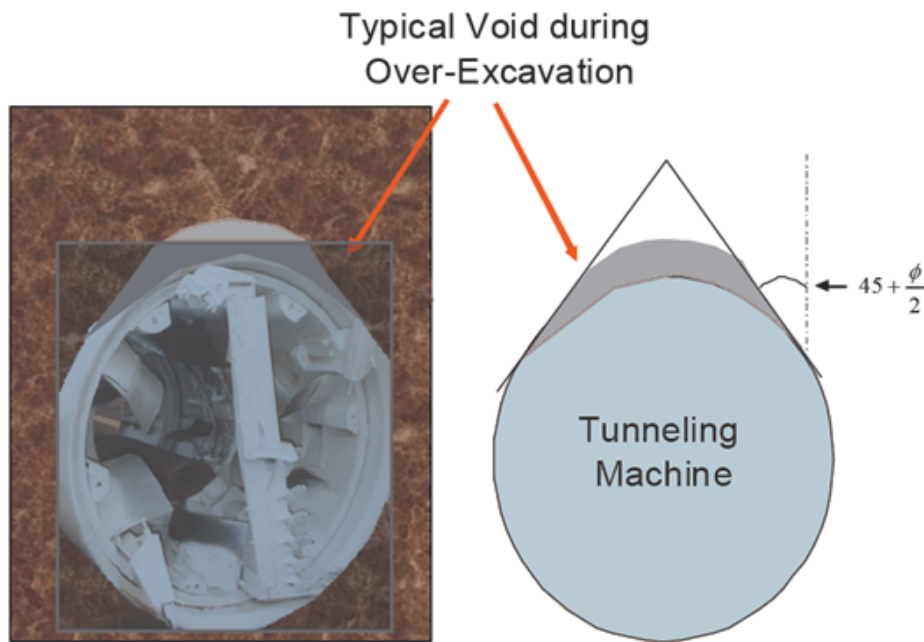


Figure 2.22. Mohr-coulomb failure criterion (Staheli, 2006).

The failure plane is inclined at $45 + \varphi/2$ degrees from the plane of the major principal stress. For limit-equilibrium analyses to be valid, the assumed slip surface must be inclined at this angle relative to the principal stresses. Discontinuities in the soil, surcharges, and wall friction all cause variation in the principal stress directions and induce curvature in the slip surfaces (Staheli, 2006).

The failure planes, as presented in the Mohr-Coulomb failure criterion, inclined at an angle $45 + \varphi/2$ present an interesting comparison when one considers the shape of a localized over-excavation over the crown of a tunneling machine. When over-excavation occurs at the face of a tunneling or pipe jacking operation in granular soils, the shape of the void that develops over the tunnel crown manifests as shown in Figure 2.23. The shape of the void presents with the failure planes inclined at a slope normal to the principal stresses and variations in the materials induce the curvature in the slip surfaces, as presented in the Mohr-Coulomb theory

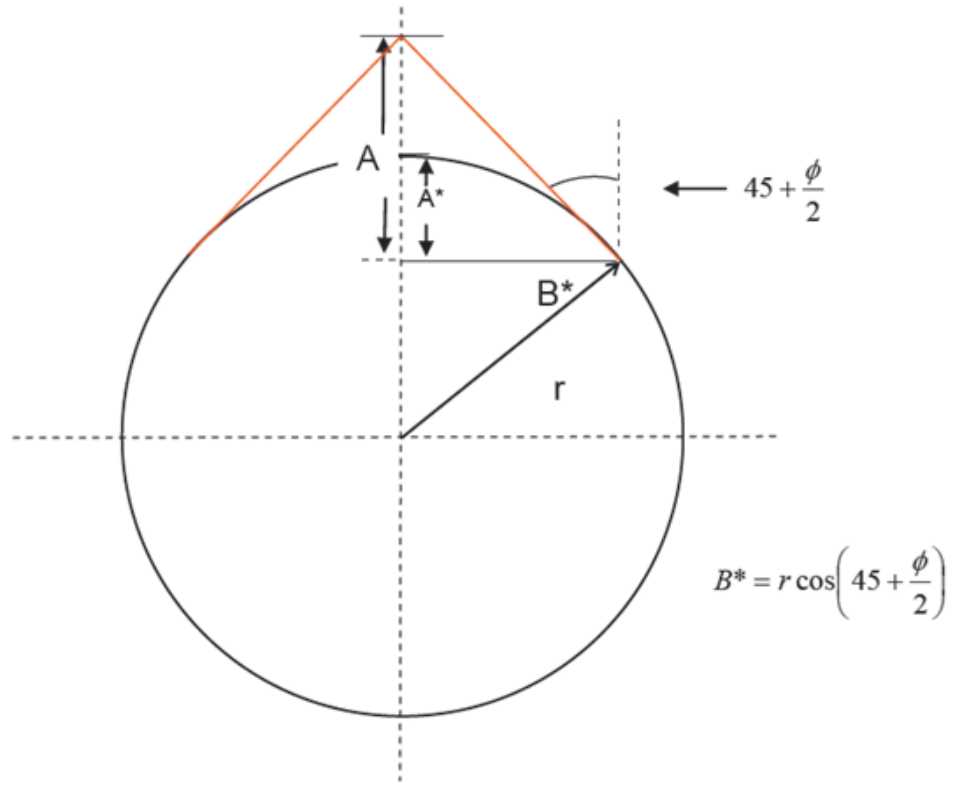


Figure 2.23. Typical void development over tunneling machine with over-excavation (Staheli, 2006).

when considering this in conjunction with Terzaghi's Arching Theory, there is reason to reconsider some of the previous interpretations on how Terzaghi's soil arching experiments, which were conducted on a flat trap door, have been applied to a rounded pipe surface (by Auld, Scherle, ATV A 161, etc).

The creation of the overcut also induces a state analogous to the trap door simulations conducted by Terzaghi. When the gage cutters on the tunneling machine remove the soil at the crown, for example, it induces a state of stress analogous to when the trap door was displaced vertically, inducing arching in the overlying soils. Terzaghi found that for a cohesionless soil, the vertical stress was independent of depth, and presented the following equation,

$$\sigma_\nu = \sigma_{\nu\infty} = \frac{\gamma B}{K \tan \phi} \quad (2.15)$$

Terzaghi found $K=1$ for soils above the yielding trap door, analogous to the soils in the zone above the pipe where the material is moving into the space cleared by the gage cutters; into the overcut.

Based on the observations of the shear planes of failure above the pipeline during over excavations and Mohr-Coulomb failure criteria, Figure 2.24 is proposed as the interpretation of the area over the pipeline over which the vertical loading is developed. From Figure 2.24, the factor B^* is developed and replaces B in Terzaghi's equation 2.15.

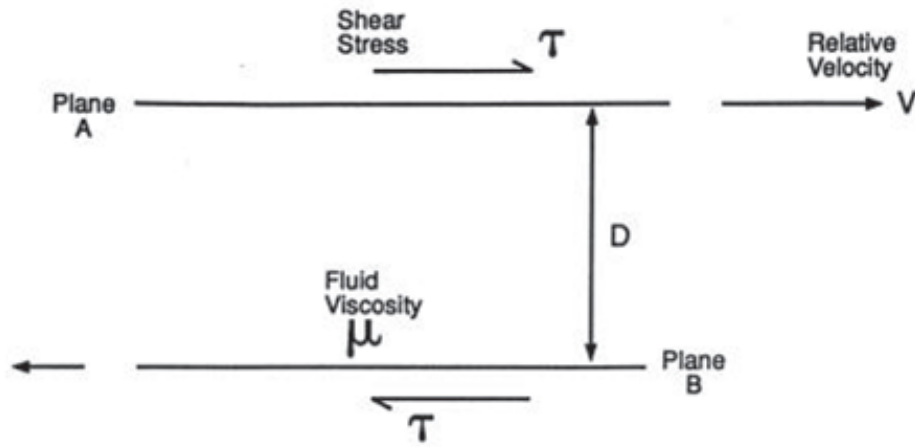


Figure 2.24. B^* Factor for use in vertical stress calculations (Staheli, 2006).

In Terzaghi's vertical stress calculation, the factor B , representing the width of the trap door, is replaced by B^* which is defined by

$$B^* = r \cos\left(45 + \frac{\phi}{2}\right) \quad (2.16)$$

where r = pipe radius and ϕ = soil internal friction angle.

Therefore, substituting B^* into Terzaghi's equation for vertical stress results in the following equation:

$$\sigma_\nu = \sigma_{\nu\infty} = \frac{\gamma r \cos\left(45 + \frac{\phi}{2}\right)}{\tan\phi} \quad (2.17)$$

Field investigations by (Milligan and Norris, 1992), and (Marshall, 1998), included jacking an instrumented pipe equipped with sensors to measure normal stresses around the circumference of the pipe on nine projects, as detailed before results of the investigations 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 material on one side of the pipe that is harder than on the other. They also showed that using the self-weight of the pipe on concrete pipe projects typically grossly over-estimates normal stresses. (Zhou, 1998) performed numerical modeling studies on interface stresses between pipes and soils during jacking and found the jacking stresses to be evenly dispersed around the jacking pipe, except at areas of stress concentration at the joint where misalignment occurred due to steering corrections (Staheli, 2006).

With the findings of the distributions of normal stresses based on Milligan, Norris, Marshall, and Zhou, and the proposed model for predicting normal stress as presented in equation 2.18, the following predictive model for calculating the frictional component of jacking forces is proposed:

$$JF_{frict} = \mu_{int} \frac{\gamma r \cos(45 + \frac{\phi_r}{2})}{\tan \phi_r} \pi d l \quad (2.18)$$

Where JF_{frict} = Frictional Component of Jacking Force [tons force]

μ_{int} = Pipe-Soil Residual Interface Friction Coefficient [unit-less]

γ = Total Unit Weight of the Soil [$tons/f^3$]

ϕ_r = Residual Friction Angle of the Soil [degrees]

d = Pipe Diameter [feet]

r = Pipe Radius [feet]

l = Length of the Pipe [feet]

2.8. Rheology and Application of Slurries

Rheology is the study of the flow properties of materials. As the use of soil conditioning and lubricating agents is greatly affected by the flow properties of the

materials and their mixtures with soil, some of the basic terms and concepts will be summarized here in.

Viscosity is the relation between shear stress and shear strain rate in a liquid. Thus if two planes in a liquid are separated by a distance D and are moving parallel to each other at a relative velocity V (see Figure 2.25), the shear stress in the liquid between the plates is given by $\tau = \mu V/D$, where μ is the coefficient of viscosity. Units of viscosity are Ns/m^2 or Pa.s, but are normally expressed in terms of centipoises (1/100th of a Poise, named after a French scientist); 1.0 cp is equal to $10^{-3} Ns/m^2$. The viscosity of water at 20°C is $10^{-3} Ns/m^2$.

Newtonian fluids are those for which the viscosity is independent of the rate of shear, and include water and oil; the shear stress increases linearly with rate of shear. Non-Newtonian behavior may be described by models such as Bingham plastic, pseudo plastic or dilatants Figure 2.26. The first shows an initial yield stress below which no shear occurs, then a linear increase of shear stress with strain rate (the plastic viscosity) (Milligan, 2000).

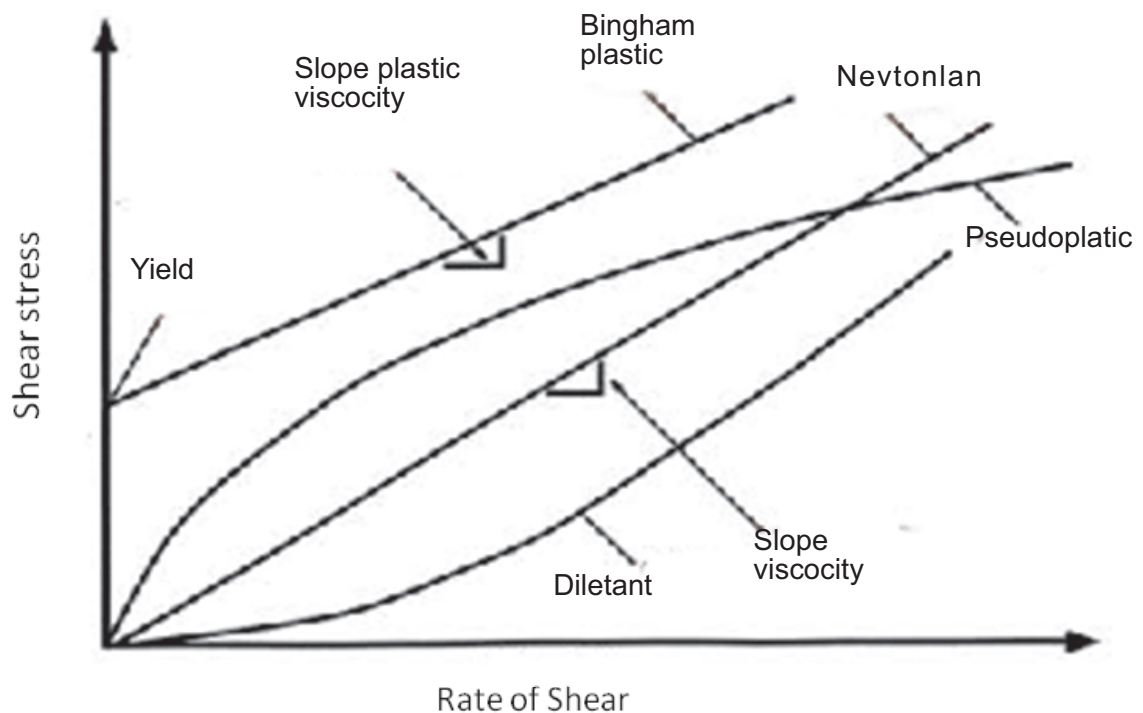


Figure 2.25. Definition of viscosity (Milligan, 2000).

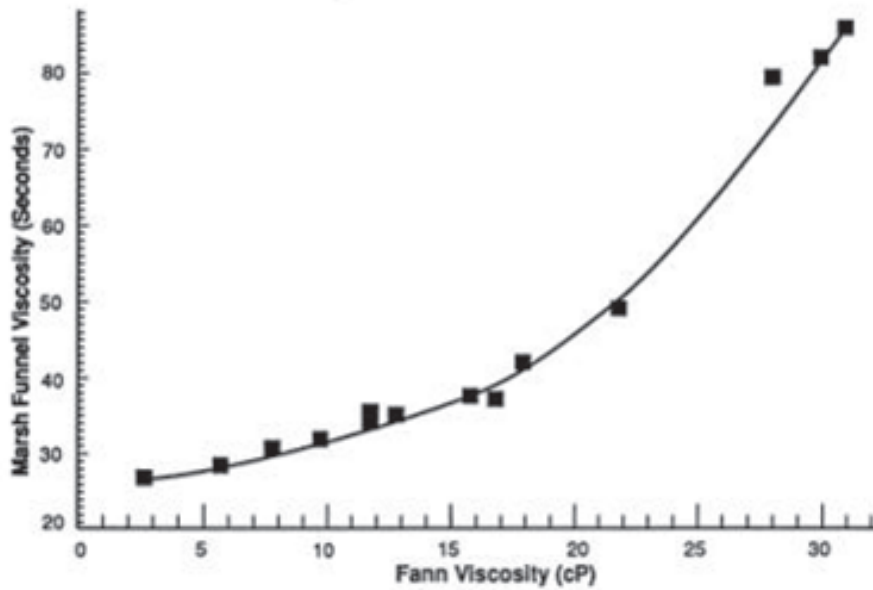


Figure 2.26. Models of viscous behavior (Milligan, 2000).

Pseudo plastic or dilatant behavior may be represented by a power law curve, so that it appears as a straight line on a logarithmic scale of shear rate:

$$\tau = K(V/D)^n \quad (2.19)$$

where K and n are constants with

$n < 1$ for a pseudo plastic fluid

$n > 1$ for a dilatant fluid and

$n = 1$ for a Newtonian fluid

Viscosity is affected by temperature, most liquids becoming less viscous at higher temperatures. In some slurries the viscosity decreases at high shear rates, and this is referred to as shear thinning. Time effects also occur in many fluids; thixotropy is the most important example. Many slurries used in tunnelling are quite complex in their rheology, but may be treated for practical purposes as Bingham fluids with thixotropic properties.

Viscosity may be measured by a number of different types of apparatus, known

as viscometers. Assuming the fluid to act as a Bingham plastic fluid, a minimum of two readings at different shear rates are needed to define the yield point and the plastic viscosity (Milligan, 2000).

A simple device often used on site is the Marsh cone; in this the time taken for a given volume of the fluid (1000cc or 1.0 US quart) to flow out of a standard conical funnel provides a measure of the viscosity. An approximate relationship between the Marsh funnel reading and the viscosity measured by a viscometer is given for bentonite slurries in Figure 2.27.

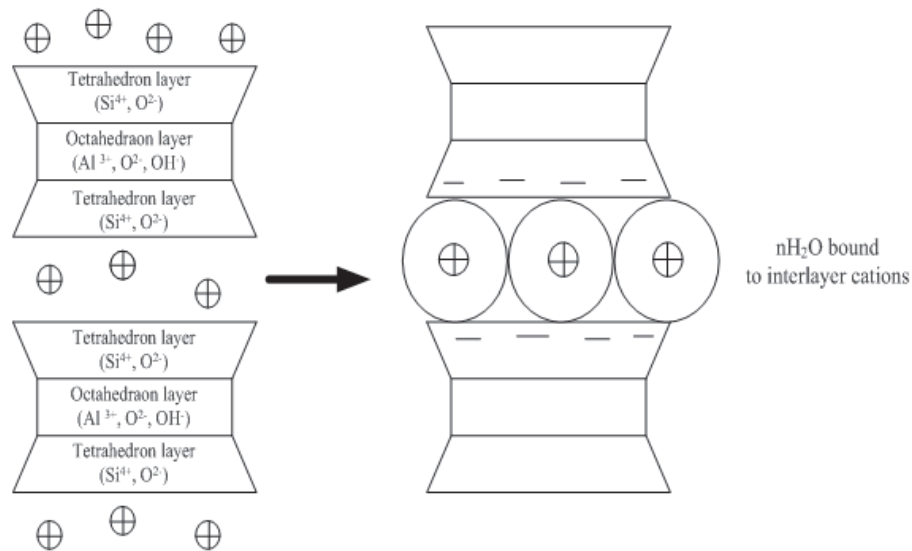


Figure 2.27. Relation between results from Marsh funnel and Fann viscometer (Milligan, 2000).

2.8.1. Bentonite

The name bentonite is popularly used for a range of natural clay minerals, principally potassium, calcium and sodium montmorillonites. The term smectite is also used for the group of minerals which includes montmorillonites. Because of the chemistry and microstructure of the clay particles, they have a strong ability to absorb water and are able to hold up to ten times their dry volume by absorption of water. Montmorillonite consists of very thin flat crystalline sheets of clay minerals which are negatively charged and are held together in stacks by positively charged sodium or calcium ions

in a layer of adsorbed water (Milligan, 2000).

In particular the particles of sodium montmorillonite are extremely small and thin, being typically of the order of $1.0\ \mu\text{m}$ or less in length and $0.001\ \mu\text{m}$ thick. The ability to absorb water comes from the relatively low bonding energy of the sheets, which allows water molecules to be adsorbed onto the internal and external sheet surfaces. Calcium ions provide a stronger bond than sodium, so that calcium montmorillonite swells less readily than sodium montmorillonite. Potassium ions provide much stronger bonding between clay sheets as the potassium ion is of exactly the right diameter to fit into the clay structure with negligible gap between the clay sheets (Milligan, 2000).

A similar material to montmorillonite but with potassium bonding is the non-swelling clay mineral known as illite. The substitution of sodium by calcium or potassium ions in montmorillonite greatly reduces the ability of the clay structure to hold water (Milligan, 2000).

Bentonite, a very highly plastic, swelling clay material composed of smectite group of minerals, is the sealing material in geosynthetic clay liners (GCL). Bentonite is a naturally occurring clay with high swelling potential, large surface area, high ion-exchange capacity and low hydraulic conductivity to water. Bentonites used in GCLs mainly consist of the three-layered (2 : 1) clay mineral montmorillonite. Bentonite consists of the three-layer clay minerals of the smectite group (Figure 2.28). The basic three-layer structure consists of silica tetrahedron, alumina octahedron, and silica tetrahedron layers where the octahedral sheet is sandwiched between the two silica sheets as shown in Figure 2.28 (Özhan, 2011).

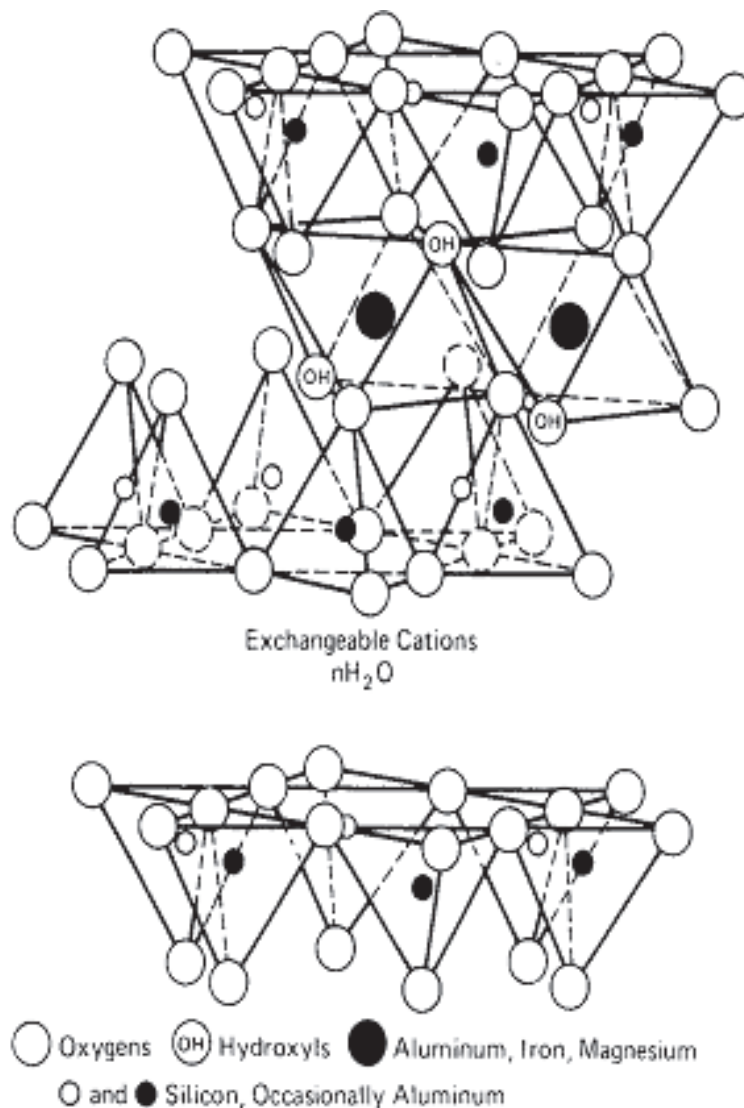


Figure 2.28. Three-layer structure of montmorillonite clay mineral (Özhan, 2011).

All the tips of the tetrahedral point towards the center of the unit cell. The oxygens forming the tips of the tetrahedral are common to the octahedral sheet as well. The anions in the octahedral sheet which fall directly above and below the hexagonal holes formed by the bases of the silica tetrahedral are hydroxyls. Three dimensional diagrammatic sketch of the montmorillonite structure is shown in Figure 2.29 (Özhan, 2011).

Bonding between successive layers is by van der Waals forces and by cations which balance charge deficiencies in the structure. These bonds are weak and easily

separated by cleavage or adsorption of water or other polar liquids (Özhan, 2011).

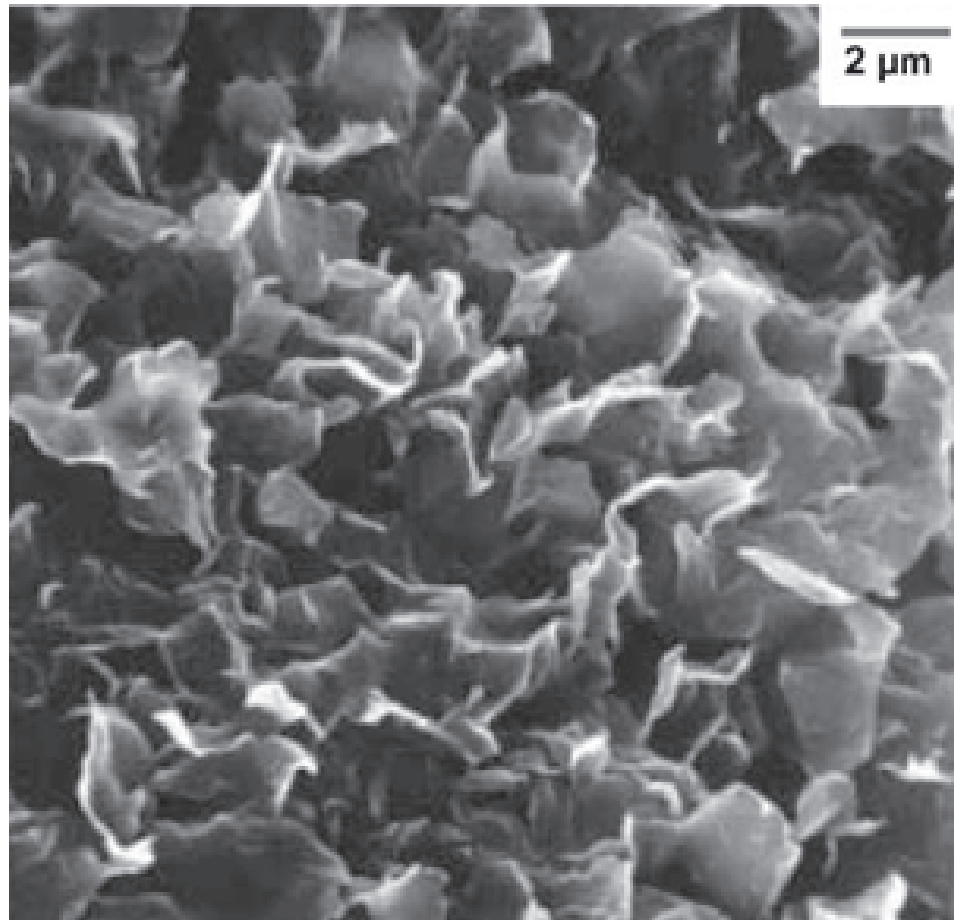


Figure 2.29. Three dimensional diagrammatic sketch of the montmorillonite structure (Özhan, 2011).

Further components of bentonite may be quartz, cristoballite, feldspars, mica (muscovite, biotite), illite, kaolinite, chlorite, carbonates (calcite, dolomite, siderite), zeolites, volcanic glass, amorph silicic acid and aluminum hydroxide, organic components, apatite, haematite, limonite, and heavy minerals (pyrite, magnetite, ilmenite, tourmaline, zircon, rutile) and others . Typical properties of bentonites in geosynthetic clay liners are shown in Table 2.9 (Özhan, 2011).

Table 2.9. Typical properties of bentonites in GCLs (Özhan, 2011).

Montmorillonite content (Methyleneblue method)	60 – 90%
Water absorption content	200 – 700 $weight\%$
Swelling volume (ASTM D 5890-95)	7-30 ml
pH value (suspension)	9-10.5
Plasticity (I_p)	140 – 380%
Friction angle (ϕ) / cohesion (c)	3 – 15°/3 – 10 kN/m^2
Permittivity (ψ)	$5 \times 10^{-8} - 3 \times 10^{-9} 1/s$
Cation Exchange capacity (CEC)	60 – 90 $mmol(eq)/100g$

Montmorillonite may occur as equidimensional flakes which are so thin as to appear more like films, as shown in Figure 2.30. Particles range in thickness from 10 $\text{\AA}$ unit layers upward to about 1/100 of the width. The long axis of the particle is usually less than 1 or 2 μm . When there is a large amount of substitution of iron and/or magnesium for aluminum, the particles may be lath or needle shaped because the larger Mg^{2+} and Fe^{3+} ions cause a directional strain in the structure. The specific surface area of smectite can be very large. The primary surface area, that is, the surface area exclusive of interlayer zones, ranges from 50 to 120 m^2/g . The secondary specific surface that is exposed by expanding the lattice so that polar molecules can penetrate between layers can be up to 840 m^2/g (Özhan, 2011).

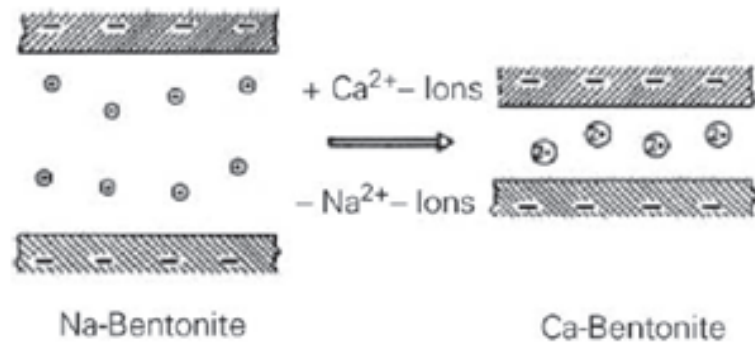


Figure 2.30. Electron scan microscope picture of montmorillonite in a pulverized natural sodium bentonite in a GCL (1 cm = 2 μm) (Özhan, 2011).

There are two main types of bentonites used in industrial facilities, sodium and calcium bentonites, which depend on their dominant exchangeable cation. But sodium bentonite is usually preferred because of its lower hydraulic conductivity. Generally, the sodium bentonite is a mixture of other ions such as magnesium, potassium and calcium. The main component of bentonite is montmorillonite which is a very good ion exchanger clay mineral. Montmorillonite has a high specific surface area, weak interlayer bonds, and high cation exchange capacity, which result in the high shrink and swell potential. Water and exchangeable cations can easily enter and separate the layers, causing montmorillonites to swell. Montmorillonite has negatively charged surfaces, as a result of the partial isomorphic substitution of Si^{4+} ions by Al^{3+} ions in the tetrahedron layers, and those of Al^{3+} ions by, for example, Mg^{2+} ions in the octahedron layer. The negative charge is balanced by cations, especially Na^+ , Ca^{2+} and Mg^{2+} ions, which are present both in the interlayer spaces and in the external surfaces. There are more calcium bentonites than sodium bentonites on earth. This is also due to the fact that calcium has a higher affinity. It is a well known fact that when a sodium bentonite gets in touch with free calcium ions, the sodium ions in the bentonite will be exchanged with calcium ions. In order to receive better swelling properties of sodium bentonite, calcium bentonite is activated with soda (sodium carbonate) and thus the primary calcium ions are exchanged by sodium ions (Özhan, 2011).

The properties of bentonite are greatly affected by the interactions between its

particles and the surrounding pore fluid. The net electrical charge on bentonite particle is negative. These negative charges cause dissolved cations in the surrounding pore water to be attracted to the surfaces of the clay. The layer of water and adsorbed ions which surrounds a clay particle is called the electrical double layer or diffuse double layer (Mitchell, 1993).

Ion exchange processes are known as equilibrium processes. The occupation of a cation exchanger depends on the concentration and type of the cations available for the exchange. The size and most of all, the charge of the cations are important. For example, bivalent calcium (Ca^{2+}) is more easily exchanged against monovalent sodium (Na^+) than vice versa. Evaluation of laboratory tests and field observations have shown that the ion exchange from calcium to sodium bentonite takes one to several years when GCLs are used in partly saturated areas (e.g. landfill cappings). The duration of the ion exchange depends on the primary cation amount of the bentonite, the quantity of bentonite per area unit, the geochemical conditions (concentration and ion distribution in the leachate), the geohydraulic conditions (water flow in the GCLs, the quantity of water drained off the GCL) and soil physical conditions (rising ionic concentration in the pore water by reduction of the water content due to desiccation, possibly extension of the contact areas by desiccation cracks). The exchange continues until an adsorption balance between the ion distribution at the bentonite and the ionic concentration in the soil solution is achieved. The ion exchange from monovalent sodium ions to bivalent calcium ions reduces the spaces between the silicate layers is shown in Figure 2.31 (Özhan, 2011).

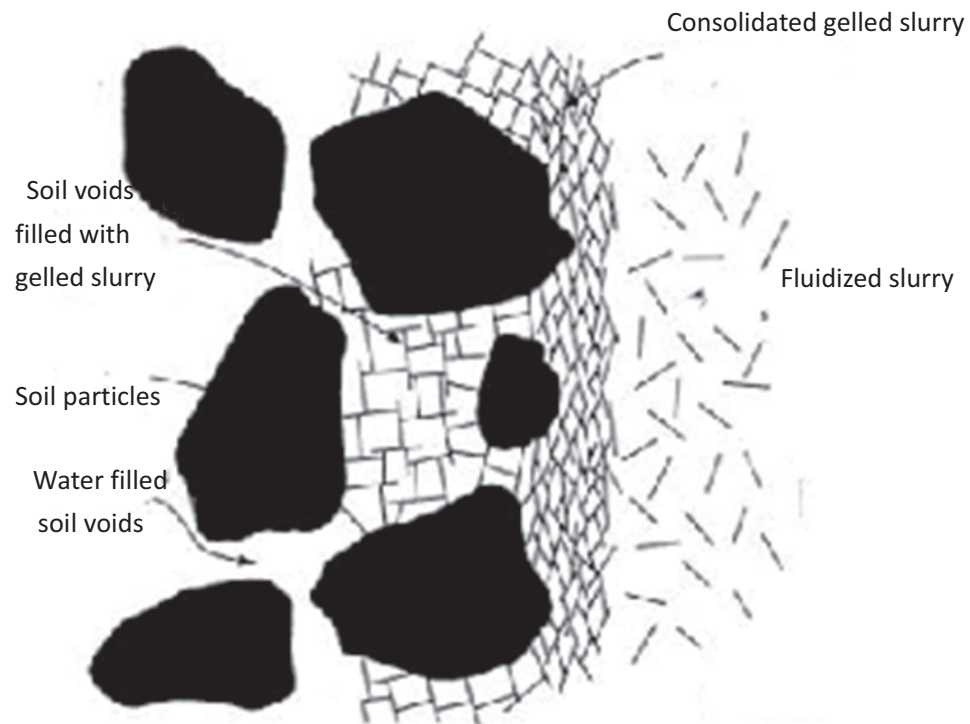


Figure 2.31. Ion exchange from sodium bentonite to calcium bentonite (Özhan, 2011).

2.8.2. Bentonite Slurries

Bentonite slurries are made by adding bentonite to fresh water and mixing in a high-shear mixer (to ensure proper dispersal of the clay particles), and then leaving the slurry for a recommended time to ensure sufficient hydration of the clay. Slurries are usually formed from a few percent of bentonite in water. Bentonite slurries are thixotropic and typically form a gel at concentrations above about 5% by weight in water (Milligan, 2000).

Bentonite - natural clay minerals - and water are mixed colloidal type grout mixers at the jacking pit area and ideally are left to swell for several hours before use. The ready suspension is then pumped into the over break void through a temporary pipe system connected to injection sockets cast into the pipes. The arrangement and distribution of injection sockets has often been a matter of experience but should be designed to ensure an even distribution of suspension around the pipe string. If a continuous layer of lubricant suspension can be formed, the pipes could theoretically

become buoyant. Some authors including (Haslem, 1986) and (Stein et. al, 1989) have indicated that pipes may become buoyant in ideal conditions, but have been unable to substantiate the suggestion (Marshall, 1998).

Bentonite suspensions have thixotropic properties - temporarily liquid when agitated but of a gel consistency when at rest. In cohesionless soils excess fluid pressures are expected to form in the suspension and the dissipation will be restricted by the thixotropy of the material (Marshall, 1998).

Stein et al (loc cit) lists the factors influencing the effectiveness of bentonite suspensions as:

- (i) the quality and grade of bentonite;
- (ii) the consistency of the suspension, and
- (iii) control of injection pressures and volumes pumped.

High grade bentonite swells readily and provides lubricant of the required viscosity. Pre-treating the mixing water for hardness and acidity may also help maximize yield. The consistency of the suspension and injection pressure is reported as greatly influencing lubricant efficiency a stiff suspension is said to yield better results than one of a thin consistency. A lower injection pressure is thought to allow a steady flow of suspension and a more uniform distribution whereas high pressures may force the volume of pumped material only into certain areas (Marshall, 1998).

2.8.3. Applications of Bentonite Injection

During microtunnelling and pipe jacking, in order to keep the jacking force as low as possible, appropriate measures are necessary to reduce the frictional resistance, e.g. cutting of an annular gap combined with the injection of lubricants into this annular gap. The frictional resistance (skin friction) is decreased effectively by the following effects (Schoesser, 2011):

- Reduction of the radial stress on the surface of the jacking pipes by maintaining the annular gap (stabilization of the ground with the effect that the surrounding subsoil does not transfer the earth pressure to the total circumference of the jacking pipe).
- Reduction of the friction coefficient μ_{int} between the surface of the jacking pipes and the subsoil by means of a lubrication layer.

The principal use of bentonite in tunnelling is in slurries in slurry shields and as a ground support and lubricating medium around jacked pipes. Such slurries generally have only about 3% by weight of bentonite, giving a Marsh cone reading of 30 – 40 seconds. The first requirement of the slurry is that it should form a filter cake of consolidated and gelled bentonite against and within the soil, which then becomes a low permeability membrane able to transmit fluid pressure in excess of groundwater pressure into effective stress between soil particles and hence stabilize the ground (Figure 2.32). To do this it must not be able to penetrate too readily into the ground; in open soils a slurry may simply dissipate into the ground without transmitting any significant stresses into the soil, causing large consumption of bentonite and poor ground support. (Jancsecz and Steiner, 1994) studied the penetration of slurry and produced the formula;

$$S = \frac{\Delta p d_{10}}{3.5 \tau_s} \quad (2.20)$$

where s is the penetration distance, d_{10} is the particle size such that 10% by weight of the soil is of smaller particle size, s is the shear resistance of the slurry, and Δp is the difference between the slurry pressure p and the ground water pressure u . For clean slurries and typical values of Δp , penetrations of several meters result in soils any coarser than medium sand with typical shear strengths of the slurry of 20 to 50Pa, and will be unacceptable when d_{10} exceeds 2 to 3 mm. However if the slurry contains soil fines (silt and fine sand) which help to block the pores, penetration is greatly reduced and there has been satisfactory experience in both the UK and Japan of using slurry support in much coarser soils (Milligan, 2000).

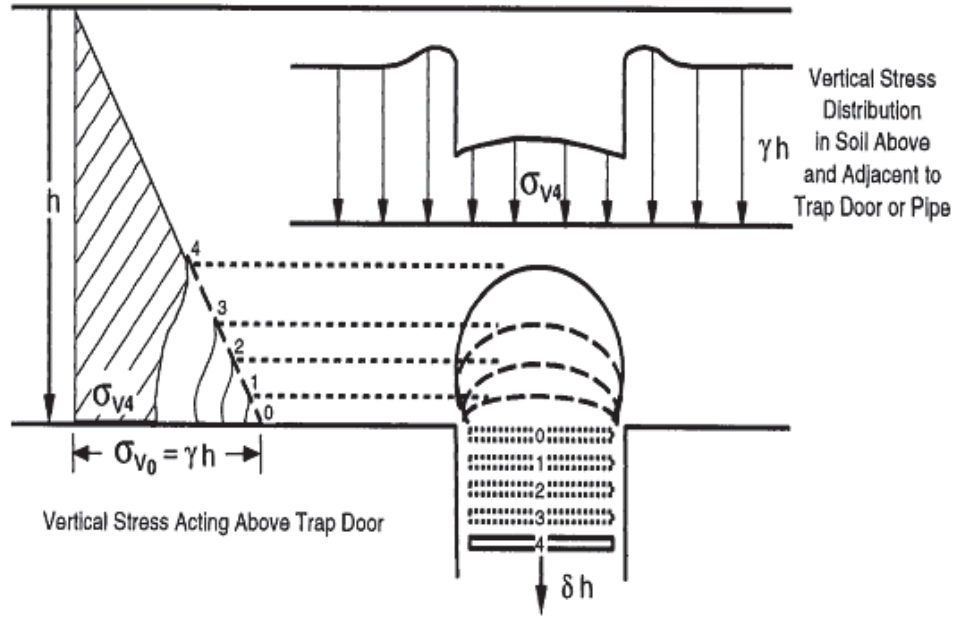


Figure 2.32. Formation of filter cake (Milligan, 2000).

Jefferis (1992) provides an expression which includes the porosity of the soil n :

$$S = \frac{\Delta p d_{10}}{\tau_s} \frac{n}{(1 - \eta)} f \quad (2.21)$$

where f is a factor to take account of the geometry and tortuosity of the flow paths within the soil, and may be about 0.3. For typical values of porosity this gives considerably shorter penetration lengths than does Jancsecz and Steiner's expression (Milligan, 2000).

(Anagnostou and Kovari, 1996) quote a similar expression from the German Standard DIN- 4126 (1986):

$$S = \frac{\Delta p d_{10}}{2.0 \tau_s} \quad (2.22)$$

They use this to determine slurry penetration into soils of different grain size, and hence the pressure gradients within the ground in front of the shield and the resulting factor of safety against face collapse. They then investigate the inter-related

effects of grain size, soil permeability, slurry pressure, slurry shear strength, rate of slurry penetration and tunnelling advance rates. They conclude that slurries with 7% bentonite content may be used to extend effective support for coarser soils with d_{10} grain sizes of up to 6 mm, but problems with face instability are likely to occur for practical tunnelling rates once the soil permeability exceeds about 10^{-4} m/sec. In practice slurries with such high bentonite concentrations are difficult to work with, and it is much more effective to use thinner slurries containing silt and fine sand to act as pore-blocking material. To reduce penetration and improve cake formation, polymers may be added. Their long-chain molecules act as reinforcing fibers and form a net to retain the bentonite particles, bridging the soil pores. Sodium CMC polymers have been extensively used in Japan for this purpose, for which PAC materials are also suitable. They may act well in combination with silt and fine sand from the previously excavated soil which has been retained in the slurry and will block pores between larger particles. (Steiner, 1996) was able to investigate the penetration of different density slurries into a well-graded glacial gravel with a small fines content and 10 – 20% in the sand range. Slurry with a density of 1.03 to 1.05 Mg/m^3 penetrated 6 to 8 m. Increasing the slurry density to 1.08 to 1.12, with 50 kg/m^3 bentonite, and adding 1 kg/m^3 of polymer and retaining some sand, reduced the penetration to 4 to 6 m. Slurry with a density of 1.14 to 1.24 penetrated 0 to 2 m. The pressure differential is not given (Milligan, 2000).

If lubrication of the pipe line is required when pipe jacking in clay, an inhibited bentonite slurry may be needed. Plastic clays which come into contact with aqueous slurries will absorb water and swell. The ground may swell sufficiently to close the over break and increase the contact stresses between pipe and ground and hence the jacking resistance. There are two methods of protecting clays in this instance: changing the reactivity of the clay and providing a barrier between water and clay particles. The latter is achieved using one of the highly anionic polymers specially produced for this purpose, such as PHPA, which act by binding the surface layer of water to the clay surface and preventing any further ingress of water to the clay particles. Only small dosages are needed, typically of the order of 0.05 to 0.1% (Milligan, 2000).

2.9. Calculation of Coefficient of Friction at the Jobsite

One method for predicting the normal load for microtunneling applications is based on trap door experiments conducted by Terzaghi in 1936 on the silo theory. These experiments investigated the effects of arching in soils (Terzaghi, 1943). In these experiments, dry, cohesionless material was placed on a platform between two plates, as illustrated in Figure 2.33. The platform had a trap door mounted to a scale that would measure the pressure on the door. The depth of the sand layer was several times greater than the width of the door. When the trap door was closed, the pressure on the platform was equal to the soil density (γ) multiplied by the height of the soil (H). However, when the trapdoor was allowed to yield in the downward direction, i.e. opened, the pressure on the trapdoor decreased to a small fraction of its original value, even for very small deflections, and the pressure on the adjoining parts of the platform increased. Terzaghi attributed this decrease in pressure to the resisting shearing stresses that developed in the sand above the trap door. He also stated that if the sand has a trace of cohesion, the trapdoor can be removed entirely and the sand will not drop out of the gap. This concept of reduced vertical pressure due to arching can be applied to microtunneling because the microtunneling machine produces an overcut around the pipeline, effectively removing the trap door and inducing the arching state of the soil (Staheli, 1996).

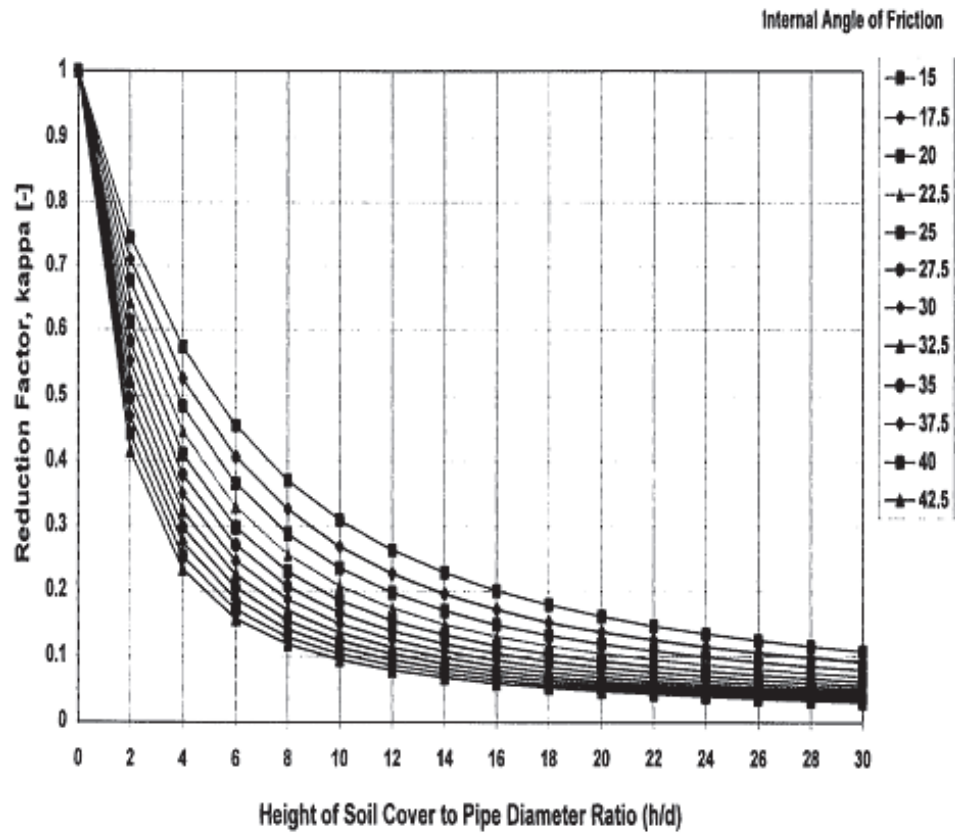


Figure 2.33. Terzaghi's trap door experiment (Staheli, 1996).

Stein et al. (1989) further developed a reduction factor called kappa, based on Terzaghi's trap door experiments, specific to soil types based on the angle of friction of the soil and the depth of soil cover over the tunnel. This reduction factor can then be multiplied by the full column load over the pipeline (γH) to determine the vertical pressure acting on the pipeline Figure 2.34 (Staheli, 1996).

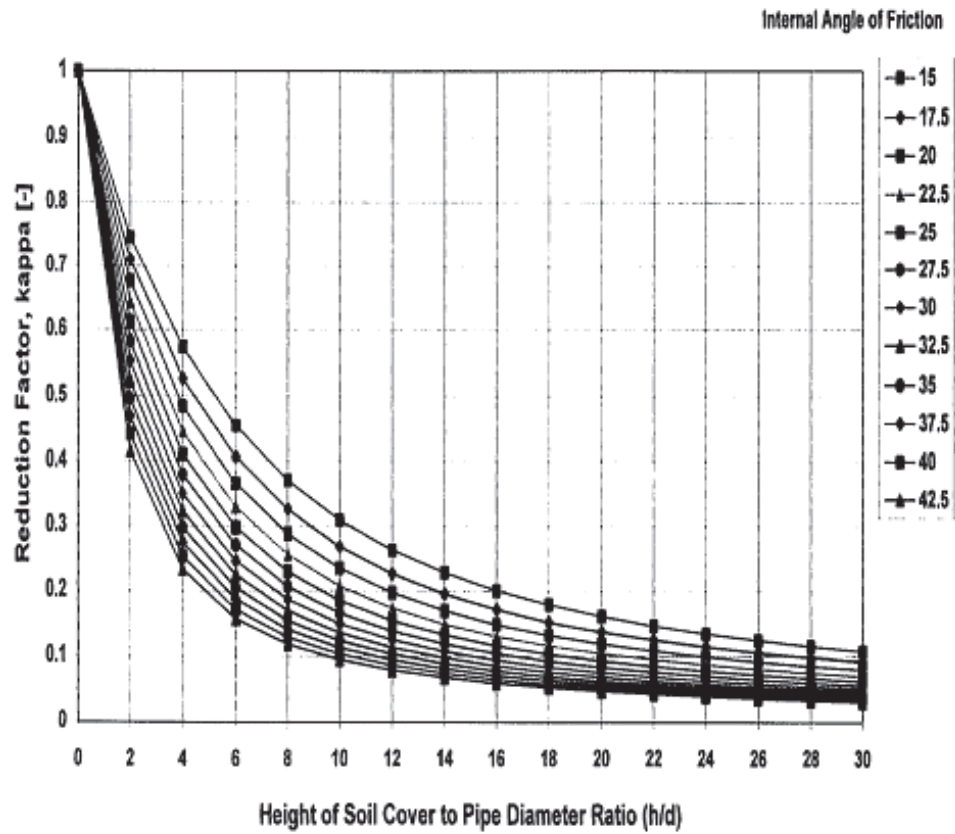


Figure 2.34. Reduction factor as a function of height of soil cover to pipe (Staheli, 1996).

$$\sigma_v = \kappa \gamma h (t/m^2) \quad (2.23)$$

where:

σ_v : Vertical stress on pipes

κ : reduction coefficient depending on ratio of $\frac{h}{d_e}$

h : height of cover

d_e : pipe external diameter

L : length of drive

τ : friction (resistance) stress

μ_{int} : coefficient of friction

F_T : total force

F_{face} : penetration force

F_{net} : net friction force

F_{net} : $F_T - F_{face}$ (ton)

τ : $\frac{F_{net}}{\pi d e L}$ (t/m^2)

μ_{int} : $\tau - \sigma_v$

2.10. Summary

In this chapter not only the models that exist for predicting the normal stresses and frictional components of jacking forces associated with microtunneling and pipe jacking operations are reviewed but also recent information about bentonite injection and rheology are summarized. The effects of lubrication were examined on a limited basis within these models.

The benefits of bentonite lubricants in maintaining tunnel bores and reducing frictional resistance have been recognized but there is still little attempt to standardize their use. Many of researchers, by examination of total jacking loads only, have demonstrated that lubrication is beneficial and can reduce frictional resistance. To better understand the function of lubrication further work was required to investigate their effects on pipe-soil interaction. One of the main aim of this thesis is improving the knowledge of the lubricity mechanism at the pipe-soil interface in order to ensure that lubricants are properly and effectively used.

3. METHODOLOGY

3.1. Introduction

In order to investigate bentonite injection mechanism in sand, two new lab test setups are prepared in Bogazici University Geotechnical Laboratory for this PhD thesis. At the first stage, bentonite injection is observed on sand sample in a Plexiglas cell. At bentonite injection test it was tried to simulate the bentonite injection in pipe jacking operations at the jobsite in which bentonite slurry is injected from the top of the pipes into the overcut and soil. During the bentonite injection in sand at the laboratory it is observed and recorded to find what the optimum concentration of sodium bentonite slurry is and which factors affect the stability of bentonite slurry are. At the second stage of bentonite injection, interface friction of sand and concrete is researched at shear box test, on the samples taken from the tests which was done at the first stage.

At the third stage of laboratory tests a new laboratory set up is used. In order to simulate frictional conditions of pipe jacking operation at the jobsite , a Plexiglas cell is filled with sand and a concrete pipe sample which models concrete pipe is put in sand mass. When the test concrete sample was pulling with a constant velocity which likes average pipe jacking progress velocity at the jobsite, frictional forces are recorded. At the first step , no bentonite is used to simulate no bentonite case at the field. At the second step, periphery of the concrete sample is filled with bentonite slurry to simulate the mass controlled bentonite injection at the jobsite. At the third step, not only periphery of concrete sample is filled with bentonite slurry but also at least five centimeters of bentonite slurry kept constantly remain on the concrete sample to simulate bentonite injection with constant injection pressure at the jobsite. By using same sand and bentonite slurry, but changing the injection conditions, not only the interface friction coefficient of sand and concrete sample without bentonite and with bentonite is measured but also optimum injection pattern is investigated. At the end, after the third stage microscope photo of the samples are taken to observe the efficiency of the bentonite injection.

3.1.1. Bentonite Injection Methods for Pipe Jacking Method

At the practice there are three types of bentonite injection; first one is controlled injection which use a limited amount of bentonite without a continuous pressure (Figure 3.1). The second type of injection is mass application which use a lot of bentonite (Figure 3.2). Mass application lubrication refers to lubrication implementation when jacking forces were allowed to increase without the use of lubrication and then lubrication is applied in large volumes along the pipe string throughout areas that had been previously tunneled. Controlled lubrication refers to the use of smaller volumes of lubrication throughout tunneling operations, often from only a single lubrication port that serves to keep jacking forces within a lower range throughout tunneling operations. The third method is automatic bentonite lubrication which keeps pressure of injected bentonite constant free of bentonite amount (Figure 3.3).



Figure 3.1. Controlled injection of bentonite (Staheli, 2006).



Figure 3.2. Mass injection application (Staheli, 2006).

The automatic bentonite injection system is fully programmable to allow the operator to inject a pre-determined amount of bentonite at all or specific areas of the tunnel without man entry to the tunnel. A pressure level control unit is connected to a number of injection stations in the tunnel bentonite line, the injection stations are normally installed at 15 metre intervals. Each injection station consists of a control box and a triple spool electro-pneumatically operated valve block assembly. The injection stations are numbered and assigned a unique address code which means each station must be installed in the pipeline in the correct sequence. Three bentonite lines from the injection station distribute bentonite to the injection ports as in any conventional or manual pipe jack lubrication system.

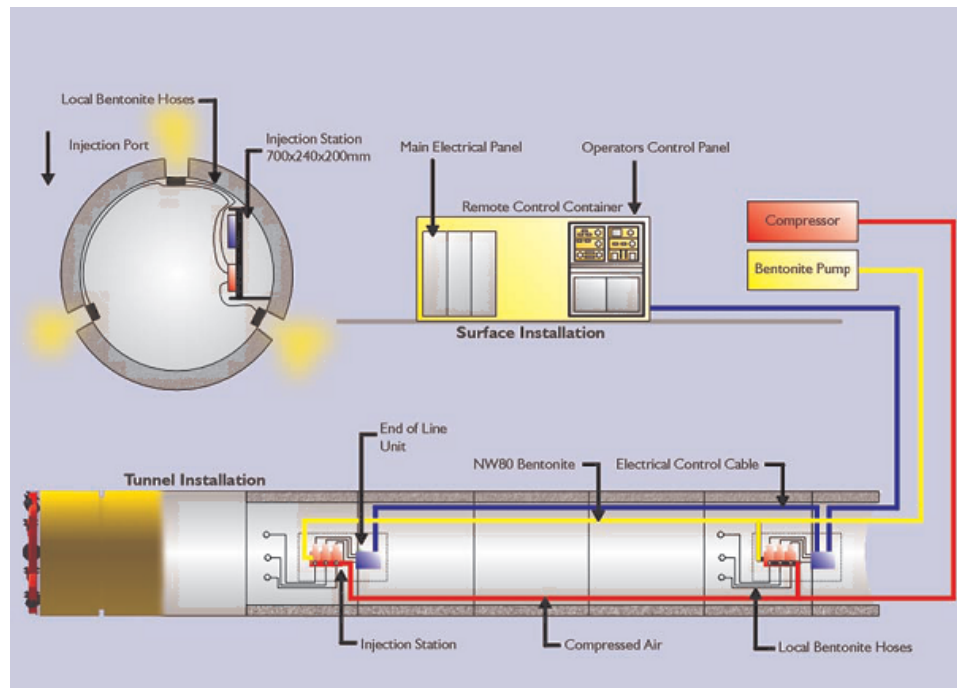


Figure 3.3. Automatic lubrication system.

The use of automatic lubrication systems from the beginning of the drives has been reported and resulted in remarkably small jacking forces at Table 3.1. However, it is not clear if the low jacking forces result from (i) the early introduction of lubrication, (ii) the automatic lubrication system or (iii) from a lubricant composition of superior quality (Staheli, 2006).

Table 3.1. Summary of experimental program.

Reference / Project	Ground conditions	Pipe OD / excavation	Depth to axis m	Lubrication system	Lubrication	Unit jacking force kN/m
Alexanderson (2001)	Medium hard	3.04	12-14	Automatic, 3 ports	water	19-48
Vibhavadi Bangkok	marine clay	3.20		ever 15 m	bentonite polymer	19-29 4.8
Broomfield (2004)	Soft Bangkok clay	1.625 n/a	approx. 5m	Automatic, 3 pots every 18m	bentonite	2.0-3.7
Minburi water pipeline, bangkok						
Milligan and Marshall	Soft to very soft	1.5	5.5-6.0	Second half	none	25
Thurrock sever (1998)	silty clay	n/a		of drive only	bentonite	14
system, UK						
Milligen and Marshall	unfathored highly	1.8 m	8.5	After 20 m drive, but only at three	none	12.7
(1998) Leytonstone	plastic firm of				bentonite	
sewer system, UK	stiff London Clay					
Milligan and Marshall	Stiff glacial till	1.8	7	n/a	none	48
(1998) Seaham sever	(or clay)				bentonite	15
system, UK						
Norris (1992)	Stiff over consolidate	2.27	11-21	none	none	54.4
Honor Oak, UK	London Clay	n/a				

3.1.2. Filtration Patterns for Injection

For injection applications there are three types of filtration patterns. First one is surface filtration in which injection penetrates less than 40 mm, second one is deep filtration in which injection penetrates between 40 - 50 mm, and third one is rheological blocking in which injection depth reaches longer than 1 meter. But which one is effective in pipe jacking has not been understood, yet.

Both to investigate filtration pattern and to investigate bentonite injection mechanism, two new lab test setups are prepared in Bogazici University Geotechnical Lab for this PhD thesis. In this study, three stages of laboratory investigation is done with pipe jacking jobsite researches. First stage of test is named as bentonite injection sand. At bentonite injection test it is observed and recorded to find what the optimum concentration of sodium bentonite slurry is and which factors affect the stability of bentonite slurry are. At the second stage, interface friction of sand and concrete is researched at shear box test, on the samples taken from the tests which was done at the first stage. At the third stage, not only the interface friction coefficient of sand and concrete sample without bentonite and with bentonite is measured but also optimum injection pattern is investigated. At the end, after the third stage optical microscope photo of the samples are taken to observe the efficiency of the bentonite injection.

3.2. Observation of Bentonite Injection in Sand at Laboratory Tests

3.2.1. Laboratory Test Set Up

In order to investigate and make observation on the injection of bentonite slurry in sand during pipe jacking, a test setup prepared as shown at Figure 3.4. Plexiglas cell has 10,5 cm diameter and it is filled with sand that has 20 cm height. At the beginning sand is saturated with water to represent the case below the water table. Then 5 cm height bentonite slurry is poured on the saturated sand and cumulatively rising pressure with 25 Kpa pressure steps each for 5 minutes is applied up to 400 KPa on the bentonite slurry existing on the sand.



Figure 3.4. Bentonite injection in sand test set up.

3.2.2. Materials Used in Bentonite Injection Test

Different concentration of bentonite slurries with different waiting times were observed by measuring their first and last marsh viscosity (Table 3.2). The bentonite and the sand used in the test kept constant. Properties of the sand used in the test is given in Table 3.3 and Figure 3.5. The internal friction angle of the sand used in the test is found $\phi=30^\circ$ from shear box test (Figure 3.6).

Table 3.2. Marsh funnel viscosity readings.

Bentonite Amount in 1 liter of water	30 gr	40 gr	50 gr	60 gr
Marsh Viscosity after preparation	30 sec	34 sec	41 sec	62 sec
Marsh Viscosity after 24 hours	32 sec	38 sec	54 sec	120 sec
Bleeding water after 24 hours	18%	10%	5%	0%

Table 3.3. Properties of sand used in bentonite injection test.

Properties of Soil	Sand
Unit weight, $\gamma(kN/m^3)$	15.8
Specific gravity of solid constituents, G_s	2.59
Effective Grain Size, D_{10} mm	0,31
Uniformity coefficient, C_u	3.23
Coefficient of curvature, C_c	1.32

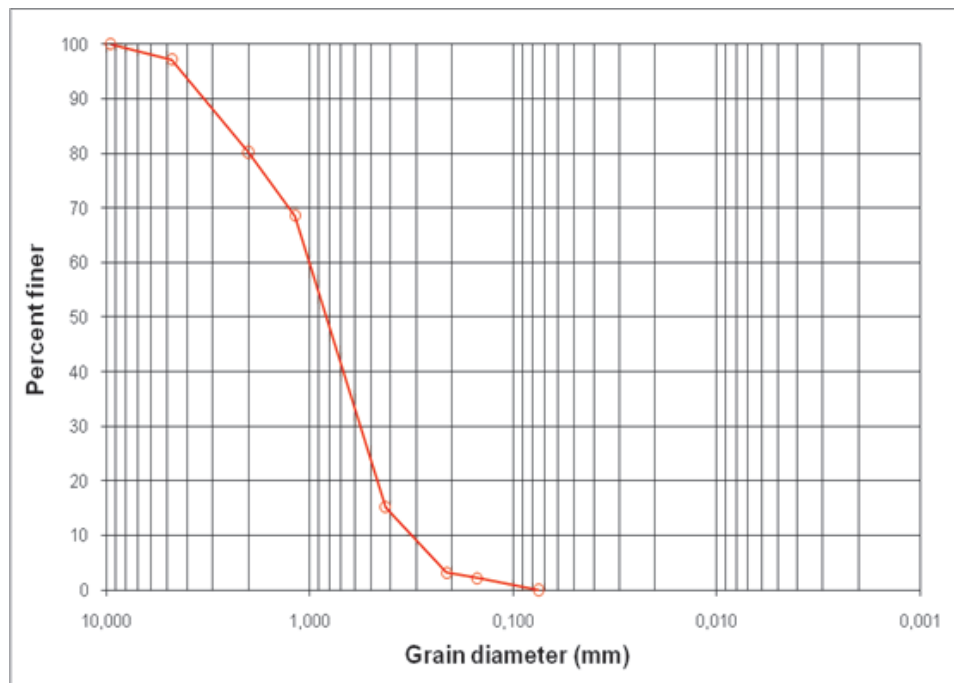


Figure 3.5. Granulometry of sand used in bentonite injection test.



Figure 3.6. Shear box test for sand.

In bentonite injection test sodium bentonite which is named Resadiye bentonite that exists in Turkey was used. According to standard of EN ISO 13500 bentonite apparent viscosity which means fan viscosity reading of 600 round per minute (*rpm*) should be higher than 30 *cP*. Filtration volume, between 7,5 minutes and 30 minutes at filter press test should be less than 15*ml*. Definition of rheological parameters are summarized below:

- Apparent viscosity = 600 *rpm* fan viscosity reading.
- Plastic viscosity = 600 *rpm* - 300 *rpm*
- Yield point = 300 *rpm* - Plastic viscosity

Rheological parameters of sodium bentonite which was used in laboratory test was measured at Istanbul Technical University Mining Faculty, Mineral Processing Engineering Department, Surface Chemistry Laboratory. These Parameters are summarized at Table 3.4.

Table 3.4. Rheological parameters of resadiye bentonite.

Type of Bentonite	Sodium Bentonite
Cation Exchange Capacity	70 meg / 100 gr
Ph	9,9
Moisture	10%
Apparent Fan Viscosity (600 rpm)	54 cP
Plastic viscosity (600 rpm - 300 rpm)	4 cP
Filtration volume	14 ml
Swelling volume (for 2 gr bentonite after 24 hours)	29 ml

3.2.3. Interface Coefficient Between Concrete and Bentonite-Sand Mixed Zone

At the second stage, interface coefficient between dry sand-concrete and between concrete-sand after bentonite injected are compared. For this comparison, the soil sample which is taken from the top contact zone of sand and bentonite slurry after the injection test and the concrete which is prepared at the lab is used in shear box test (Figure 3.7). At the end of shear box tests, soil the interface friction coefficient between dry sand-concrete is measured as μ_{int} between sand and concrete = 0,90, and soil the interface friction coefficient between concrete -sand after bentonite injected is measured as μ_{int} between sand-bentonite contact zone and concrete = 0,46, respectively.



Figure 3.7. Shear box test for sand-bentonite and concrete interface.

3.2.4. Results of Bentonite Injection and Shear Box Test

Fifteen tests of bentonite slurry injection with 3%, 4%, 5%, 6%, 7% bentonite concentrations were done in the first stage of laboratory tests. As a result it is observed that optimum concentration of bentonite in order to obtain a stable slurry is 6% bentonite in water. This means that 60 gram bentonite in 1 lt of water. Bentonite slurry should be wait at least one day for hydration. For the sand that has effective particle diameter (D_{10}) 0,32 mm which was used in the first stage of laboratory tests, it can be said that filtration result is observed as surface filtration at Figure 3.8. If waiting duration is not adequate for hydration, loose of bentonite becomes too much as shown at Figure 3.9 which means bentonite slurry can not provide membrane effect.



Figure 3.8. 6% Bentonite injection after 24 hours of hydration.



Figure 3.9. 5% Bentonite injection just after preparation.

From the shear box results and bentonite injection tests it was observed that even though bentonite slurry does not remain between pipe and soil the interface friction coefficient μ_{int} (μ_{int} between sand-bentonite contact zone and concrete = 0,46) decreased to half of the sand concrete interface friction (μ_{int} between sand and concrete = 0,90) which looks similar to the decrease of friction forces after bentonite injection at the jobsite cases.

3.3. Sand Concrete Interface Friction Tests With and Without Bentonite

3.3.1. Laboratory Test Set Up Sample Preparation

The measurement of interface friction between soil and concrete preparation of sample was not easily repeatable at the second stage of bentonite injection test. As the third stage of laboratory tests, a new lab test setup is prepared at Bogazici University Labs in order to measure sand - concrete pipe interface friction by modeling concrete jacking pipe and sand interface friction as shown in the Picture below. This new test setup is prepared not only to model the job site conditions properly but also to repeat tests under same conditions by keeping all parameters constant except bentonite injection. One of the main aim of this test is to find the effects of both pressure and amount of bentonite injection on the lubrication efficiency which have not clarified yet.

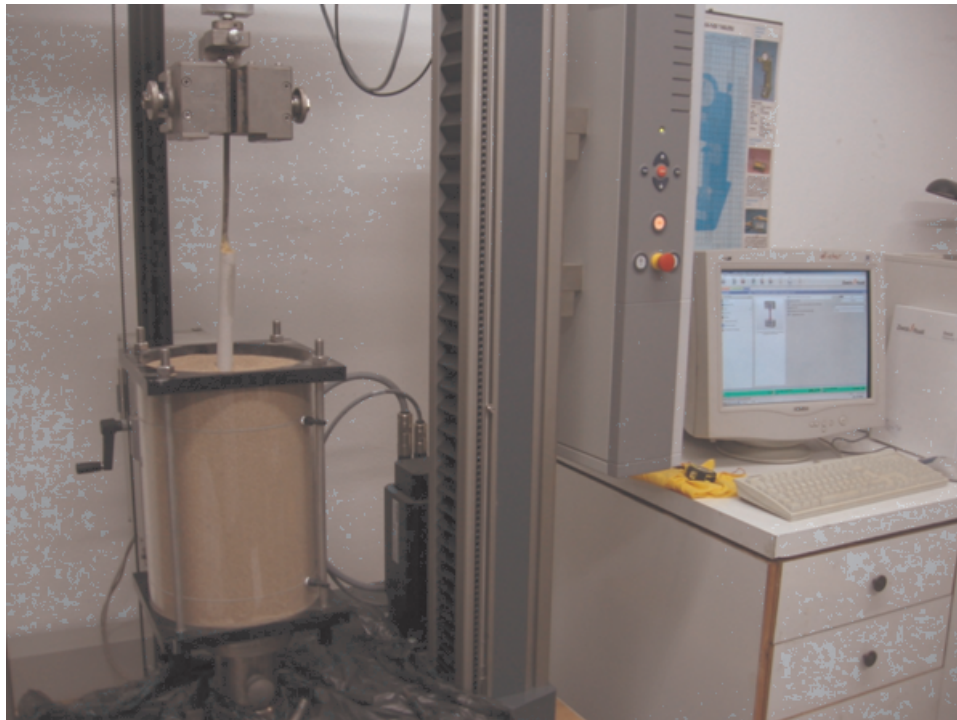


Figure 3.10. Sand-concrete interface friction force measurement test set up.

To measure friction force between sand and concrete, a Plexiglas cell which has 32 cm height and 23 cm diameter is filled with Sile sea sand up to 10 cm (7,5 kg sand) then concrete sample that represents the pipe jacking pipe which has 2 cm diameter

and 20 cm height is put on the sand. The void around the concrete sample is filled with 15 kg of the same sand.

The test cell is subjected to vibration for 1 minute and the relative density of the sand is measured as 89% (ASTM-D4253). The concrete sample is pulled from the sand with 50 mm/min velocity to represent actual jobsite mean progress velocity.

As mentioned before, there are three types of bentonite injection in the practice. First one is controlled injection which use a limited amount of bentonite without a continuous pressure. The second type of injection is mass application which use a lot of bentonite, the third method is automatic bentonite lubrication which keeps pressure of injected bentonite constant free of bentonite amount.

To represent practice, three types of situations are tested under same conditions. Firstly the situation without bentonite is measured, secondly the worst bentonite injection is represented by putting a tube around the concrete and filling the tube with bentonite then tube is pulled off. Thirdly concrete is put in a tube then sand is filled and periphery of concrete is filled with bentonite and tube is pulled up to the surface of sand and bentonite feeding goes on until at least 5 cm of bentonite will remain over the surface of sand to represent fully pressurized injection application.

3.3.2. Materials Used in the Test

Resadiye bentonite which was used in bentonite injection test is used in the third stage, too. For sand sample uniform Sile sea sand was used (Table 3.5 and Figure 3.11).

Table 3.5. Properties of Şile sea sand.

Unit weight,	$\gamma = 16.7 \text{ (kN/m}^3\text{)}$
Uniformity coefficient,	$C_u = 2,$
Effective Grain Size,	$D_{10} = 0,2 \text{ mm}$
Coefficient of curvature,	$C_c = 1.13$

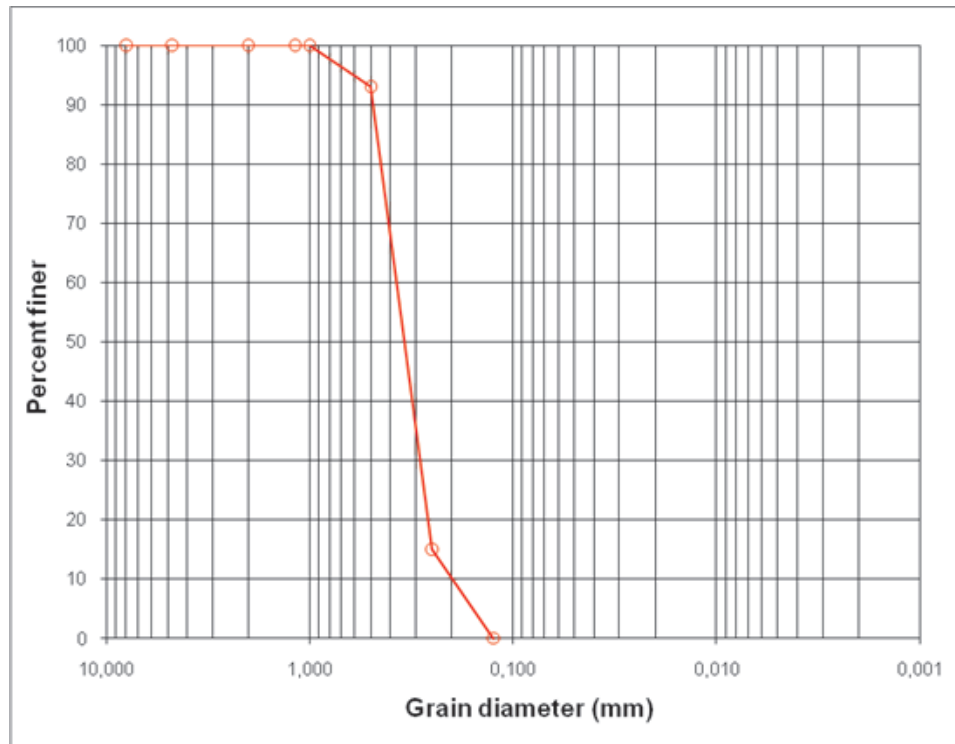


Figure 3.11. Granulometry of Sile sand.

3.3.3. Results of Sand Concrete Interface Friction Tests With and Without Bentonite Injection

3.3.3.1. Result of Test 1. The situation without bentonite is modeled in Test 1. After the preparation of sand, the test cell is subjected to vibration for 1 minute and the relative density of the sand is measured as 89%. (ASTM-D4253). The concrete sample is pulled from the sand with 50 mm/min velocity to represent actual jobsite mean progress velocity. Result of Test 1, dense sand and concrete interface friction force graphic is shown in Figure 3.12. From Figure 3.12 interface coefficient of friction μ_{int} can be found as 0,46. To observe the surface of concrete the optical microscope photo of concrete is also shown at Figure 3.13.

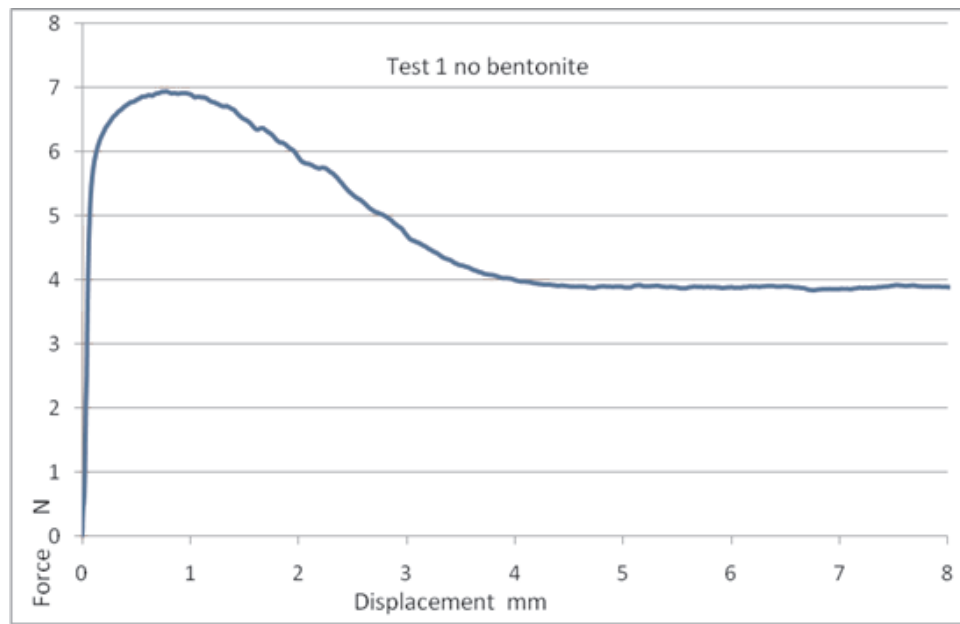


Figure 3.12. Test 1, dense sand and concrete interface friction force without bentonite.

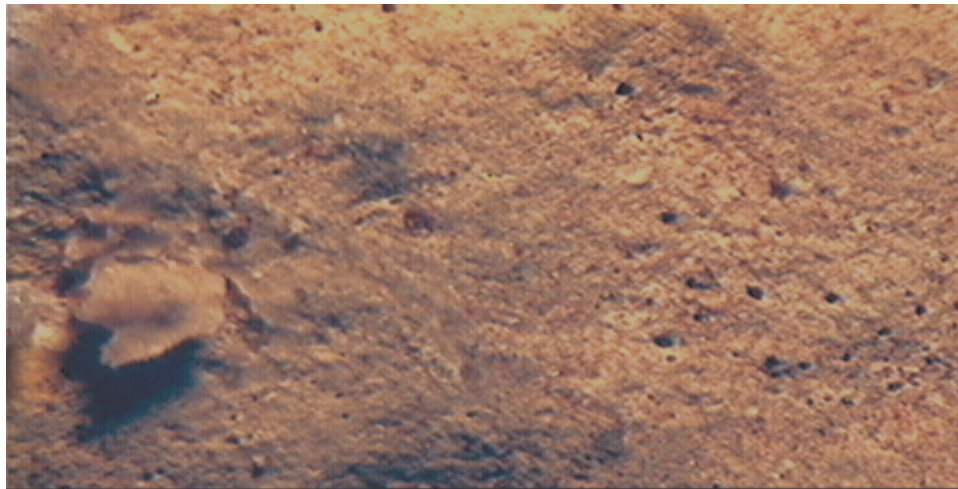


Figure 3.13. Surface of concrete sample.

3.3.3.2. Result of Test 2. At Test 2 sand sample is prepared like in Test 1, but in this case bentonite is poured around the concrete sample to model controlled bentonite injection. The controlled bentonite injection is represented by putting a tube around the concrete and filling the tube with bentonite then tube is pulled off. In this case concrete is submerged to bentonite before test but there is not any injection pressure during test. Aim of this test is modeling inadequate bentonite injection without constant injection pressure. Result of Test 2, dense sand and concrete interface friction

force graphic is shown in Figure 3.14. From Figure 3.14 interface coefficient of friction μ_{int} can be found as 0,24. The optic microscope photo of concrete which shows interface surface is at Figure 3.15.

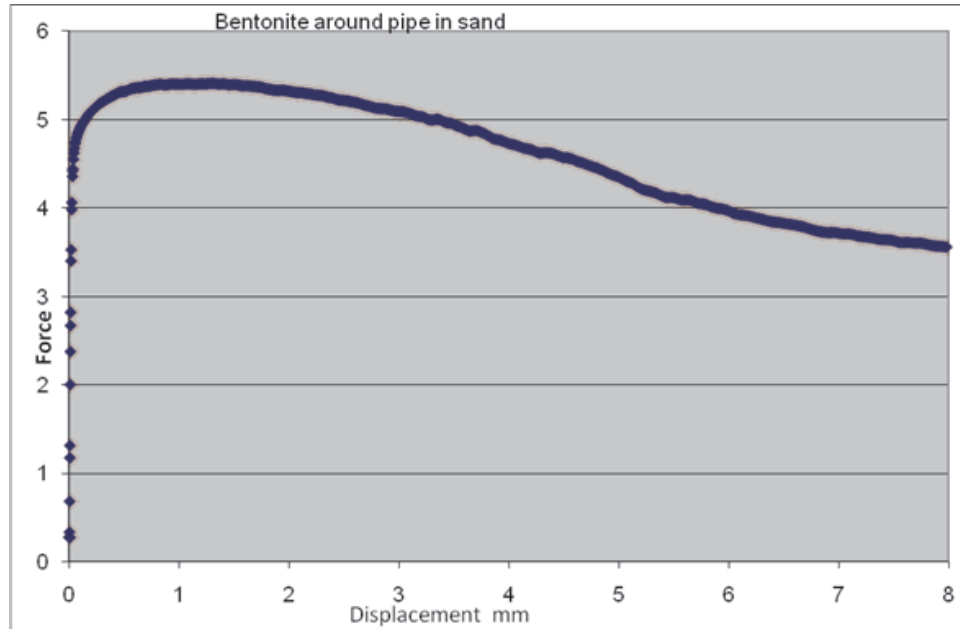


Figure 3.14. Test 2, concrete is submerged to bentonite before test but there is not any injection pressure (inadequate bentonite injection without constant injection pressure).

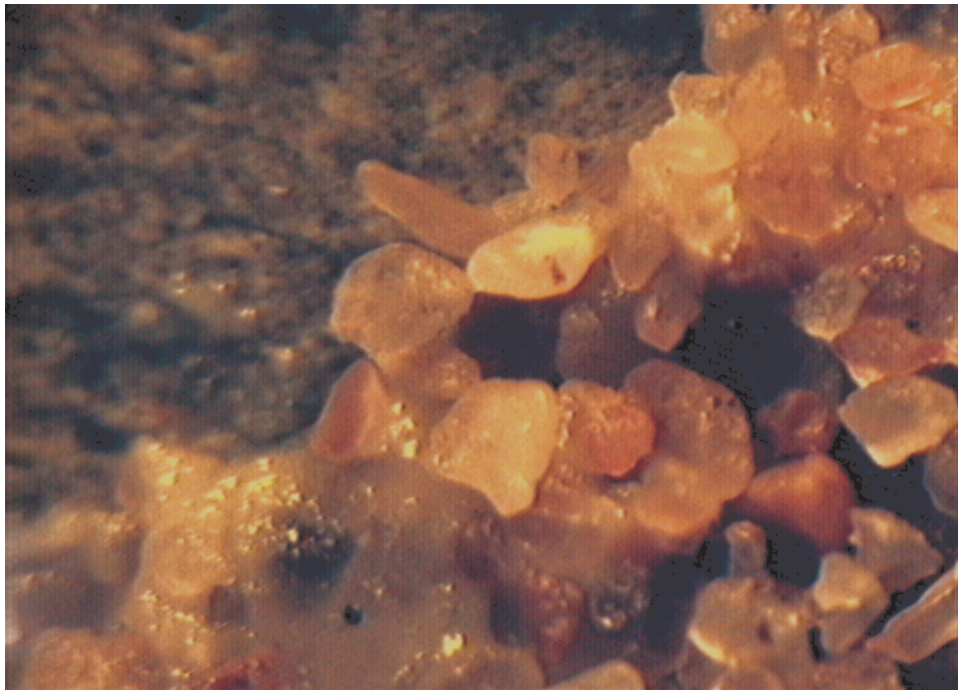


Figure 3.15. The optical microscope photo of concrete after Test 2.

3.3.3.3. Result of Test 3. At Test 3 sand sample is prepared like in Test 1 and Test 2, but in this case the concrete pipe sample is put in a tube then sand is filled and vibrated. Then, periphery of concrete is filled with bentonite and tube is pulled up to the surface of sand and bentonite feeding goes on until at least 5 cm of bentonite will remain over the surface of sand to represent fully pressurized injection application. Result of Test 3, dense sand and concrete interface friction force graphic is shown in Figure 3.16. From Figure 3.16 interface coefficient of friction μ can be found as 0,06. The optical microscope photo of concrete which shows surface is at Figure 3.17.

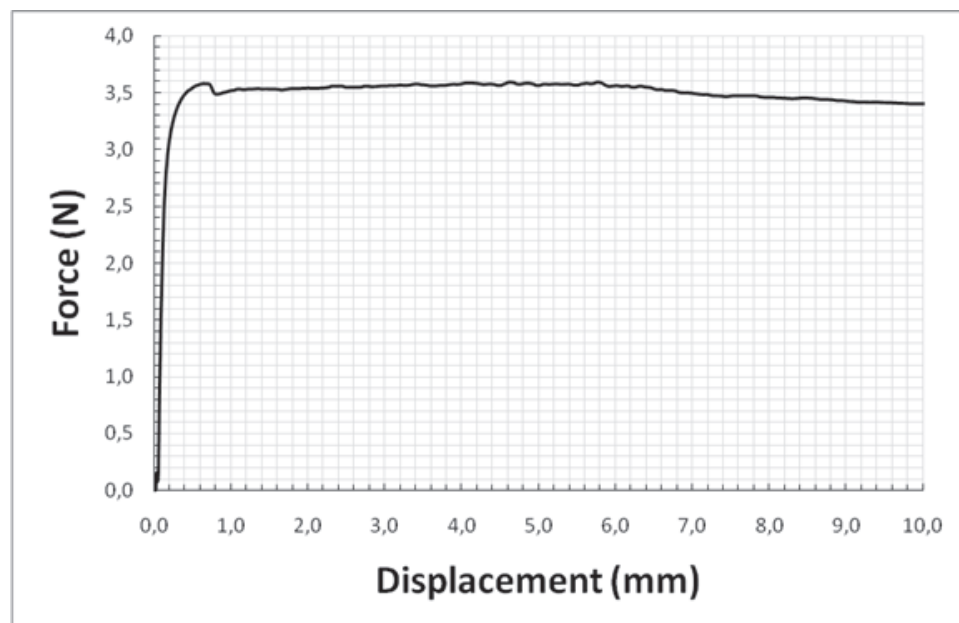
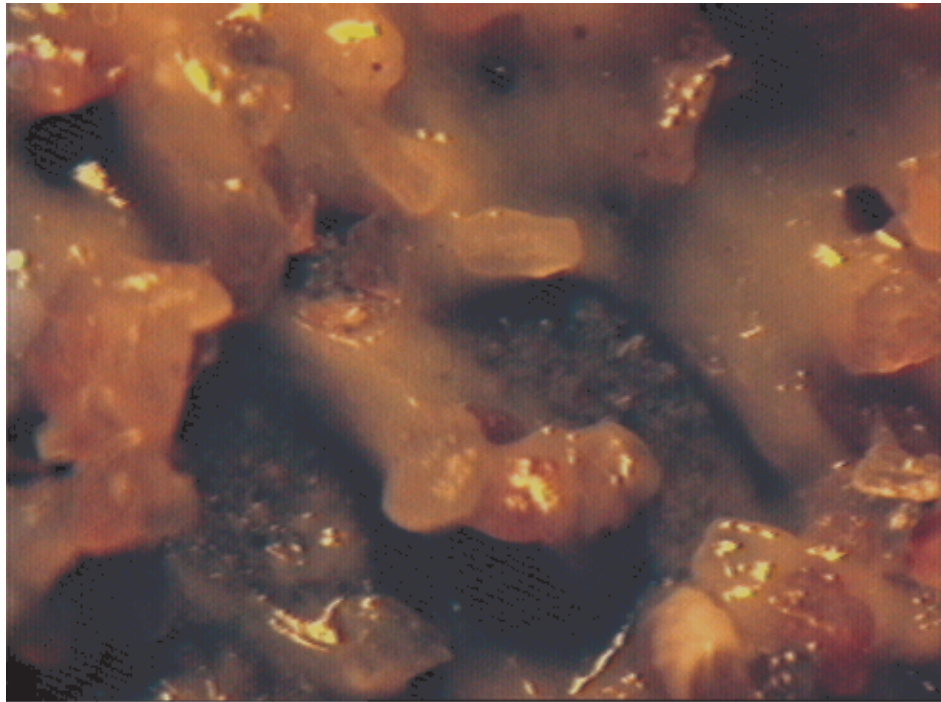
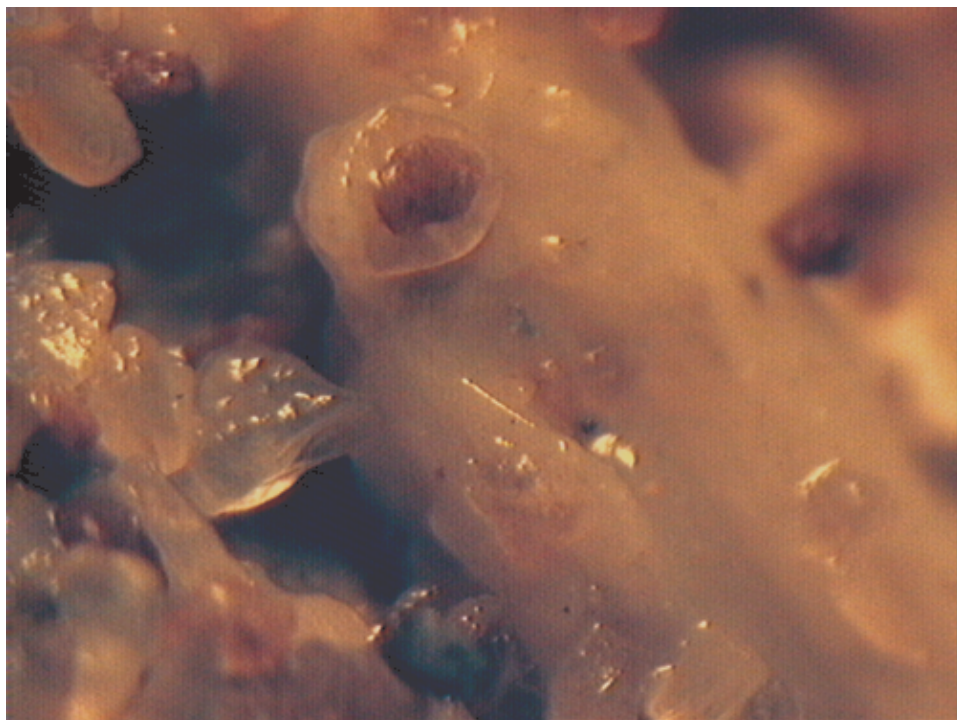


Figure 3.16. Test 3 fully pressurized injection application.



(a) Optical microscope photo of concrete after Test 3

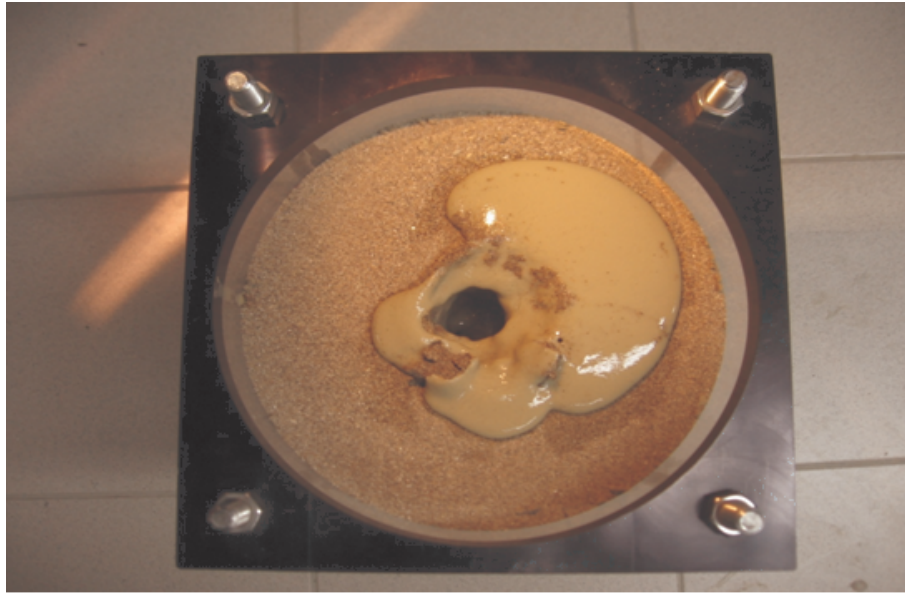


(b) Optical microscope photo of concrete after Test 3

Figure 3.17. The optical microscope photo of concrete after Test 2.

Even though fully bentonite pressurized Test 3 is retested with more than 5 cm

heights such as 10 cm and 20 cm height of bentonite over the surface of the sample, test results ($\mu_{int} = 0,06$) did not change. Also After the Test 3 it was observed that empty hole was not collapsed (Figure 3.18).



(a) Hole is not collapsed after Test 3



(b) Hole is not collapsed after Test 3

Figure 3.18. Hole is not collapsed after Test 3.

3.4. Summary of Laboratory Test Results

- (i) At bentonite injection tests it was observed that optimum concentration of bentonite in order to obtain a stable slurry is 6%. This means that 60 gram bentonite in 1 lt of water should be wait at least one day for hydration. For medium sand used in the first stage of lab tests, it can be said that filtration result is observed as surface filtration.
- (ii) According to Usegi et al. the influential factors of sand-concrete friction are concrete roughness and mean diameter of sand. Neither concrete strength nor sand type affects yielding (peak)friction coefficient. Hence, they defined a normalized roughness R_n which is equal to R_{max} divided by $(D_{50})_{mean}$ sand diameter. When R_n is rough enough (R_n bigger than 130) peak friction coefficient reaches shear stress ratio of sand and shear yield occurs within the sand mass not at the interface of sand -concrete.

As a result Usegi says that, the characteristics of sand-concrete friction is similar to those of the friction between sand and rough steel surfaces and the maximum coefficient of friction depends on concrete roughness and mean diameter of sand. the upper limit of maximum coefficient of friction is the maximum shear stress ratio of the sand dependent on the relative density [1].

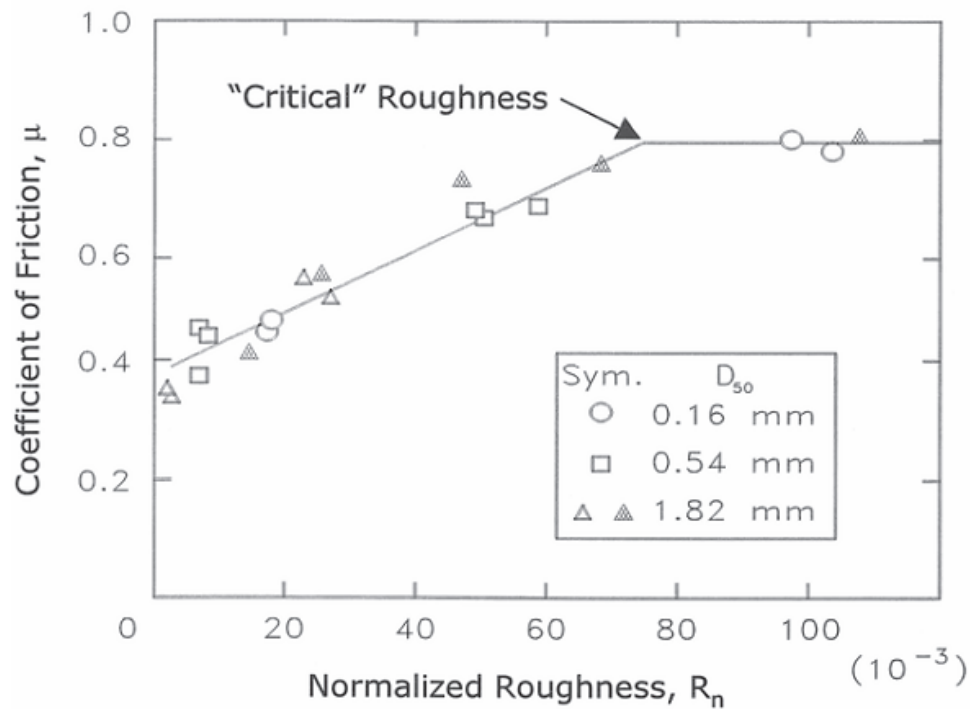


Figure 3.19. Normalized surface roughness vs. friction coefficient for sand-steel interfaces (Staheli, 2006).

The shear box results showed that even though bentonite slurry does not remain between pipe and soil the interface friction coefficient μ_{int} (μ_{int} between sand-bentonite contact zone and concrete = 0,46) decreased to half of the sand concrete interface friction (μ_{int} between sand and concrete = 0,90). If we comment on these results with the results of Usegi we can say that, sand-concrete friction without bentonite is similar to those of the friction between sand and rough steel surfaces. Hence, if we do not use bentonite, concrete sand interface does not occur between sand and concrete due to the roughness of concrete, but shear occurs within the sand mass. After bentonite injection, interface occurs between sand and concrete so that interface coefficient starts to decrease.

After the injection of bentonite slurry, jacking forces can decrease to half of the jacking forces at the case of no bentonite injection due to the decrease of surface roughness of the interface between soil and pipe such as decrease of μ_{int} from 0,8 to 0,4 which can be seen at Figure 3.19.

- (i) It is observed that at TEST 1, when no bentonite is used, interface coefficient of friction is 0,46. When controlled bentonite injection without constant injection pressure is applied, interface coefficient of friction μ_{int} decreased to 0,24 in TEST 2. As a result it can be said that, even though injection is not applied under a constant pressure e.g. controlled bentonite injection, after the application of bentonite injection, coefficient of friction forces can decrease to half of the the case of no bentonite injection.
- (ii) When bentonite is applied under constant pressure such as at Test 3, the interface friction coefficient μ_{int} is decreased from 0,46 to 0,06 which means there is not any interface between sand and concrete but bentonite and pipe. Hence, by applying continuous bentonite injection interface between bentonite and concrete pipe occurs and friction coefficient μ_{int} is reduced to approximately 10 percent of soil concrete pipe interface friction coefficient.
- (iii) Even though fully bentonite pressurized at Test 3 is retested with more than 5 cm heights such as 10cm and 20 cm height of bentonite over the surface of the sample, test results ($\mu_{int} = 0,06$) did not change. This means that high pressure injection is not required for lubrication, but injection pressures like contact injection pressures (between 1-2 bar) higher than water pressure may be adequate to keep overcut open.

4. FIELD CASE HISTORIES

4.1. Sile Project

4.1.1. Description of the Sile Project

A sewage project is started at Sile region in Istanbul by pipe jacking method. The internal diameter of the pipe is 800 mm and the external diameter is 1080 mm. A slurry machine which has 1100 mm diameter and 20 mm overcut is used in this project. Soil is low plastic clayey silt. The overburden depth is between 2,5 and 3 meters. The results of the jacking forces, not only with bentonite lubrication but also lubrication with bentonite plus polymer are measured.

4.1.2. Jacking Forces Between Shafts 24 and 21

This part of the pipeline is below the water table level. Total length is 155 meter and jacking is finished in 10 days. Bentonite is used as lubricant. During operation friction force is 1 t / m, because of the stoppages 12 hours at 101 meter, 16 hours at 132 meter and 12 hours at 146 meter jacking is finished with 35 tons extra jacking force.

Table 4.1. Soil parameters between PJ 24 and PJ 21.

	C_u (KN/m ²)	γ (kN/m ²)	W_n (%)	(LL)	(PI)	Clay-Silt Percentage	Soil Classification
Sample 1	23	19,5	28	31	9	21% Clay 79% Silt	Low plastic Clayey Silt
Sample 2	25	19	31	39	10	25% Clay 75% Silt	Low plastic Clayey Silt

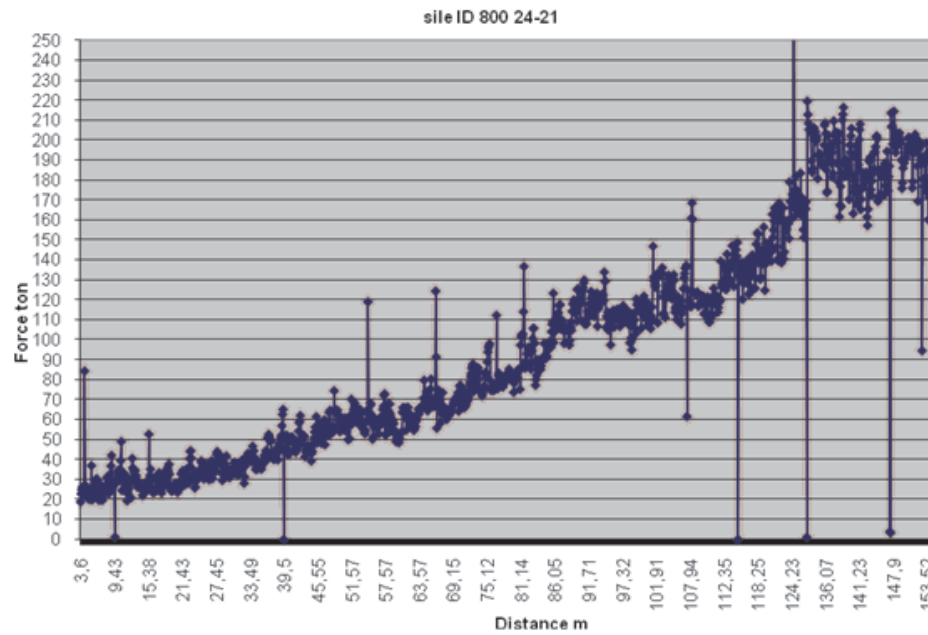


Figure 4.1. Sile collector project pipe jacking data between pits 24-21.

4.1.3. Jacking Forces Between Shafts 24 and 25

This part of the pipeline is below the water table level. Total length is 54 meter and jacking is finished in 6 days. Bentonite is not used as lubricant. During operation friction force is 1,5 t / m.

Table 4.2. Soil parameters between PJ 24 and PJ 25.

	C_u (KN/ m^2)	γ (kN/ m^2)	W_n (%)	(LL)	(PI)	Clay-Silt Percentage	Soil Classification
Sample 1	25	19,00	31,0	39	10,0	25% Clay 75% Silt	Low plastic Clayey Silt
Sample 2	48	19,50	27,0	37	10,0	18% Clay 82% Silt	Low plastic Clayey Silt

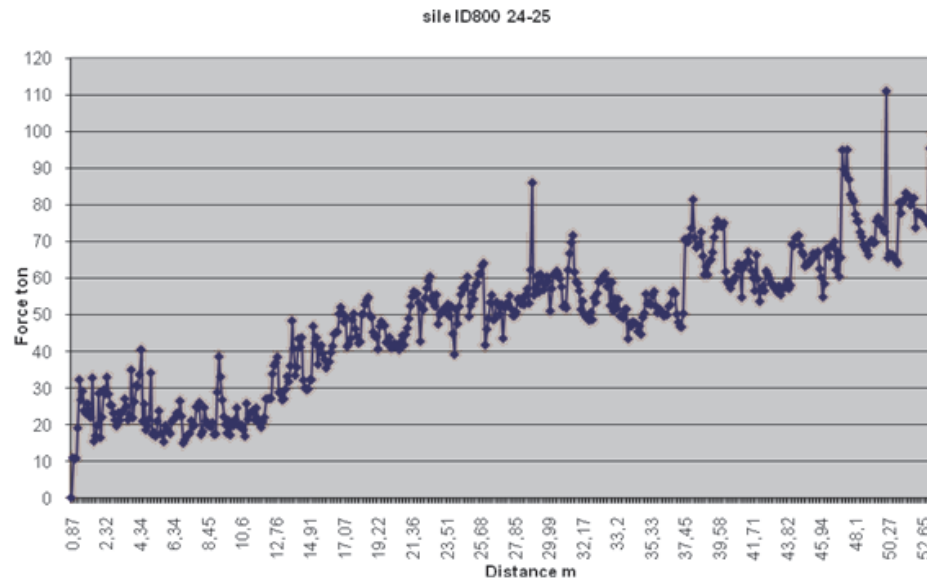


Figure 4.2. Sile collector project pipe jacking data between pits 24-25.

4.1.4. Jacking Forces Between Shafts 17 and 19

This part of the pipeline is above the water table level. Total length is 150 meter and jacking is finished in 17 days. Bentonite is used as lubricant. Until 1 day stoppage at 100 meter friction force is 0,7 t / m, but after this stoppage and the 16 hours stoppage at 109 meter, 9 hours at 132 meter and 14 hours at 141 meter and 9 hours stoppage at 149 meter jacking is finished with 55 tones extra jacking force.

Table 4.3. Soil parameters between PJ 17 and PJ 19.

	C_u (KN/m ²)	γ (kN/m ²)	W_n (%)	(LL)	(PI)	Clay-Silt Percentage	Soil Classification
Sample 1	50	20,6	21	33	9	29% Clay 71% Silt	Low plastic Clayey Silt
Sample 2	40	19,9	25	28	8	19% Clay 81% Silt	Low plastic Clayey Silt
Sample 3	14	19,7	27	30	8	17% Clay 83% Silt	Low plastic Clayey Silt

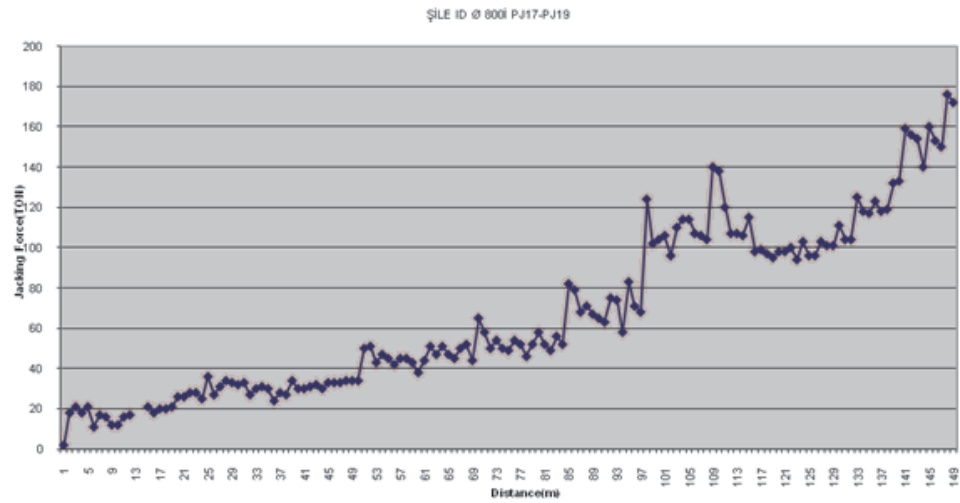


Figure 4.3. Sile collector project pipe jacking data between pits 17-19.

4.1.5. Jacking Forces Between Shafts 14 and 17

This part of the pipeline is above the water table level. Total length is 160 meter and jacking is finished in 28 days because of 15 days stoppage due to machine problem. Bentonite with polymer is used as lubricant. Friction force is calculated 1 t / m. After 15 days stoppage at 93 meter and 5 hours stoppage at 155 meter jacking is finished with 55 tons extra jacking force.

Table 4.4. Soil parameters between PJ 14 and PJ 17.

	C_u (KN/m^2)	$\gamma(\text{kN}/\text{m}^2)$	$W_n(\%)$	(LL)	(PI)	Clay-Silt Percentage	Soil Classification
Sample 1	21,5	19,9	26	29	8	20% Clay 80% Silt	Low plastic Clayey Silt
Sample 2	27	20,1	24	30	7	25% Clay 75% Silt	Low plastic Clayey Silt
Sample 3	50	20,6	21	33	9	29% Clay 71% Silt	Low plastic Clayey Silt



Figure 4.4. Collector project pipe jacking data between pits 14-17.

4.1.6. Conclusions

If we try to make comparison among the various drives in the same soil conditions, same overcut and same cover depth, bentonite lubrication can cause approximately 50% decrease on friction forces in low plastic clayey silt. When we make comparison between case where bentonite is used as an additive and not , it can be said that polymer is not so much helpful at lubrication, but it is useful at stoppages which is a very important problem at clayey soil due to the expansion of clay and cause locking of the jacking.

Table 4.5. Comparison of net friction forces between the case with and without bentonite.

Pit Number	Distance(m)	Net Friction Force(Ton)	Bentonite Condition
PJ 24-PJ 21	54	$60 - 20 = 40$	With Bentonite
PJ 24-PJ 25	54	$90 - 10 = 80$	No Bentonite
PJ 17-PJ 19	54	$65 - 20 = 45$	With Bentonite
PJ 14-PJ 17	54	$74 - 30 = 44$	With Bentonite and Polymer

As a result in clayey silt formations, against unpredictable stoppages, polymer

should be used with bentonite lubrication to keep clay expansion time long and to cause starting jacking forces decrease after the stoppages.

4.2. Göksu River Project

4.2.1. Description of Contracting Göksu River Sewage Collector Project

Göksu River Sewage Collector Project is a wastewater collecting project which aims to collect and transfer sewage to Kucuksu Wastewater Treatment Plant. The project is carried out beside the Istanbul Bosphours in alluvial sand below the water table. Depth of the cover is between 3–4 meters and 1400 mm inner diameter concrete pipes are used in this project. The effect of bentonite injection is observed in this project.

4.2.2. Jacking Forces Between Shafts 5 and 6

Between pits 5 – 6, no bentonite is used for lubrication and the 63 meters long drive was finished at 400 tons of total jacking force (Figure 4.5).

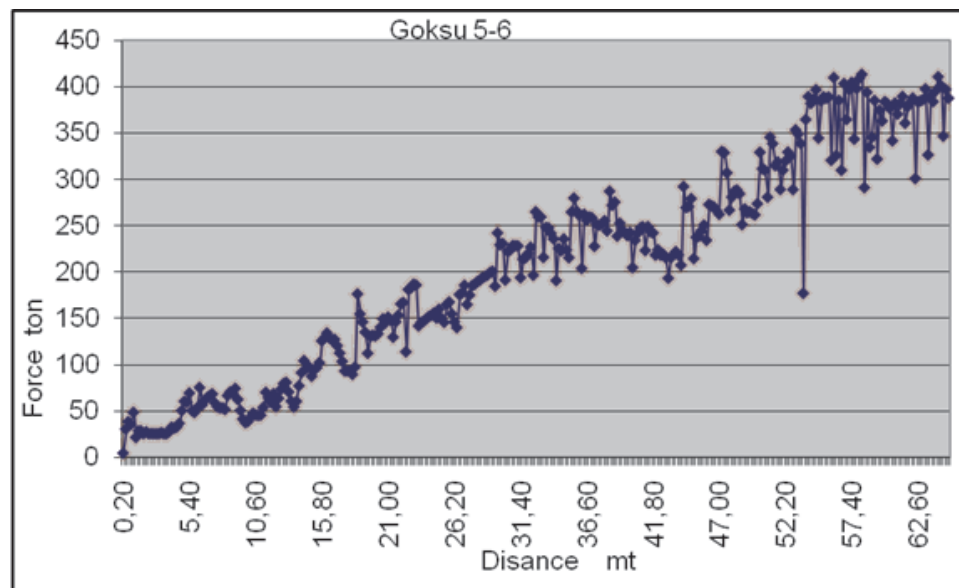


Figure 4.5. Göksu river sewage collector project pipe jacking data between pits 5-6.

4.2.3. Jacking Forces Between Shafts 6 and 8

In the second drive, between pits 6 – 8 (Figure 4.6), after 22 meters, bentonite injection started to be applied till the end of the drive. After the bentonite injection, 55 meters long drive was able to be finished at 200 tons of total jacking forces (Figure 4.6).

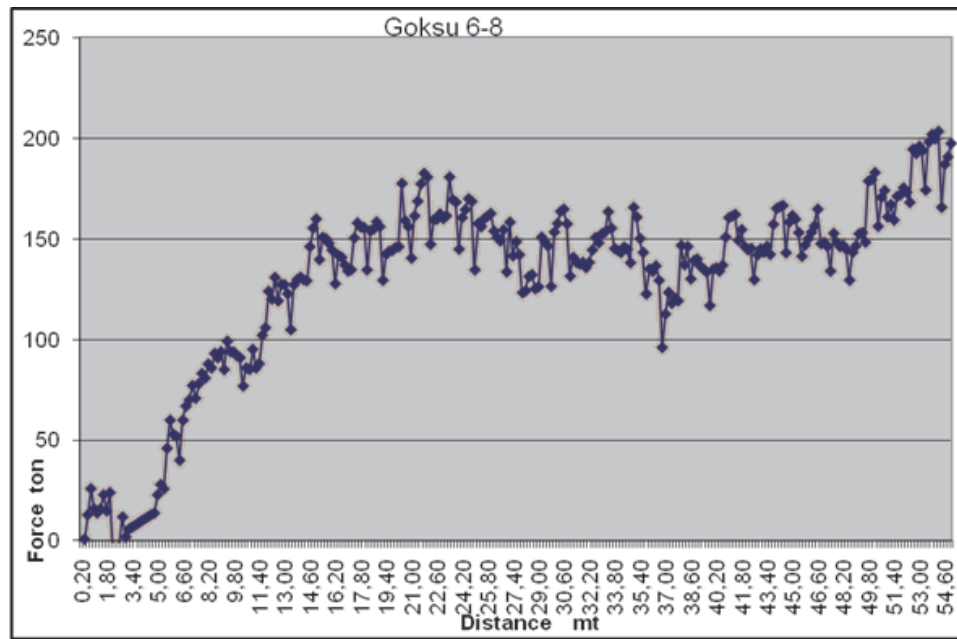


Figure 4.6. Göksu river sewage collector project pipe jacking data between pits 6-8.

4.2.4. Conclusions

As a result, it is observed that, in the case with bentonite injection application (Figure 4.6), the total jacking force is half of the total jacking force in the case without bentonite injection (Figure 4.5) at the end of the drives. This result means that after the application of bentonite injection, concrete - sand interface coefficient decreased to half of the case without bentonite.

4.3. Tuzla River Sewage Collector Project

4.3.1. Description of Tuzla River Sewage Collector Project

Tuzla River Sewage Collector Project is a wastewater collecting project which aims to collect the improperly discharged sewage from the Tuzla River and transfer it to the Tuzla Wastewater Treatment Plant. The length of the project is 5000 meters and 1400 mm inner diameter concrete pipes were used in this project. The duration of the project was written as 36 months in the contract but it was finished in 18 months after 6 months for machine production and delivery.

The soil condition is very dense sand and depth of cover is between 4.5 - 15 meters for the Tuzla River Sewage Collector Project. The effect of bentonite injection was observed in this project.

4.3.2. Jacking Forces Between Shafts 5 and 6

The total length between the 5 and 6 numbered pits was 144 meter and depth of cover was 8 meters. At the middle of the 5 - 6 drive when jacking forces increased suddenly, bentonite injection was started to be applied until the end of the drive (Figure 4.7, Tuzla 5 – 6). After the start of bentonite injection at meter 78, the jacking forces started to decrease. At the end of the drive, the total jacking force was recorded as 400 tons such as at the middle of drive. This result means that after the application of bentonite injection, the concrete - sand interface coefficient decreased to half of the case without bentonite.

In this drive effect of late bentonite injection is observed. Average daily progress was 25 meters and 5 tons of bentonite was used. As a result, it can be said that although bentonite injection is started at the middle of the drive, it caused 50% reduction of jacking forces.

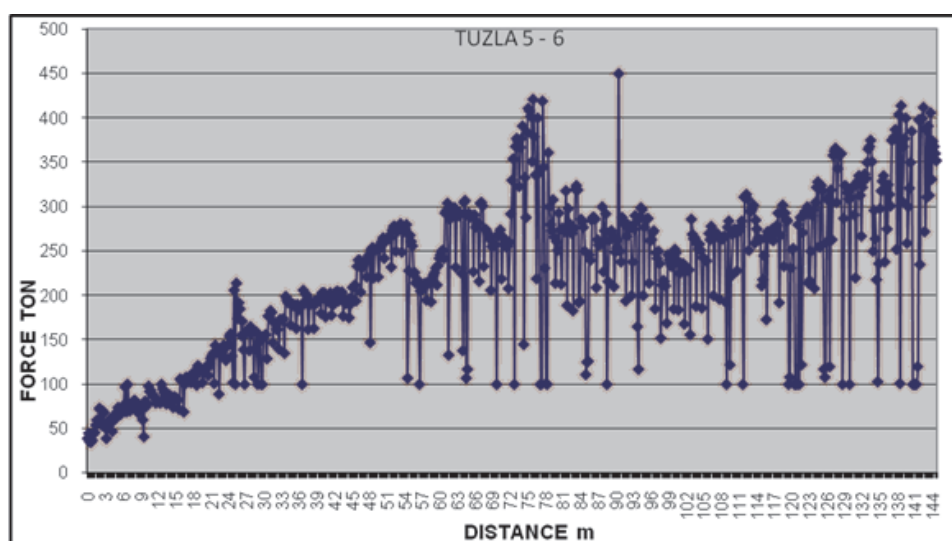


Figure 4.7. Tuzla River Sewage Collector Project Pipe jacking data between pits 5-6.

4.3.3. Jacking Forces Between Shafts 7 and 8

Effect of in time bentonite injection is observed in Tuzla Project between pits 7 – 8. Bentonite injection application is started from the beginning and applied until the end of the drive. Depth of the cover was 5 m and soil is dense sand. Average daily progress was 20 meters and 4 tons of bentonite was used during the drive (Figure 4.8).

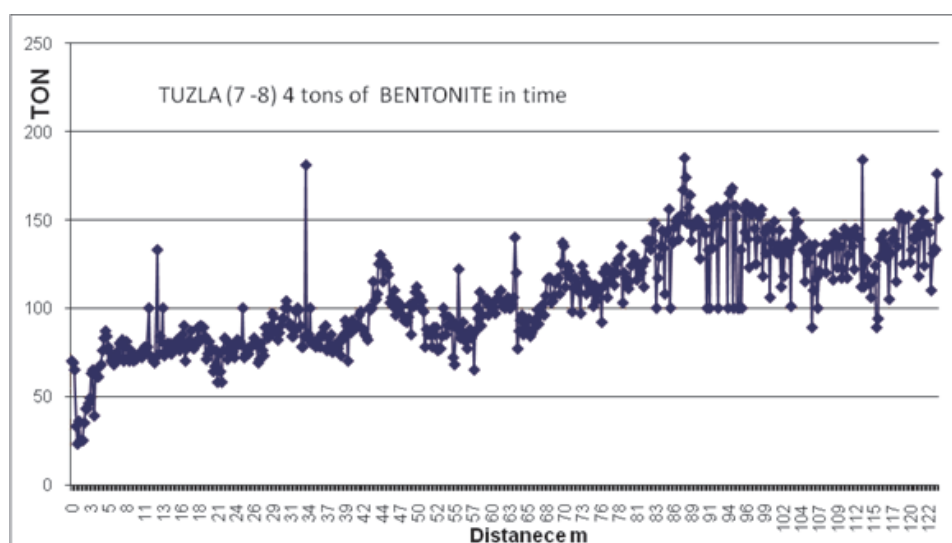


Figure 4.8. Tuzla river sewage collector project pipe jacking data between pits 7-8.

4.3.4. Conclusions

It is observed in Tuzla Project that, in the case with bentonite injection application, the total jacking force is half of the total jacking force at the end of the drive between pits 5 – 6 (Figure 4.7). This result means that after the application of bentonite injection, concrete - sand interface coefficient decreased to half of the case in which no bentonite is used.

One of the most important data obtained from Tuzla project is that, at drive between pits 7 – 8 interface coefficient of friction μ_{int} is measured as 0,05. This result is 10% of the case without bentonite. In other words, it can be said that between pits 7 – 8, overcut was not collapsed and interface was occurred between bentonite and pipe. As an important result it can be said that, if bentonite injection is able to be applied in sync with excavation constantly, 10 times longer drives can be achieved in sand due to the meaningful decrease (10%) of the interface friction coefficient. Hence, in order to keep overcut open and cause smooth interface between concrete pipe and sandy soil in time bentonite injection should be started from the beginning and applied continuously in sync with excavation is required to reach ten time longer drives.

In Tuzla Project at drive between 5 – 6, average daily progress was 25 meters and 5 tons of bentonite is used, but at drive between pits 7 – 8, average daily progress was 20 meters and 4 tons of bentonite was used. Hence, it can be concluded that, in pipe jacking projects, in order to keep overcut open, bentonite injection should not be applied slower than excavation speed. Due to collapse of overcut, even though much more bentonite was injected, late bentonite injection could not be as useful as in time bentonite injection.

In Tuzla Project, by reducing pipe jacking forces with bentonite injection, a 233 meter long pipe jacking drive was achieved without any need to use an interjack station. By making longer drives and reducing the number of jacking pits, the duration of the project is reduced meaningfully. All of these advantages allowed finishing the project in 24 months instead of 36 months with 30% less cost. In this project, the maximum

daily progress and average daily progress were recorded as 52 meter and 21 meter respectively.

4.4. Comparison of Cost Between Cases with and Without Bentonite

According to Akkaya Baris [19], percentage of pipe cost in total cost is 17%, machine working cost in total cost is 34%, man and material cost in total cost is 21% and construction of pit cost in total cost is 20%. Cost comparison of a 1000 meters length pipe jacking project between cases with and without bentonite is calculated below. For both cases daily excavation progress is accepted 10 meters, and mobilization time from one pit to another is accepted 10 days.

(i) Case 1

- For the case without bentonite average jacking distance may be 125 m, this means that at least 9 pits are needed with 7 times of mobilization from one to another.
- Time needed for excavation= $\frac{1000m}{10m/day} = 100$ days
- Time needed for mobilization between pits= 7×10 days= 70 days
- Total time= $100+70 = 170$ days.

(ii) Case 2

- For the case with bentonite average jacking distance may be 500 meters with 3 pits, with 1 times of mobilization from one to another.
- Time needed for excavation= $\frac{1000m}{10m/day} = 100$ days
- Time needed for mobilization between pits= 1×10 days = 10 days
- Total time= $100 + 10 = 110$ days.
- Decrease of cost between case 1 and case 2.
- Cost of pit= $\frac{9-3}{9} = 0,67$ $0,67 \times 20\% = 13\%$
- Cost of man and material = $170 - 110 = 60$ $60 \times 21\% = 12,6\%$
- Total decrease at cost = $13\% + 12,6\% = 25,6\%$

As can be seen from the total cost reduction calculation above, as a result of making longer drives and reducing the number of jacking pits by bentonite slurry injection,

the duration and cost of the project is reduced meaningfully.

4.5. Summary

Several pipe jacking applications in Istanbul are mentioned in this chapter. Sile Project, Göksu River Sewage Collector Project and Tuzla River Sewage Projects were summarized. 800 mm and 1400 mm inner diameter concrete pipes were used in these projects. Soil conditions were low plastic clayey silt, alluvial sand, very dense sand. Depths of the covers were changing between 3 and 15 meters. The effect of bentonite injection on jacking forces was observed in these projects.

In Sile project it can be said; in the same soil conditions, same overcut and same depth of cover, bentonite lubrication can cause approximately 50% decrease on friction forces in low plastic clayey silt. Although polymer is not so much helpful for lubrication, it is useful at stoppages which is a very important problem at clayey soil due to the expansion of clay and cause locking of the jacking. As a result in clay formations against unpredictable stoppages, polymer should be used with bentonite lubrication to keep clay expansion time long and to cause starting jacking forces decrease after the stoppages.

The Göksu River Sewage Collector Project was carried out adjacent to the Istanbul Bosphorus in alluvial sand below the water table. The depth of the cover was between 3 – 4 meters. In different drives with same drive length and cover depth it is observed that, in the case with bentonite injection application, the total jacking force is half of the total jacking force in the case without bentonite injection at the end of the drives.

In the Tuzla River Sewage Collector Project, the soil is very dense sand and depth of cover is between 4,5 and 15 meters. At the middle of drive between pits 5 – 6 when jacking forces rose suddenly, bentonite injection was started to be applied until the end of the drive. At the end of the drive, the total jacking force was measured as same as the force at middle of the drive.

All in all, in the Tuzla Project, by reducing pipe jacking forces with bentonite injection, a 233 meter long pipe jacking drive was achieved without any need to use an interjack station. By making longer drives and reducing the number of jacking pits, the duration of the project is reduced meaningfully. All of these advantages allowed finishing the project in 24 months instead of 36 months with 30% less cost. In this project, the maximum daily progress and average daily progress were recorded as 52 meters and 21 meters respectively.

As a result, it can be said that bentonite injection application in sandy soils can result in 50% reduction of jacking forces. In pipe jacking projects, in order to keep overcut open, bentonite injection should not be applied slower than the excavation(progress) speed. Even though much more bentonite injected, due to collapse of overcut, late bentonite injection could not be as useful as in time bentonite injection.

Bentonite injection which starts just after the excavation gives opportunity to make longer drives and to reduce the number of jacking pits, so that the duration of the projects are reduced meaningfully. Also it can be said that, if bentonite injection is able to be applied in sync with excavation constantly, 10 times longer drives can be achieved in sand due to the meaningful decrease (87%) of the interface coefficient of friction. Hence, in order to keep overcut open and cause smooth interface between concrete pipe and sandy soil in time bentonite injection should be started from the beginning and applied continuously in sync with excavation is required to reach ten time longer drives. All of these advantages allow finishing pipe jacking projects in a shorter time with less cost.

5. EVALUATION OF JOBSITE RESULTS WITH LABORATORY TEST RESULTS

5.1. Introduction

In this chapter, interface coefficient of friction is calculated from the data which are obtained from laboratory tests and jobsite pipe jacking cases. Effect of bentonite injection on interface coefficient of friction in both laboratory tests and jobsite cases are evaluated. Laboratory test results are compared with the case results which are tried to be simulated.

5.2. Calculation of Coefficient of Friction at the Jobsite

Vertical stress on the pipe is calculated according to Terzaghi's trap door model. Coefficient of reduction is taken from Figure 2.13 and meaning of symbols are;

- h = Depth of cover
- κ = Coefficient of reduction
- d_e = Pipe external diameter
- F_{net} = Net frictional part of jacking force

5.2.1. CASE 1, Göksu River Sewage Collector Project

Göksu River Sewage Collector Project is carried out beside the Istanbul Bosphorus in alluvial sand below the water table, and depth of the cover is 3 m. At CASE1, between pits 5 – 6 (Figure 5.3), no bentonite is used for lubrication and the 63 meter long drive was finished at 400 tons of total jacking forces and interface coefficient of friction μ_{int} is calculated as 0,50. Göksu River between pits 5 – 6:

- Distance between 0 and 57 meters, $L = 57$ meters
- $h = 3$ $d_e = 1,7$ m $h/d_e = 2 \rightarrow$ form Figure 2.32 $\kappa = 0,5$

- $\gamma = 1.8 \text{ t/m}^2$ $\sigma_\nu = \kappa\gamma h = 0,5 \cdot 1,8 \cdot 3 = 2,4 \text{ t/m}^2$
- $F_{\text{net}} = 397 - 27 = 370 \text{ ton}$
- $\tau = \frac{F_{\text{net}}}{\pi d_e L} = \frac{370}{3,141,757} = 1,21 \text{ t/m}^2$
- $\mu_{\text{int}} = \frac{\tau}{\sigma_\nu} = \frac{1,21}{2,4} = 0,5$

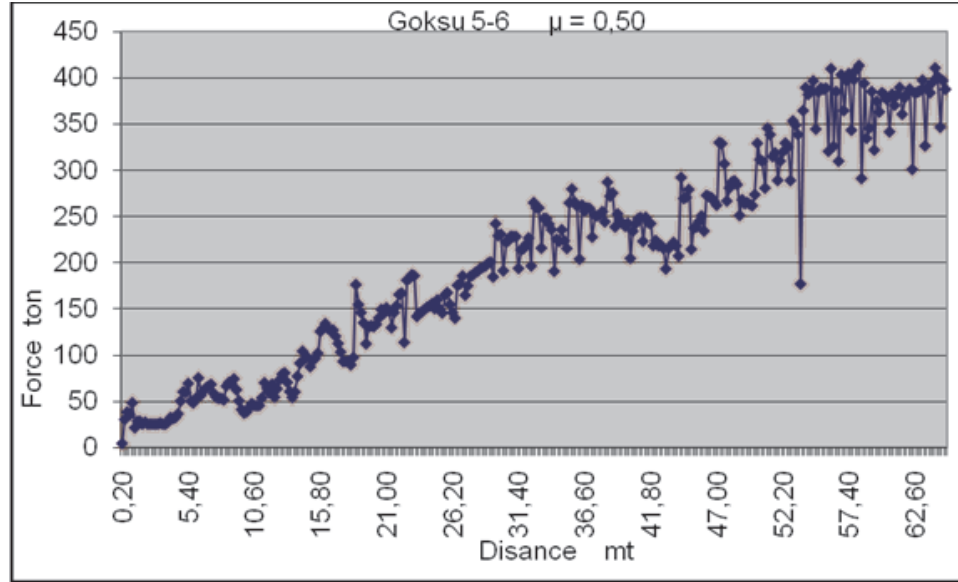


Figure 5.1. Göksu river project jacking forces between pits 5-6 without bentonite.

5.2.2. CASE 2, Göksu River Sewage Collector Project

In the second drive of Göksu River Sewage Collector Project, between pits 6 – 8 (Figure 5.4), after 22 meters, bentonite injection started to be applied till the end of the drive. After bentonite injection, coefficient of friction μ_{int} is 0,5 reduced to 0,25 and 55 meters long drive was finished at 200 tons of total jacking forces.

Göksu River between pits 6 – 8: Before bentonite injection

- Distance between 0 and 22 meters, $L = 22 \text{ meters}$
- $h = 3 \text{ m}$ $d_e = 1,7$ $h/d_e = 2 \rightarrow$ from Figure 2.32 $\kappa = 0,5$
- $\gamma = 1.8 \text{ t/m}^2$ $\sigma_\nu = \kappa\gamma h = 0,51,8 \cdot 3 = 2,4 \text{ t/m}^2$
- $F_{\text{net}} = 182 - 26 = 156 \text{ ton}$
- $\tau = \frac{F_{\text{net}}}{\pi d_e L} = \frac{156}{3,14 \cdot 1,7 \cdot 22} = 1,27 \text{ t/m}^2$
- $\mu_{\text{int}} = \frac{\tau}{\sigma_\nu} = \frac{1,27}{2,4} = 0,5$.

Göksu River between pits 6-8 for last 20 meters. After bentonite injection;

- $L = 20$ meters, $= 1.8 \text{ t/m}^2$
- $F_{\text{net}} = 200 - 135 = 65$ tons
- $\sigma_v = 2.4 \text{ t/m}^2$
- $\tau = \frac{F_{\text{net}}}{\pi d e L} = \frac{65}{3.143 \cdot 1.417} = 0.61 \text{ t/m}^2$
- $\mu_{\text{int}} = \frac{\tau}{\sigma_v} = \frac{0.61}{2.4} = 0.25$.

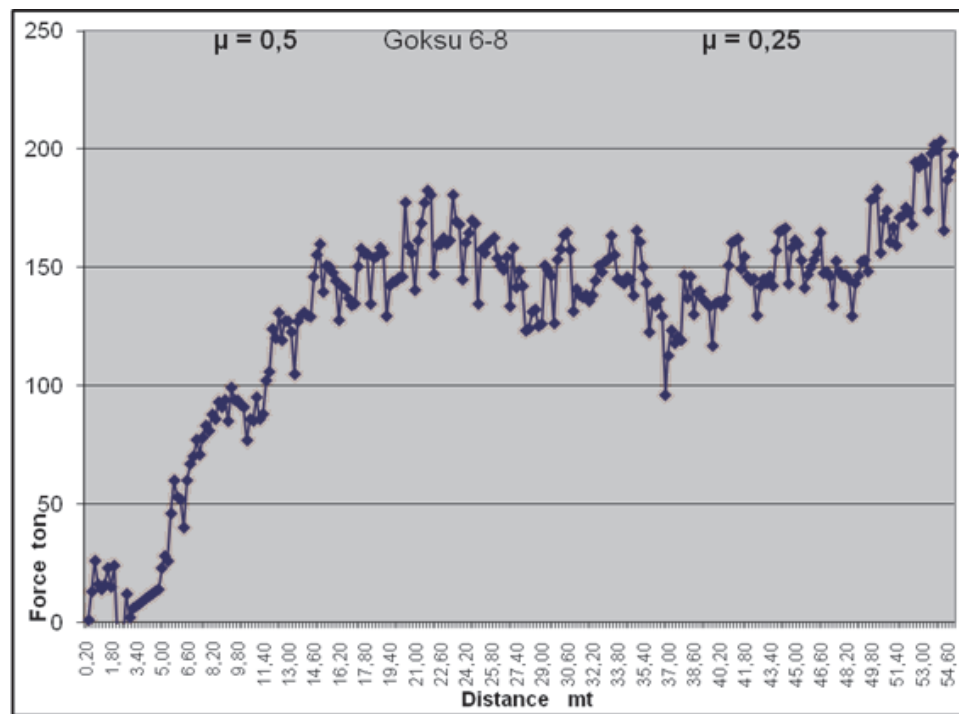


Figure 5.2. Göksu River Project jacking forces between pits 6-8 after bentonite injection.

5.2.3. CASE 3, Tuzla river sewage collector project

The Tuzla River Sewage Collector Project is carried above water table, and soil condition is very dense sand. Effect of in time bentonite injection is observed in Tuzla Project between pits 7 – 8. Bentonite injection application is started from the beginning and applied until the end of the drive. Depth of the cover is 5 m and soil is dense sand. Average daily progress is 20 meters. 4 tons of bentonite used and interface coefficient of friction μ_{int} is measured as 0,05 (Figure 5.5).

- Tuzla River between pits 7-8:
- $\gamma = 1.8 \text{ t/m}^2$ L= 122 meters h=5 meters
- $d_e = 1,7 \text{ meters}$ h / $d_e = 2,9 \rightarrow$ from Figure 2.32 = 0,4
- $\sigma_\nu = \kappa\gamma h = 0,41,85 = 3,6 \text{ t/m}^2$
- F net = 143-23= 120 ton
- $\tau = \frac{F_{net}}{\pi d_e L} = \frac{120}{3,141,7122} = 0,19 \text{ t/m}^2$
- $\mu_{int} = \frac{\tau}{\sigma_\nu} = \frac{0,19}{3,6} = 0,05.$

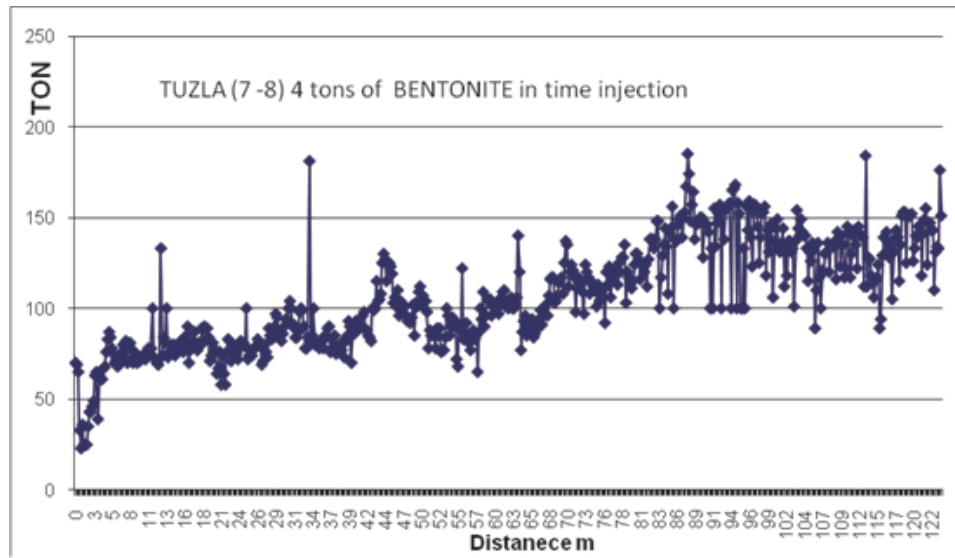


Figure 5.3. Tuzla river project jacking forces between pits 7-8 with bentonite injection.

In this drive, 4 tones of bentonite was used from the beginning until the end of drive constantly. As a result, coefficient of friction decrease approximately 10% of the case without bentonite.

5.3. Calculation of Coefficient of Friction at the Laboratory Tests

- W = weight of concrete pipe sample =29 gr = 2,9 N
- $\gamma_{sand} = 1,72 \text{ gr/cm}_3$
- $\phi_{sand} = 36^\circ$
- D_c = diameter of concrete sample = 2 cm L_c =Length of concrete sample = 20 cm

- μ_{int} = coefficient of friction
- $\mu_{int} = \frac{\tau}{\sigma}$
- $K_0 = 1 - \sin \phi = 0,41$
- $\sigma = \gamma(L_c/2)K_0$
- $\tau = (F_{Tensile} - W) / (L_c \pi D_c)$
- $\sigma = 1,7 \cdot 2 \cdot 10 \cdot 0,41 = 7,052 \text{ gr/cm}^2$

5.3.1. Test 1: No Bentonite

- $F_{tensile} = 7 \text{ N}$, $W = 2,9 \text{ N}$, $F_{net} = F_t - W$
- $F_{net} = 4,1 \text{ N} = 0,41 \text{ kgf}$
- $\tau = \frac{0,41}{2023,14} = 3,2610^3 \text{ kg/cm}^2$
- $\mu_{int} = \frac{\tau}{\sigma} = \frac{3,26}{7,052} = 0,46$
-

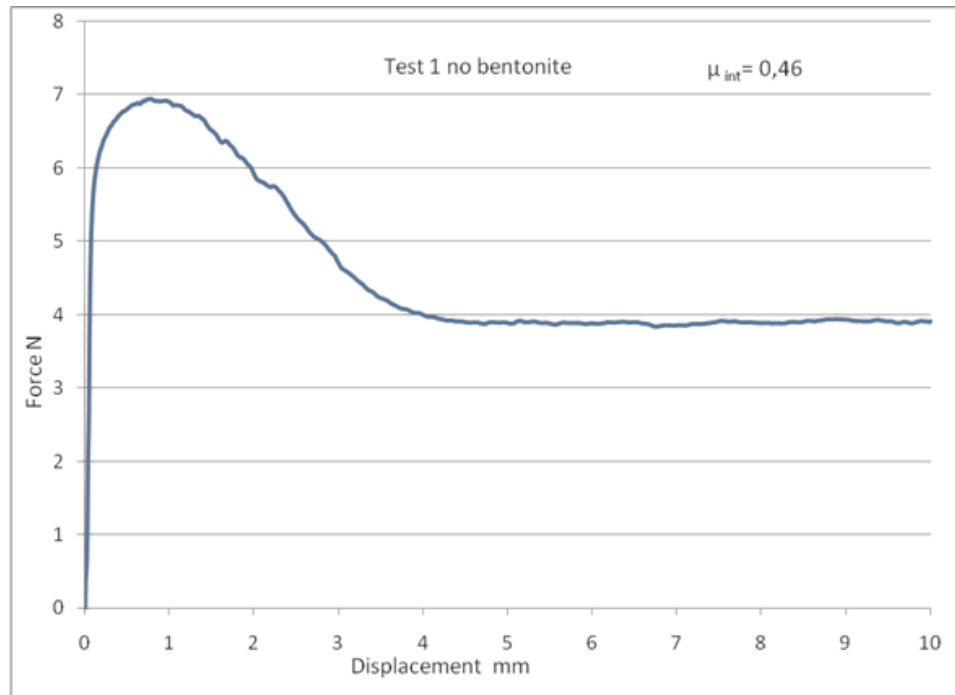


Figure 5.4. Test 1, dense sand and concrete interface friction force without bentonite.

5.3.2. Test 2: Bentonite Without Constant Pressure

- $F_{net} = 5,4 - 2,9 = 2,5 \text{ N} = 0,25 \text{ kg}$

- $\sigma = 1,72 \cdot 10,41 = 7,052 \text{ gr/cm}^2$
- $\tau = \frac{0,25}{2023,14} = 2 \cdot 10^{-3} \text{ kg/cm}^2$
- $\mu_{int} = \frac{2 \cdot 10^{-3}}{7 \cdot 10^{-3}} = 0,28$

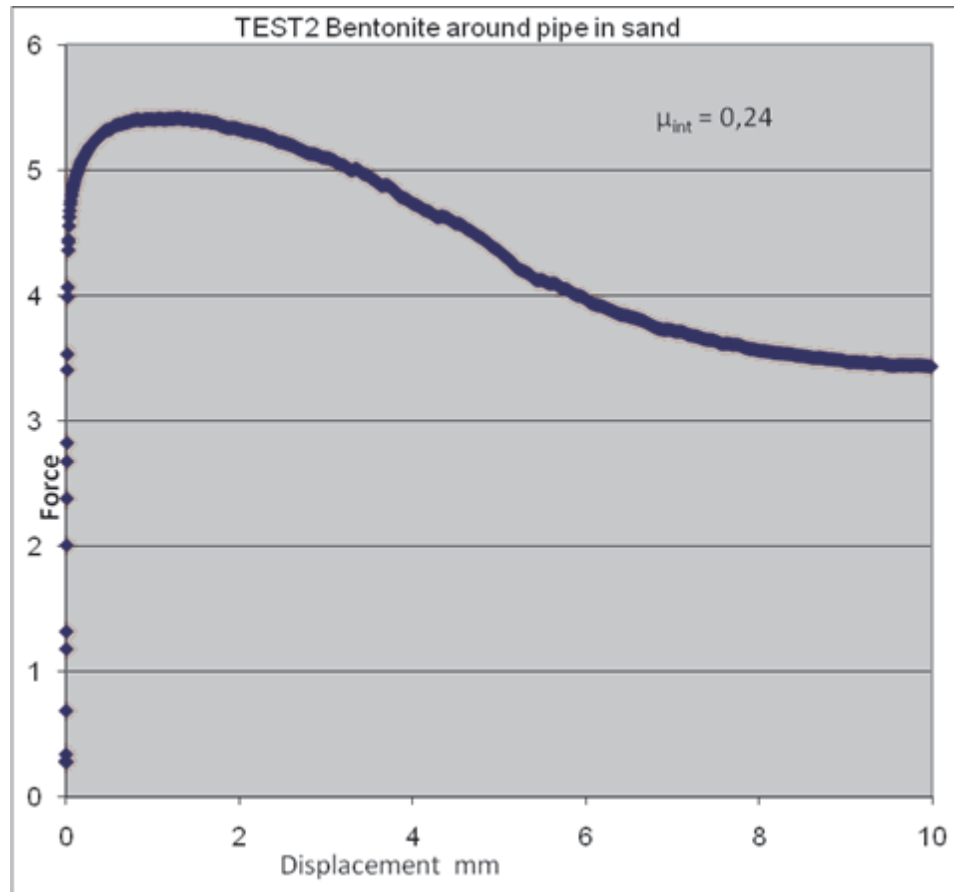


Figure 5.5. Test 2 Inadequate bentonite injection without constant injection pressure.

5.3.3. Fully Pressured Bentonite Injection

- $F_{tensile} = 0,35 \text{ kgf}$ $F_{net} = 0,35 - 0,29 = 0,06 \text{ kgf}$
- $\tau = \frac{0,06}{2023,14} = 4,70 \cdot 10^{-4} \text{ kg/cm}^2$
- $\mu_{int} = \frac{\tau}{\sigma} = \frac{4,710^{-4}}{710^{-3}} = 0,06$

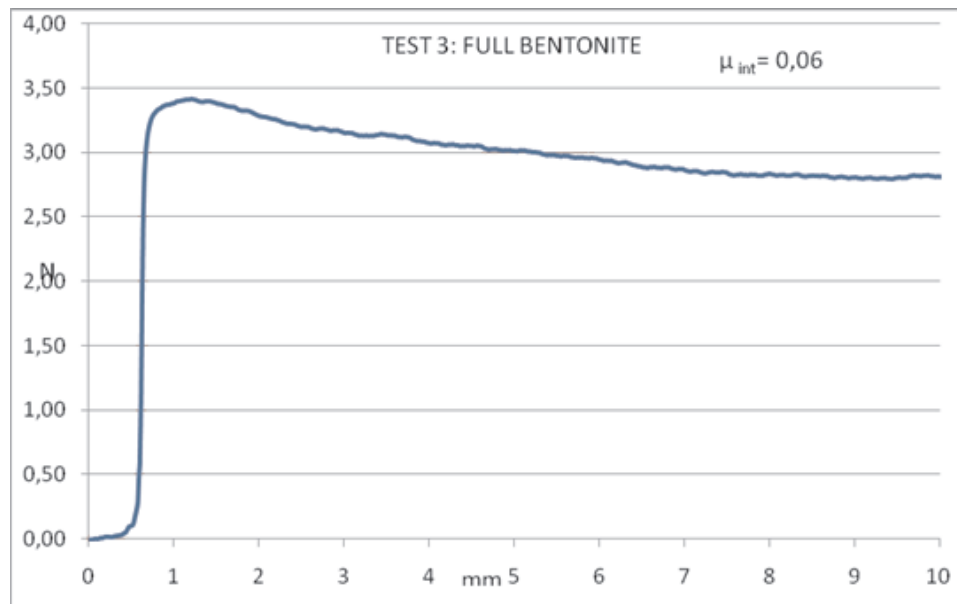


Figure 5.6. Test 3, fully pressured injection application.

5.4. Comparison of Cases With Laboratory Tests

The similarities and differences between laboratory tests and job site cases from view point of the change of the coefficient of friction after bentonite injection will be mentioned in this subtitle.

5.4.1. Comparison of Test 1 and Case 1

Without bentonite at TEST 1 coefficient of friction μ_{int} is 0,46 and at Case 1 coefficient of friction μ_{int} , is 0,50.

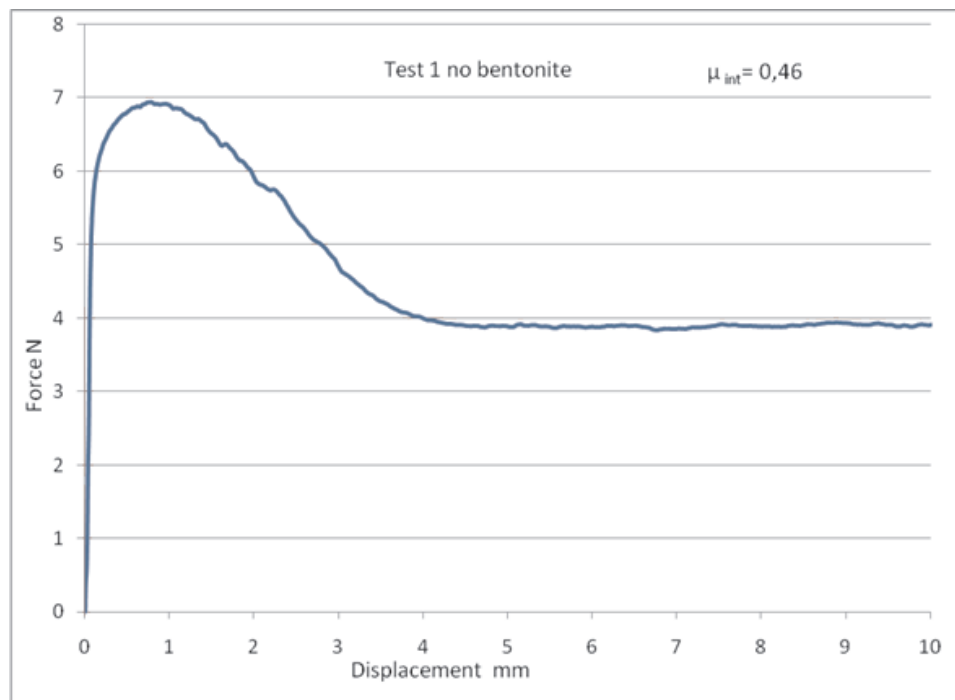


Figure 5.7. Test 1, dense sand and concrete interface friction force without bentonite.

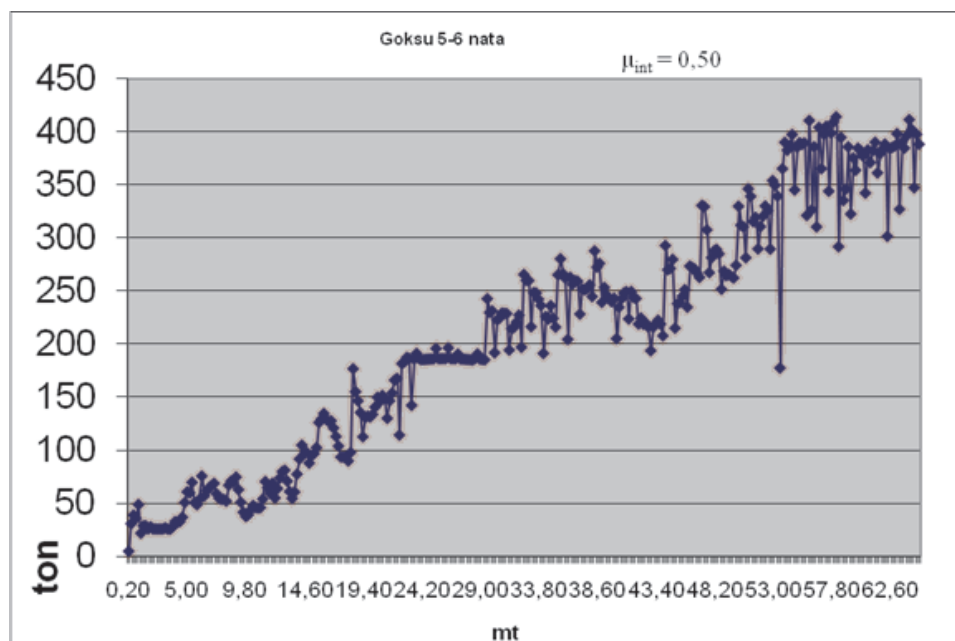


Figure 5.8. Case 1, Göksu river project jacking forces between pits 5-6 without bentonite.

5.4.2. Comparison of Test 2 and Case 2

Bentonite injection without constant pressure that models controlled bentonite injection, coefficient of friction at TEST 2 is 0,24 and coefficient of friction at CASE 2 is 0,25. When bentonite injection is applied at Test 2, the interface coefficient of friction is decreased from 0,46 to 0,24 such at the jobsite Case 2. After the bentonite injection in sand at Case 2, interface coefficient of friction μ_{int} , reduced from 0,5 to 0,25.

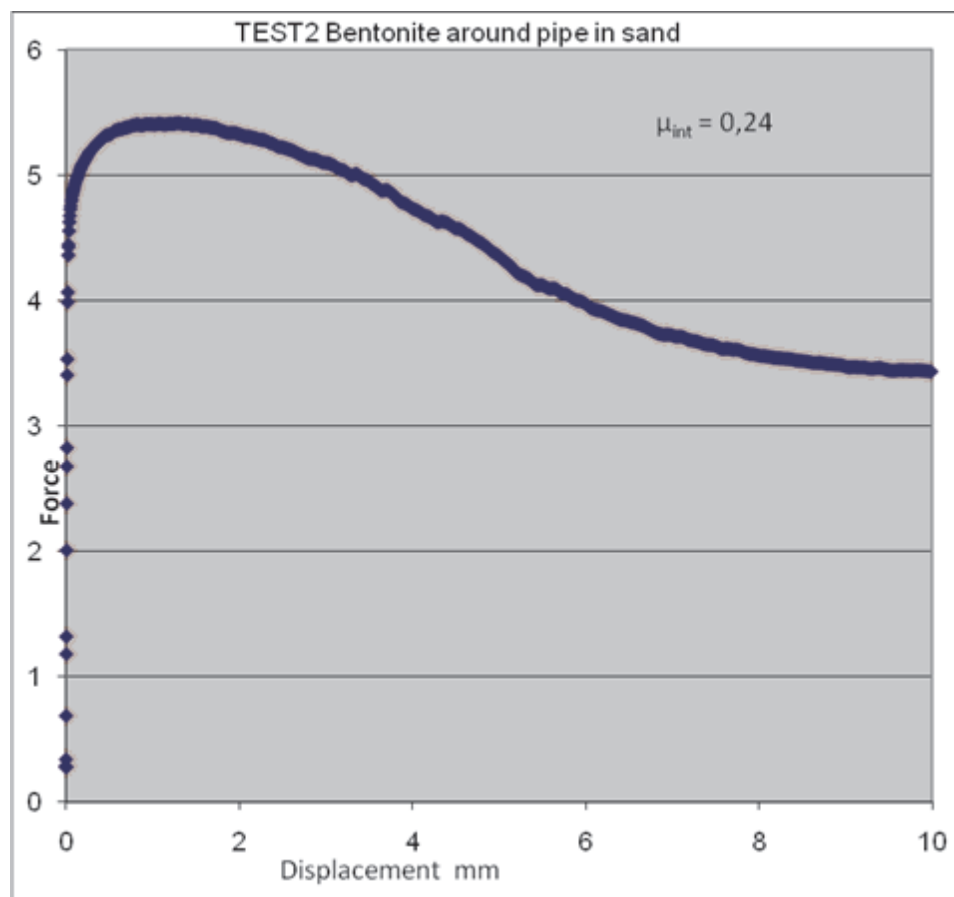


Figure 5.9. Test 2, Inadequate bentonite injection without constant injection pressure.

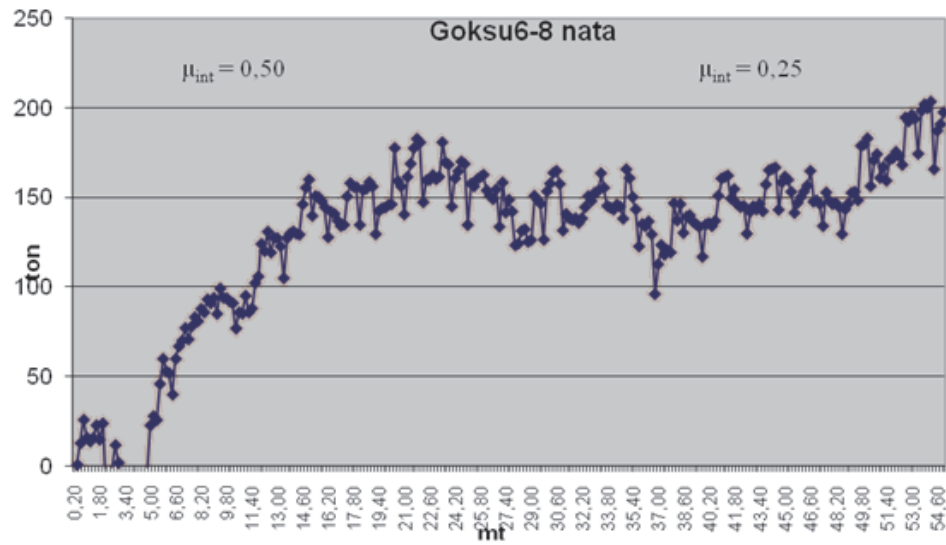


Figure 5.10. Case 2, Göksu project jacking forces between pits 6-8 after bentonite injection.

5.4.3. Comparison of Test 3 and Case 3

Bentonite injection under constant pressure, coefficient of friction at TEST 3 is 0,06 and coefficient of friction at CASE 3 is 0,05. When continuous bentonite injection is applied under constant pressure such as at Test 3, the interface coefficient of friction is decreased from 0,46 to 0,06 which means there becomes no interface between sand and concrete but there occurs interface between bentonite and pipe such as observed in Case 3, coefficient of friction is decreased from 0.5 to 0,05.

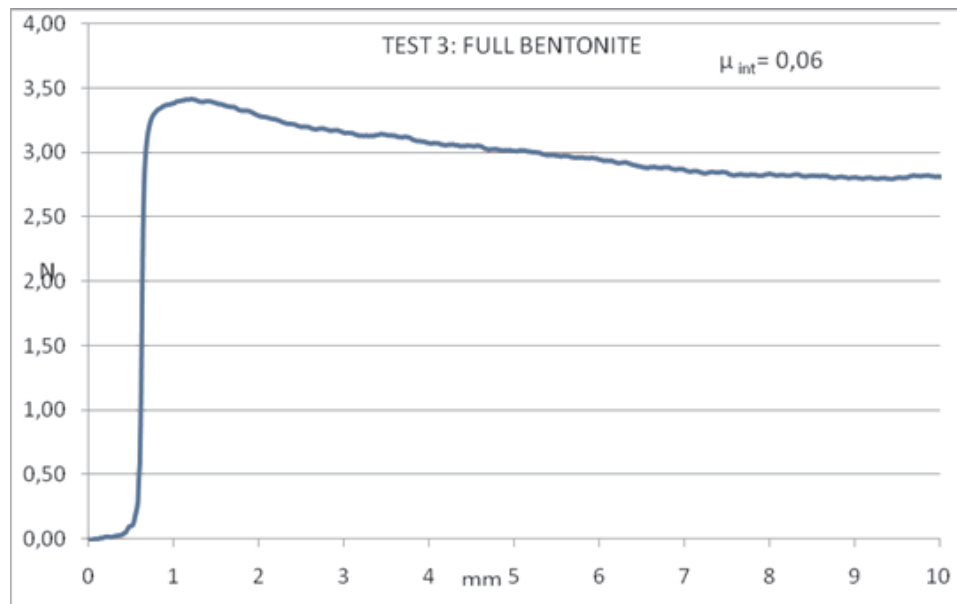


Figure 5.11. Test 3, fully pressured injection application.

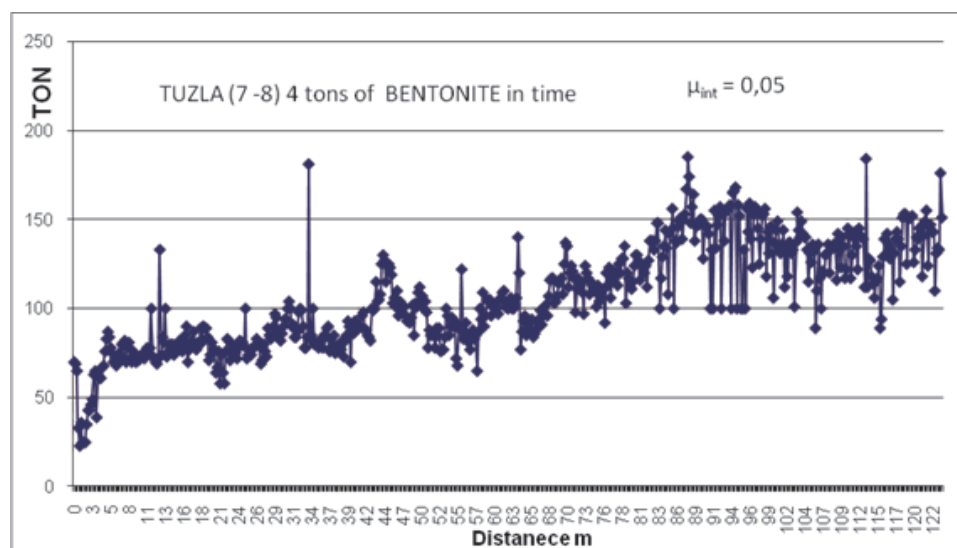


Figure 5.12. Case 3, Tuzla project jacking forces between pits 7-8 with bentonite injection.

5.5. Summary

- (i) When pipe jacking work is in sand high frictional loads can develop which limit jacked drive length. Even though injection system is not adequate to make injection constantly, in order to prevent high frictional loads, bentonite injection should be used in sand. But it should be keep in mind that in late and

unpressured constant injection, friction coefficient can be reduced down to the half of the beginning case before bentonite injection.

- (ii) Permanent bentonite injection which is applied constantly in sync with excavation can cause 90% percent decrease of interface friction coefficient. Because, in steady injection case, interface is able to occur just between pipe and bentonite under the soil.
- (iii) As a result , bentonite injection which starts just after the excavation can cause at least two times longer drives. Hence, not only number of jacking pits decrease to half but also mobilization time will decrease meaningfully. All of these parameters will help very important cost and time decreases in pipe jacking projects.
- (iv) Eventhough fully bentonite pressured Test 3 is retested with more than 5 cm heights such as 10 cm and 20 cm height of bentonite over the surface of the sample, test results ($\mu_{int} = 0,06$) did not change. Case 3 ($\mu_{int} = 0,05$) was not also done under high pressure.
- (v) All in all, in order to keep overcut open, there is no need for high pressures, injection pressures like contact injection pressures higher than water pressure may be adequate.
- (vi) During pipe jacking operation, stoppage can cause blockage of the operation. To prevent sudden rising of friction forces due to the blockage at the stoppages, pressurized bentonite must be grouted into overcut simultaneously with TBM advance in order to obtain an interface between pipe and soil.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

During pipe jacking, in order to keep the jacking force as low as possible, frictional force which is the main part of the jacking force should be kept low. In order to keep the frictional jacking force as low as possible appropriate measures such as cutting of an annular gap combined with the injection of lubricants into this annular gap is essential. According to soil type the function of injection changes. Depending on the density of cohesionless soils and consistency of cohesive soils, bentonite injection either provides stabilization of the surrounding ground or lubrication of the pipe string. If cohesionless soil is loose and cohesive soil is very soft, stabilization of the surrounding becomes decisive as function of bentonite slurry. If cohesionless soil is fairly dense and cohesive soil is soft, both stabilization of the surrounding and lubrication of the pipe string become decisive functions of bentonite slurry. If cohesionless soil is dense and cohesive soil is hard, lubrication of the pipe string becomes decisive as function of bentonite slurry.

In this study sand - concrete interface behavior with and without bentonite slurry, was investigated with the help of bentonite slurry injection in sand test, shear box test and pull out tests made in the laboratory and measurements made on real microtunnel constructions at the jobsite. In the bentonite slurry injection in sand test, penetration of bentonite slurry in sand is observed and optimum stable slurry is investigated. In the shear box test interface coefficient of friction between bentonite slurry injected sand and concrete is tried to be measured. In the pull out test, interface friction force between sand and concrete sample is measured for no bentonite condition, bentonite slurry injection without constant injection pressure condition and bentonite slurry with constant injection pressure condition. At the part of jobsite study, jacking forces are recorded with and without bentonite slurry injection. After the calculation of interface coefficient of friction from laboratory pull out tests data and jobsite jacking force data, from view point effect of bentonite

slurry injection on interface coefficient of friction, laboratory and field results compared and evaluated together.

This study summarizes the results of the effect of bentonite slurry injection on pipe jacking forces in sand, and provides recommendations for further research on the type (pattern) of bentonite slurry injection. Though jacking forces have previously been studied by others, these studies have largely focused on empirical analyses, and few have investigated the bentonite penetration mechanisms. The focus of the study contained herein is to identify the mechanism of lubrication at the interface between concrete jacking pipes and sand, and to develop a bentonite slurry injection method not only useful for lubrication but also soil stabilization. The primary conclusions about results of the research are summarized herein.

- (i) It is impossible to make an efficient bentonite injection without a stable slurry which provides not only stabilization but also lubrication. Bentonite slurry concentration and aging is very important to obtain a stable injection slurry. At the laboratory tests it is observed that, bentonite slurry should contain bentonite mass 6% of water mass and should be hydrated for 24 hours, because if bentonite slurry is not hydrated enough with adequate concentration, after injection into overcut, bentonite slurry will start to bleed water and cannot function as an impermeable membrane.
- (ii) It is observed that at TEST 1, when no bentonite is used, interface coefficient of friction μ_{int} is 0,46. When controlled bentonite injection without constant injection pressure is applied, interface coefficient of friction μ_{int} decreased to 0,24 in TEST 2 such as at Case 2 interface coefficient of friction μ_{int} is reduced from 0,5 to 0,25. Even though injection is not applied under a constant pressure e.g. controlled bentonite injection, after the application of bentonite injection, coefficient of friction forces can decrease to half of the case of no bentonite injection. As a result, it can be said that bentonite injection application in sand can cause 50% reduction of the jacking forces, so that bentonite injection can provide at least two times longer drives.
- (iii) When bentonite is applied under constant pressure such as at Test 3, the

interface friction coefficient is decreased from 0,46 to 0,06 which means that there is not any interface between sand and concrete but only bentonite and pipe. Hence, it can be said that by applying steady bentonite injection, friction coefficient was reduced to approximately to 10% of the coefficient of interface friction between soil and concrete pipe. So that, if bentonite injection is able to be applied in sync with excavation constantly, 10 times longer drives can be achieved in sand due to the meaningful decrease (87%) of the interface friction coefficient.

- (iv) By making 10 times longer drives, not only the number of jacking pits but also the mobilization time will decrease meaningfully. All of these parameters provide finishing pipe jacking projects in a shorter time with at least 20% less cost.
- (v) Even though fully bentonite pressurized Test 3 is repeated with more than 5 cm, 10 cm and 20 cm height of bentonite over the surface of the sample, interface coefficient of friction ($\mu_{int} = 0,06$) did not change. This means that high pressure injection is not required for lubrication, but injection pressures like contact injection pressures but higher than water pressure may be adequate to keep overcut open.
- (vi) In the recently studies on lubrication, from the observations and measurements on jacking forces at the jobsite applications alone, the reason of 90% reduction of coefficient of friction is concluded as the application of mass bentonite slurry injection which means application of large volumes of bentonite slurry during microtunneling. So that, for efficient lubrication it was recommended to make bentonite slurry injection as much as possible. But in this study from the evaluation of laboratory tests with field data together for efficient lubrication such as 87% reduction of interface coefficient of friction, it was concluded that bentonite slurry injection should be done under a constant injection pressure with continuous feeding of bentonite slurry in a pressure closed system.
- (vii) In Tuzla Project at drive between 5-6, average daily progress was 25 meters and 5 tons of bentonite was states to be used lately and at distance 122 meters jacking forces reached 300 tons, but at drive between 7-8, average daily progress was 20 meters and 4 tons of bentonite was used in time and at distance 122

meters jacking forces reached 150 tons. Even though much more bentonite is injected, due to collapse of overcut, late bentonite injection could not be as useful as in time bentonite injection. Hence, as a result it can be said that, in pipe jacking projects, in order to keep overcut open, bentonite injection should be started just after the beginning of excavation and should not be applied slower than the excavation (progress) speed.

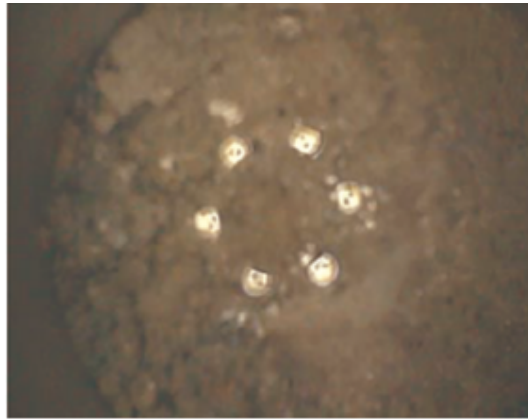
- (viii) All in all, a sufficient amount of lubricant that has a proper concentration should be injected into the annular gap under a certain injection pressure. This lubricant penetrates into the subsoil to a certain depth and prevents, under an adequate balancing pressure, the direct contact between the surface of the jacking pipe and the surrounding ground. In the annular gap, which is completely filled with the bentonite suspension, a lubrication layer is formed around the pipe surface. The skin friction acting on the surface of the jacking pipe is decreased effectively by the reduction of the friction coefficient ν_{int} .

6.2. Recommendation of a Renovation for Further Research

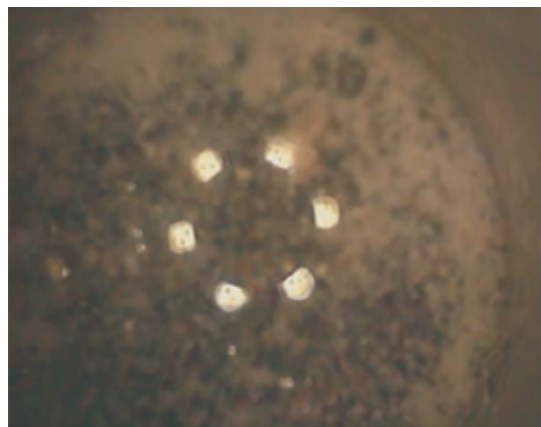
In common practice, controlled bentonite injection that applied from the top of the pipes to overcut is used alone for lubrication, but bentonite injection without constant pressure cannot prevent collapse of soil on the pipes. As shown in photographs at Figure 6.1 a,b,c which were taken from the top of jacking pipes to observe convergence of the overcut gap of a microtunneling project, when amount and pressure of the bentonite in overcut reduce, overcut gap begins to collapse and frictional forces starts to rise suddenly.



(i) Overcut gap photo near to bentonite injection port.



(ii) Overcut gap photo away from bentonite injection port.



(iii) Overcut gap photo far away from bentonite injection port.

Figure 6.1. (i) Overcut gap photo near to bentonite injection port, (ii) Overcut gap photo away from bentonite injection port and (iii) Overcut gap photo far away from bentonite injection port (Khazaei, Shimada and Kawai, 2006).

To prevent collapse, tail bentonite injection in which bentonite grout should be injected from the back of TBM shield into overcut horizontally, and simultaneously with TBM advance (Figure 6.2).

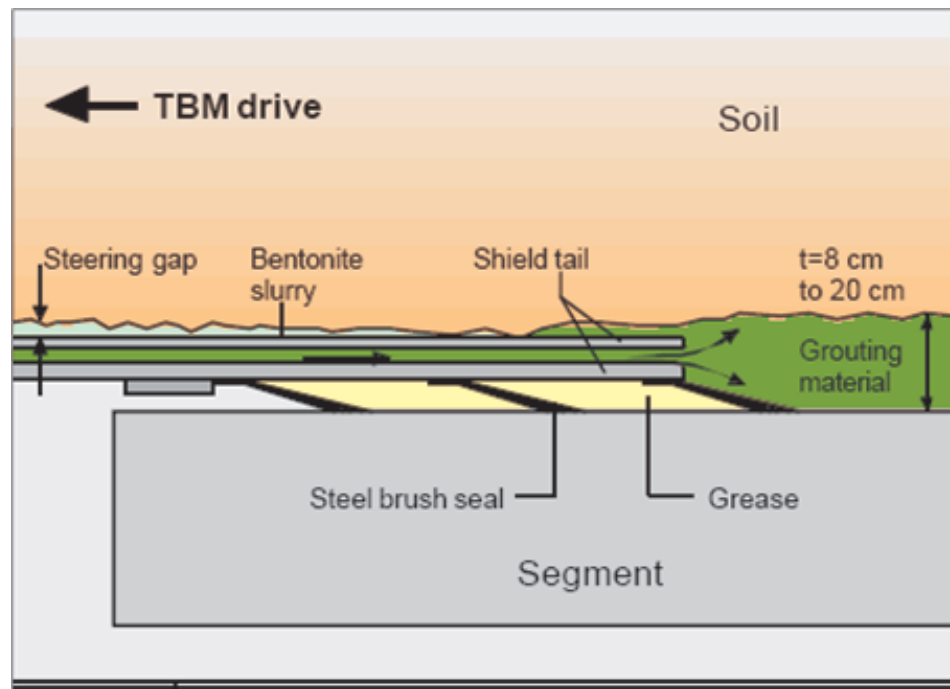


Figure 6.2. Tail bentonite injection (Babendererde, 1999).

In order to keep overcut open and prevent smooth interface between concrete pipe and sandy soil, tail injection is required such as applied in segmental lining tunnel constructions (Figure 6.2). In addition to tail injection there must be a pressure lock at the end of the tail of the machine to keep injection pressure constant (Figure 6.3).

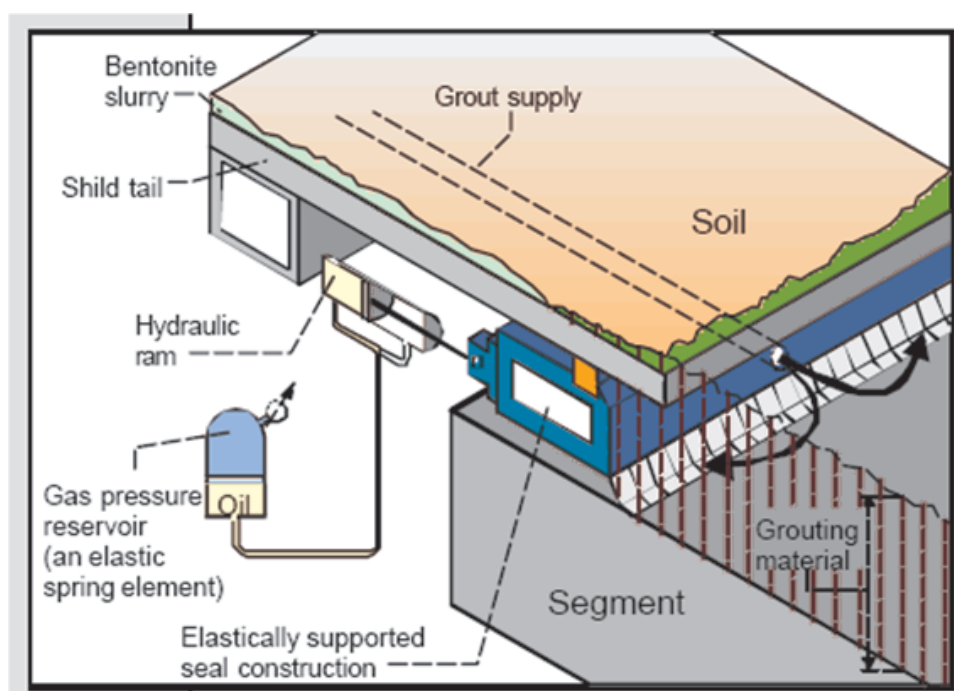


Figure 6.3. Tail bentonite injection with pressure lock (Babendererde, 1999).

Double Stage Injection System in which amount and pressure of bentonite is controlled fully automatically and injection is applied from both tail of TBM and top of pipes, can give opportunity to make less risky but more economical and longer pipe jacking drives. Collapse of soil on the pipe string can be prevented, if bentonite injection should be applied in sync with excavation from both tail of TBM and top of pipes with constant pressure lock (Figure 6.3). Hence, 10 times longer drives can be achieved in sand due to the meaningful decrease (87%) of the interface friction coefficient.

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