Abstract

Horizontal Directional Drilling (HDD) has been used internationally for the trenchless installation of utility conduits and other infrastructure. However, the mud loss problem caused by excessive mud pressure in the borehole is still a challenge encountered by trenchless designers and contractors, especially when the drilling crosses through cohesionless material. Investigation of mud loss problem is necessary to apply HDD with greater confidence for installation of pipes and other infrastructure.

The main objectives of this research have been to investigate the maximum allowable mud pressure to prevent mud loss through finite element analysis and small scale and large scale laboratory experiments. The recent laboratory experiments on mud loss within sand are reported. Comparisons indicate that the finite element method provides an effective estimation of maximum mud pressure, and “state-of-the-art” design practice- the “Delft solution” overestimates the maximum mud pressure by more than 100%. The surface displacements exhibit a “bell” shape with the maximum surface displacement located around the center of the borehole based on the data interpreted using Particle Image Velocimetry (Geo-PIV) program.

A parametric study is carried out to investigate the effect of various parameters such as the coefficient of lateral earth pressure at rest $K_0$ on the maximum allowable mud pressure within sand. An approximate equation is developed to facilitate design estimates of the maximum allowable mud pressure within sand.

A new approach is introduced to consider the effects of coefficient of lateral earth pressure at rest $K_0$ on the blowout solution within clay. The evaluations using finite element method indicate that
the new approach provides a better estimation of the maximum allowable mud pressure than the “Delft solution” in clay when initial ground stress state is anisotropic ($K_0 \neq 1$). Conclusion of this research and suggestions on future investigation are provided.
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Finally, and most importantly, I wish to thank my parents, brothers, sisters, and my girl friend- Xinning, for their encouragement, endless support, and love. To them I dedicate this thesis.
Statement of Originality

(Required only for Division IV Ph.D.)

I hereby certify that all of the work described within this thesis is the original work of the author. Any published (or unpublished) ideas and/or techniques from the work of others are fully acknowledged in accordance with the standard referencing practices.

(Hongwei Xia)

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List of Symbols

$A_s$  Cross section area of the test cell
$c'$  Cohesion of the soil
$C_u$  Uniformity coefficient
$D$  Diameter of borehole
$E$  Elastic modulus
$G$  Shear modulus
$G_s$  Specific gravity
$H$  Equivalent cover depth or burial depth
$H_i$  Initial height of full saturated slurry
$H_f$  Final height of full saturated slurry
$K_0$  Coefficient of later earth pressure at rest
$L$  Length of borehole
$M$  Critical state stress ratio
$P_i$  Internal mud pressure
$P_0$  Initial overburden pressure
$P_{\text{max}}$  Maximum mud pressure
$P_{\text{limiting}}$  Limiting mud pressure
$P_{\text{max all}}$  Maximum allowable mud pressure
$P_f$  Mud pressure at onset of plastic failure
$Q$  A function of the shear modulus and effective stress
$R$  The radius to the point at which the stresses are calculated
$R_0$  Initial radius of the borehole
$R_{p,\text{max}}$  Maximum allowable radius of the plastic zone
$S_u$  Undrained shear strength
$V_s$  The volume of Speswhite Kaolin powder
$W_i$  Water content of the saturated slurry
$\kappa$  A Parameter used to distinguish between cylindrical and spherical analysis
λ Parameter of critical state model
Γ Parameter of critical state model
σ₀ Initial effective stress
σᵣ Radial stress component
σ₀ Tangential stress component
ϕ Internal friction angle
σ Total overburden pressure above the crown
γ Unit weight
ν Poisson’s Ratio,
ψ Dilation angle
θ Position angle to vertical
Chapter 1

INTRODUCTION

1.1 INTRODUCTION

The need to construct new, or replace most of the deteriorated existing underground utility infrastructure and to expand utility services in North America has increased significantly in recent years. Conventional pipeline installation is using cut-and-cover methods which involve the placement of the pipeline into a trench excavated to the desired construction depth, and then backfilling with excavated materials. Social impacts from conventional cut-and-cover methods to install underground infrastructure have also increased, especially in densely populated urban areas and in environmentally sensitive zones. Trenchless technologies (TT) have been developed as an alternative to conventional cut-and-cover methods for installation and replacement of underground facilities. These procedures minimize disruption to society, reduce environmental impact, and provide more cost effective solutions in many situations.

The North American Society of Trenchless Technology (NASTT 2001) defines Trenchless construction as “a family of methods, materials, and equipment capable of being used for the installation of new and replacement or rehabilitation of existing underground infrastructure with minimal disruption to surface traffic, business, and other activities”. The first widely employed trenchless technology was pipe lining used to rehabilitate gas, sewer, and water mains in the late 1970s. The early 1980s saw the development of microtunnelling for installation of pipelines in urban environments where surface disruption was a primary concern. Other trenchless technologies were subsequently developed to provide more efficient means of rehabilitating,
monitoring, and installing of buried conduits. Tables 1.1 and 1.2 show the large family of these trenchless construction methods (Iseley et al. 1997).

1.2 HORIZONTAL DIRECTIONAL DRILLING

1.2.1 Introduction

One of the principal trenchless technologies, Horizontal Directional Drilling (HDD) was pioneered in the United States in the early 1970s by an innovative road boring contractor who successfully completed a 183-m (600-ft) river crossing using a modified rod pushing tool with no steering capacity (Ariaratnam et al. 1999). By integrating existing technology from the oil well drilling industry and modern surveying and steering techniques, HDD is now commonly used for installing utilities under natural or manmade obstacles, especially transportation corridors. This method has revolutionized complicated river crossings for pipelines which were initially performed using conventional dredging methods, or by rerouting through long distances to cross over at a bridge location. Other benefits associated with HDD are minimized ground disturbance and the potential for reduced impacts on existing infrastructure and sensitive environmental areas.

The HDD industry is divided into three major sectors: large-diameter HDD (maxi-HDD), medium-diameter HDD (midi-HDD), and small-diameter (mini-HDD, also called guided boring), according to their typical application areas (Iseley et al. 1997). Table 1.3 describes and compares typical maxi, midi, and mini-HDD systems. Maxi-HDD and Midi-HDD were primarily used by the oil and gas industry on large-diameter, cross country pipeline transmission lines. Increasingly, Mini-HDD is being approved and used for small-diameter gas distribution lines in urban and suburban areas, as well for municipal water and telecommunication cables crossings at airports, highways, and waterways because of its advantages mentioned above.
1.2.2 Horizontal Directional Drilling Installation Process

The HDD process is divided into three stages: Pilot bore, pre-ream, and product pipe pull back. The pilot bore stage consists of drilling a pre-planned bore using a steerable slanted-face drill head that can be tracked from the ground surface. A schematic of the pilot bore process is shown in Figure 1.1.

Typical pilot bore diameters range from 100mm to 150mm, corresponding to the size of the drilling stem/pipe and drill bit used during the installation. The initial angle of the pilot bore is equivalent to the angle of the inclined carriage of the drill rig, typically adjusted to between 5° to 15° (Szczupak 1988). A battery powered electromagnetic transmitter located in the drill head assembly emits a magnetic signal that a receiver is able to read and interpret to provide data on drill head location, orientation, depth, pitch and roll. Walkover receivers or wire line technology is used to track the path of the drill head. The monitoring frequency varies depending on requirements for grade control. When a direction change is required, the drill rod assembly is thrust into the soil with a specific drill head orientation (slant face of the drill head oriented opposite to the direction change). When thrusting is accompanied by drill rod rotation, the current alignment of the drilling tool is maintained. The pilot bore drilling continues until the drill head assembly exits the ground surface at an intended location away from the drill rig. The pilot bore is generally smaller than the final design diameter of the conduit and is used to establish the prescribed bore alignment.

The pre-ream stage consists of installing a larger diameter reamer onto the drill string and running it through the existing pilot bore while rotating the drill rod assembly as shown in Figure 1.2. The primary task of the reamer is to enlarge the bore along the alignment while mixing the cutting soils with drilling fluid to form slurry that can be removed from the borehole under an induced
pressure gradient. Depending on subsurface conditions, installation parameters, drill rig capabilities, and pipe characteristics, the bore may be pre-reamed with multiple reamer passes and/or different sized reamers. As a general rule of thumb, the final borehole diameter should be the lesser of 1.5 times of the diameter of product pipe or the pipe diameter plus 300mm for installations greater than 200mm in diameter (Bennett et al. 2001)

Once the bore is sufficiently enlarged, the product pipe is attached to a pulling head and rotating swivel connected to the reamer and drill rod assembly. The product pipe is installed as the pulling head and reamer assembly is pulled back toward the drilling rig as shown in Figure 1.3. Use of the swivel helps isolate the pipe from the rotating drill rods and reamer, thereby reducing pipe torsion during the pull back process. For shorter bore lengths (less than 200 m), the product pipe is often attached to the reamer as it is pulled back to the entrance area for the last time.

Selection of proper drill bits and reamers for a drilling project is important. Soft ground requires wider blades for steering control. Hard ground conditions require stepped or tapered carbide for penetration. Heterogeneous ground conditions can present difficulties in the selection of the proper drilling equipment. Reamer choice is dictated by the site conditions. For clays, the reamer must be able to chop up the clay to prevent large pieces from clogging the reamer. Fluted or spiral reamers may be required to deal with rock conditions.

1.2.3 Drilling Fluid Used during Horizontal Directional Drilling
The HDD process involves use of a drilling fluid (also referred to as drilling mud) made up primarily of water and bentonite, with PH values between 8 and 10. In some cases, polymers are used as additives to improve the drilling fluid when the required fluid properties cannot be attained with fresh water and bentonite alone. Bentonite based drilling fluids are generally used
for sand and gravel soils. These granular materials are often referred to as non-reactive materials since they have low swelling and expansion potential in the presence of water. Polymer based drilling fluids are generally used in clay and shale deposits to suppress swelling tendencies.

The primary purposes of the drilling fluid are to remove the cuttings from the borehole, stabilize the borehole, and act as a coolant and lubricant during the drilling process. In addition, sand and gravel formations require the drilling fluid to (1) form a thin layer of ‘filtercake’ to prevent loss of drilling fluid into permeable formations, and (2) help support the borehole from collapsing. When drilling passes through cohesive soil (clay or silt), the drilling fluid is required to (1) enhance cleaning of the drill bit or back reamer, (2) delay natural in situ soil swelling tendencies in cuttings and surrounding soils, and (3) prevent balling (soil clogging) in front of the drill bit or back reamer.

1.2.4 Mud Loss Problems during Horizontal Directional Drilling

Generally, the drilling mud is pumped from the drill bit under pressure during maxi-HDD and midi-HDD operations. However, high drilling fluid pressures can cause inadvertent mud return or mud loss to the ground surface either as a result of tensile failure (frac-out) or unconfined shear failure (blowout) of the soil surrounding the borehole depending on the soil types, strength, and the initial stresses.

Legitimate concerns arise as a result of the pressurized fluid during horizontal directional drilling and the consequences of borehole collapse, mud loss or mud return through hydrofracture or blowout failure. Borehole collapse is typically caused by either failure to maintain internal borehole mud pressure (mud pressure is less than minimum mud pressure value) to keep the
borehole stable or by an unfavorable drilling stratum containing weak layer, highly fracture rock, noncohesive alluvial material, or cobbles. There are several negative consequences caused by the collapsed borehole such as creating high friction on the drill pipes or the product pipe which will damage the pipes or reduce the service life due to the high tension stress. The drilling fluid pressure in the annulus of the borehole has to be sufficient to prevent the collapse of the borehole, and also has to be smaller than a critical value (soil capacity) that soil can support to prevent the mud loss. Reasonable limits must be placed on maximum mud pressure in the annular space of the borehole to prevent inadvertent drilling fluid returns to the ground surface. Therefore, the maximum allowable mud pressure that soil can support during HDD must be determined, and used as an upper limit for the mud pressures that develop during HDD operations, to prevent mud loss and its consequences.

1.3 MOTIVATION, RESEARCH OBJECTIVES AND METHODOLOGY

1.3.1 Motivation

Mud loss can hinder the construction process, increase construction costs, reduce the service life of adjacent or overlying infrastructure or even damage adjacent infrastructure, and cause serious environmental problems especially when drilling under waterways or protected sensitive environment zones. These side effects of mud loss have curbed some pipe companies from employing this trenchless technology for installation of new pipelines. To apply this new technology with greater confidence for installation of buried pipelines, problems associated with the directional drilling process such as the exact nature of mud loss and the associated ground heave have to be investigated during project design, and then appropriate precautions can be taken to minimize the undesired side effects during construction.
The maximum mud pressure design is still based on generally either:

a. A rule of thumb (maximum pressure of 1 psi (7kPa) per foot (0.305m) of burial depth) or

b. The closed form solution introduced by Lugar and Hergarden (1988) and adopted by the U.S. Army Corps of Engineers (USACE) (generally considered to be “state-of-the-art” practice; also called the “Delft solution” by the trenchless technology community).

Considerable evidence (e.g. Duyvestyn 2004, Moore 2005, Wang and Sterling 2006, Elwood et al. 2007) is available that indicates the Delft solution provides an unsafe maximum allowable mud pressure. The causes of the overestimation of the maximum allowable mud pressure are likely related to the assumptions used in deriving the cavity expansion theory (Vesić 1972) on which it is based. In particular,

- The problem is defined as responding under plane strain condition.
- The borehole is axially symmetric, and the soil medium is homogeneous, isotropic, and of infinite size (neglecting the influence of the ground surface).
- The soil medium is assumed to be in an isotropic initial stress condition (the coefficient of lateral earth pressure, $K_0=1.0$, neglecting the influence of $K_0$).
- The effect of gravity is neglected (i.e., gradients of stress with depth).
- The gradient of soil strength is neglected (i.e. assume constant soil strength with depth).
- The soil response is modeled as elastic until the onset of shear failure which is defined using the Mohr-Coulomb failure criterion (based on cohesion and frictional angle of the material).
- Elastic deformations in the plastic zone are neglected.
These assumptions do not represent the real soil stress condition since soil is usually in an anisotropic stress condition \((K_0 \neq 1)\). Typical values of \(K_0\) for sand range from 0.35 to 0.65, for normally consolidated \((NC)\) clay range from 0.5 to 0.65, for lightly overconsolidated \((LOC)\) clay \(K_0\) is close to 1, and for heavily overconsolidated \((HOC)\) clay, \(K_0\) may be even larger than 3. The studies of Kennedy et al. (2004a) indicated that the maximum allowable mud pressure to prevent mud loss through hydrofracture depends on the initial stress in the soil (coefficient of lateral earth pressure at rest, \(K_0\)), and indicate that the coefficient of lateral earth pressure at rest can have an important effect on the maximum allowable mud pressure. Elwood (2008) discussed the measurement of horizontal stress using a novel soil pressure sensor (Talesnick, 2005) in the course of small scale hydrofracture/blowout tests within sand. The coefficient of lateral earth pressure at rest \((K_0)\) in the test cell ranges from 0.48 to 0.61 calculated from the measured horizontal soil pressures. This soil pressure sensor is planned to be employed during Kaolin clay consolidation (trying to obtain the coefficient of lateral earth pressure at rest, \(K_0\)) and hydrofracture/blowout test within a purely cohesive soil (Kaolin clay). To understand the function of the soil pressure sensor and evaluate its effectiveness for measuring horizontal stress, an additional investigation is performed in which the soil pressure sensors are used to measure the stress distribution around a buried HDPE pipe within uniform sand (both in dense and loose state). The discussions of measuring stress distribution on the buried HDPE pipe using the novel soil pressure sensor are presented in Appendix F.

During the pilot boring process, the stress condition around the drill bit may be three dimensional rather than satisfying plane strain condition, and the stress-free ground surface also influences the growth of plastic zone as indicating from results of finite element analysis within a purely cohesive clay (Xia and Moore, 2006). Installation designers and contractors still often encounter mud loss or inadvertent mud returns resulting from excessive mud pressures calculated using the
Delft solution, and numerous unexpected failures (i.e. inadvertent mud return) during the drilling projects have been reported (e.g. Christopher et al. 2003, Mathew et al. 2003, Harris, 2005, David et al. 2007).

The above discussion indicates that the Delft solution may not be able to simulate the real soil response during directional drilling, and to provide a reliable maximum mud pressure that the host soil can support. Limited physical data are available to evaluate the state-of-the-art practice, and to indicate the mechanism of mud loss (hydrofracturing or blowout). A primary objective of the current research is to design laboratory experiments to study the mechanisms of mud loss, and to quantify the maximum mud pressures (those that precipitate mud flow to the ground surface). Finite element analyses are employed to facilitate understanding of the mud loss problem and to quantify the maximum allowable mud pressure, since laboratory studies alone cannot be sufficient, as it is very difficult or impossible to study all possible boundary conditions, borehole geometries and soil conditions in the laboratory.

1.3.2 Research Objectives and Methodology

The main objectives of this research are to understand better the mechanisms of mud loss through scaled laboratory tests and finite element studies in sand and clay, to evaluate the effectiveness of the state-of-the-art practice (the “Delft solution”), and to quantify the maximum mud pressure when drilling through sand and clay. Another goal of this research is to help understand how the initial stress condition (i.e. the coefficient of lateral earth pressure at rest, $K_0$) influences the mode of mud loss and the soil capacity (the maximum mud pressure) to resist the mud pressure, as well as to explain the effects of sand properties (dilation angle in particular). To achieve the above goals, the following research methods have been employed:
• Past studies on mud loss mechanisms and methods to estimate the maximum mud pressure are reviewed. This review includes new small scale and large scale laboratory tests to study mud loss and quantify the maximum mud pressure reported by Elwood (2008).

• Both plane strain and three dimensional finite element analyses are employed to simulate the small scale and large scale laboratory tests reported by Elwood et al. (2007).

• Two additional large scale laboratory tests are conducting involving different test dimensions (greater cover depth and longer test borehole), to extend the comparisons between the results from laboratory tests, the “Delft solution”, and finite element analysis.

• Parametric studies are carried out to consider other factors such as soil dilation and $K_0$ on the maximum allowable mud pressure. Approximate equations for estimating the maximum allowable mud pressure during HDD within cohesionless material are developed by carrying out parametric studies on non-cohesive soils using plane strain finite element analysis.

• A theoretical study is performed to develop a new approach to estimate the maximum allowable mud pressure in purely cohesive soils (clay under undrained loading conditions) to consider the effect of coefficient of lateral earth pressure at rest, $K_o$.

• Plane strain finite element analyses are performed to evaluate the new approach.

• The preparations for hydrofracture tests within purely cohesive clay (Kaolin) are discussed including the design of a top extension box, the choice of test sample, the measurement of horizontal soil pressure, affiliated data acquisition system, and the study of possibility of conducting hydrofracturing experiments in purely cohesive soil using the small test cell.
1.4 OUTLINE OF THE THESIS

This thesis consists of seven chapters. Chapter One is an introduction to define the problem, and explain the motivation and objectives of the research. It ends with this overview of the organization of the thesis.

Chapter Two reviews the past research on mud loss (hydrofracture or blowout failure) within cohesionless material during drilling. The details of the investigations on mud loss within poorly graded sand reported by Elwood’s are outlined. Both plane strain and three dimensional finite element analyses are carried out to evaluate the performance of the small scale and large scale laboratory tests. The results from the finite element analysis and scaled laboratory tests are compared with the state-of-the-art practice (the “Delft solution”) that is currently used by designers to calculate the maximum allowable drilling mud pressure (Arends 2003).

Chapter Three presents the details of two additional large scale laboratory tests conducted to study the mud loss mechanism, and further understand the maximum allowable mud pressure that sand can support at different cover depths. Finite element analyses are also performed to compare with the results from large scale laboratory tests and the state-of-the-art practice. Conclusions and suggestions are made based on these comparisons.

Chapter Four describes the parametric studies of maximum allowable mud pressure within cohesionless material to better quantity the maximum allowable drilling mud pressure under different field conditions. The parameters to be considered include the coefficient of lateral earth pressure at rest ($K_0$), the internal friction angle of the host sand ($\phi$), the dilation angle of the host sand ($\psi$), the unit weight of the sand ($\gamma$), the elastic modulus ($E$), and the ratio of cover depth ($H$).
to the diameter of the borehole ($D$). Approximate expressions are also developed to permit design calculations using a spreadsheet.

Chapter Five reviews mud loss within purely cohesive soil (clay responding under undrained conditions), including the design equation of blowout (Arends 2003) and hydrofracturing (Kennedy 2004a and 2004b). A new approximate approach is developed to estimate the maximum allowable drilling mud pressure. The new approach can be used to calculate the maximum allowable drilling mud pressure in cases where the lateral earth pressure coefficient, $K_0$, is not covered by either the solution of the Kennedy et al. (2004a) solution or current design equation. Plane strain finite element analyses were performed to evaluate the new approximate procedure. Chapter Five ends with a study of the maximum allowable drilling mud pressure and the mechanism controlling mud loss for both normally consolidated (NC) and overconsolidated (OC) clays (lightly overconsolidated and heavily overconsolidated clays).

Chapter Six describes the preparations for hydrofracturing tests within purely cohesive clay, and the use of finite element analysis to study the possibility of performing hydrofracturing tests with purely cohesive soil in the small test cell.

The seventh chapter summarizes the research procedures and results, and conclusions. A brief discussion of the value of the proposed design guidelines for HDD in clay and poorly graded sand is presented. Recommendations for future work needed to gain further understanding of the phenomena are then presented, including additional numerical modeling as well as laboratory and field tests.

Appendix A presents the data measured from the direct shear test to obtain the basic sand properties for the finite element analysis. Appendix B presents the example input file for the finite
element program ABAQUS. Appendix C presents the details of Mann-Whitney test. Appendix D presents a calculation example to illustrate the use of the new approximate equation to estimate the maximum mud pressure within sand. Appendix E presents the use of the new approach to estimate the maximum mud pressure within clay. Appendix F presents a research study undertaken using a novel soil sensor to illustrate use of those cells to measure the soil pressure acting on the surface of the buried HDPE pipe.

REFERENCES


Table 1.1 Description of trenchless construction methods (modified from Iseley et al. 1997)

<table>
<thead>
<tr>
<th>Method Type</th>
<th>Method Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auger Boring (AB)</td>
<td>A technique that forms a bore hole from a drive shaft to a reception shaft by means of a rotating cutting head. Spoil is transported back to the drive shaft by helical wound auger flights rotating inside a steel capability. It does not provide continuous support to the excavation face. AB is typically a 2-stage process (i.e. casing installation and product pipe installation)</td>
</tr>
<tr>
<td>Slurry Boring (SB)</td>
<td>A technique that forms a bore hole from a drive shaft to a reception shaft by means of a drill bit and drill tubing (stem). A drill fluid (i.e. bentonite slurry, water, or air pressure) is used to facilitate the drilling process by keeping the drill bit clean and aiding with spoil removal. It is a 2-stage process. Typically, an unsupported horizontal hole is produced in the first stage. The pipe is installed in the second stage.</td>
</tr>
<tr>
<td>Microtunneling (MT)</td>
<td>A remotely controlled, guided pipe jacking process that provides continuous support to the excavation face. The guidance system usually consists of a laser mounted in the drive shaft communicating a reference line to a target mounted inside the MT machine’s articulated steering head. The MT process provides ability to control excavation face stability by applying mechanical or fluid pressure to counterbalance the earth and hydrostatic pressures.</td>
</tr>
<tr>
<td>Pipe Ramming (PR)</td>
<td>A technique for installing steel casings from a drive shaft to a reception shaft utilizing the dynamic energy from a percussion hammer attached to the end of the pipe. A continuous casing support is provided and over excavation or water is not required. This is a 2-stage process.</td>
</tr>
<tr>
<td>Pipe Jacking (PJ)</td>
<td>A pipe is jacked horizontally through the ground from the driven shaft to the reception shaft. People are required inside the pipe to perform the excavation and/or spoil removal. The excavation can be accomplished manually or mechanically.</td>
</tr>
<tr>
<td>Soil Compaction (SC)</td>
<td>This method consists of several techniques for forming a borehole by in-situ soil displacement using a compaction device. The compaction device is forced through the soil, typically from a drive shaft to a reception shaft, by applying a static thrust force, rotary force and/or dynamic impact energy. The soil along the alignment is simply displaced rather than removed. This is a 2-stage process</td>
</tr>
<tr>
<td>Horizontal Directional Drilling (HDD)</td>
<td>A 3-stage process that consists of drilling a small diameter pilot hole along a predetermined path, then enlarging the borehole using a reamer, and pulling a product pipe into place through that borehole.</td>
</tr>
<tr>
<td>Type&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Pipe/Casing Installation Mode</td>
</tr>
<tr>
<td>-----------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>AB</td>
<td>Jacking</td>
</tr>
<tr>
<td>SB</td>
<td>Pulling/Pushing</td>
</tr>
<tr>
<td>MT</td>
<td>Jacking</td>
</tr>
<tr>
<td>PR</td>
<td>Hammering/Driving</td>
</tr>
<tr>
<td>PJ</td>
<td>Jacking</td>
</tr>
<tr>
<td>SC</td>
<td>Pulling</td>
</tr>
<tr>
<td>HDD</td>
<td>Pulling</td>
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</tbody>
</table>

<sup>a</sup> AB-Auger Boring; SB-Slurry Boring; MT-Microtunneling; PR-Pipe Ramming; PJ-Pile Jacking; SC-Soil Compaction; HDD-Horizontal Directional Drilling

<sup>b</sup> Steel-Steel Casing Pipe, RCP-Reinforced Concrete Pipe, GFRP-Glass-Fiber-Reinforced Pipe, PCP-Polymer Concrete Pipe, VCP-Vitrified Clay Pipe, DIP-Ductile Iron Pipe, PVC-Polyvinyl Chloride Pipe, HDPE-High Density Polyethylene Pipe
Table 1.3 Comparison of the main features of typical Maxi-, Midi-, and Mini-Horizontal Directional Drilling (modified from Iseley et al. 1997)

<table>
<thead>
<tr>
<th>Description</th>
<th>Product Pipe</th>
<th>Depth</th>
<th>Bore Length</th>
<th>Variants of Drilling fluid</th>
<th>Typical Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maxi-HDD</td>
<td>600-1200mm</td>
<td>≤61m</td>
<td>≤1500 m</td>
<td>Fluid recirculation</td>
<td>River and Highway crossings</td>
</tr>
<tr>
<td></td>
<td>(24-48 in)</td>
<td>(200 ft)</td>
<td>(5,000 ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Midi-HDD</td>
<td>250-600mm</td>
<td>≤23m</td>
<td>≤274 m</td>
<td>Fluid recirculation and suspension</td>
<td>Under Rivers and roadways</td>
</tr>
<tr>
<td></td>
<td>(10-24 in)</td>
<td>(75 ft)</td>
<td>(900 ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mini-HDD</td>
<td>50-250mm</td>
<td>≤4.5m</td>
<td>≤183 m</td>
<td>Fluid suspension</td>
<td>Telecom, power cables, and gas lines</td>
</tr>
<tr>
<td></td>
<td>(2-10 in)</td>
<td>(15 ft)</td>
<td>(600 ft)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 1.1 Pilot boring during Horizontal Directional Drilling (modified from Ariaratnam et al. 2001)

Figure 1.2 Back reaming during Horizontal Directional Drilling (modified from Ariaratnam et al. 2001)

Figure 1.3 Pipe pull back during Horizontal Directional Drilling (modified from Ariaratnam et al. 2001)
2.1 INTRODUCTION

Horizontal Directional Drilling (HDD) is commonly used in North America to minimize social, economic and environmental impact on the surrounding community relative to conventional pipe installation methods (i.e. “cut-and-cover” installation). Pressurized drilling fluid (mud), a mixture of bentonite and water plus polymer additives, is normally used during the drilling processes to transport excavated drill cuttings back to the ground surface, to clean and cool the drill bit, to reduce sidewall resistance on the pulled pipe during the pull back stage, and to prevent the borehole from collapse (Ariaratnam et al. 1999). Excessive mud pressures may result in mud flow through tensile fractures (hydrofracture or frac-out) or unconfined plastic flow (blow-out) within the soil in the vicinity of the borehole, and causing problems such as those described by Christopher et al. (2003), Mathew et al. (2003), Harris (2005), Duyvestyn (2006), and David et al. (2007).

Mud loss mostly occurs as a result of excessive mud pressure in the borehole, and researchers have been working to quantify the maximum mud pressure in a borehole that can be supported by the surrounding soil as one potential approach to control the mud loss phenomenon. Some investigations on the maximum allowable mud pressure to prevent the occurrence of hydrofracturing or blowout have been carried out for drilling in purely cohesive soils, either using theoretical methods, experimental investigations, or numerical simulations (e.g. Anderson et al. 2007).

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In this chapter, past investigations of mud loss mechanisms and maximum mud pressure within cohesionless materials are reviewed. Some results from recent experiments (Elwood et al. 2007, Elwood 2008) on mud loss in cohesionless materials are presented (both small scale and large scale tests). A series of numerical analyses are conducted for comparison with the experimental investigations; those investigate the influence of the boundary conditions associated with the test apparatus as well the testing borehole configurations (i.e. three dimensional response versus those for plane strain conditions). Finally, the results from these numerical simulations and the laboratory hydrofracturing/blowout tests are used to evaluate the state-of-the-art practice (the “Delft solution”), and some conclusions and suggestions are provided regarding current studies and further investigations.
2.2 REVIEW OF PAST RESEARCH

2.2.1 Theoretical Investigation

In general, past theoretical investigations were usually based on the assumption that most cases of mud loss resulted from shear failure or blowout, where unconfined plastic flow develops in the soil surrounding the borehole. Keulen et al. (2001) introduced the use of cavity expansion theory (usually called the “Delft solution” by the trenchless technology community) for shear failure as a result of internal cavity expansion, and this is generally considered to be the state-of-the-art practice for estimating maximum allowable mud pressure by the U.S. Army Corps of Engineers-USACE (Carlos et al. 2002). This closed formed solution is based upon the cylindrical cavity expansion theory of Vesic (1972). It assumes that the borehole is axially symmetric and infinitely long, and the medium is homogeneous, isotropic, and of infinite size. The medium surrounding the borehole is assumed to be in an isotropic initial stress condition (i.e. $K_0=1$), and it is assumed to respond under plane strain conditions. The medium response is modeled as elastic until the onset of shear failure, which is quantified using the Mohr-Coulomb failure criterion as a function of cohesion and friction angle. Increments of elastic deformation are assumed to obey Hooke’s law, and elastic strains in the plastic zone are neglected. Luger and Hergarde (1988) advanced the work of Vesic by developing a numerical model based on the expansion of a pre-existing cylindrical cavity within an infinite medium, and appear to be the first to have applied it to investigate directional drilling. The solution defines maximum allowable mud pressure and limiting mud pressure as:

$$P_{\text{max,alt}} = (P_f + ccot\phi) \left\{ \left( \frac{R_{\text{c,max}}}{R_0} \right)^{-z} + Q \right\}^{\frac{-\sin\phi}{1+z\sin\phi}} - ccot\phi \quad [2.1]$$

$$P_{\text{lim}} = (P_f + ccot\phi)(Q)^{\frac{-\sin\phi}{1+z\sin\phi}} - ccot\phi \quad [2.2]$$
where: $P_{\text{max, \text{all}}}$ is maximum allowable mud pressure, $P_{\text{lim}}$ is limiting mud pressure when $R_{p,\text{max}}$ approaches infinity, $P_{\text{max}}$ is maximum mud pressure from the Delft solution is the pressure which produce the plastic zone up to the ground surface ($R_{p,\text{max}}/R_0=2H/D$ in Equation 2.1, $H$ is the burial depth, $D$ is the diameter of the borehole), $P_f$ is mud pressure at onset of shear failure, $P_f = \sigma_0'(1 + \sin\phi) + c \cos\phi$, $\sigma_0'$ is the initial effective overburden stress, $\phi$ is internal friction angle of the soil, $c$ is cohesion of the soil, $Q$ is a function of the shear modulus and effective stress, which can be expressed as: $Q = \frac{\sigma_0' \sin\phi + c \cos\phi}{G}$, $G$ is shear modulus of the soil, $R_0$ is initial radius of the borehole, $R_{p,\text{max}}$ is the maximum allowable radius of plastic zone.

To estimate maximum allowable mud pressure in real drilling practice, Lugar and Hergarden (1988) suggested that mud pressure should be less than 90% of the limiting mud pressure calculated from Equation 2.2 in purely cohesive materials. However, Van and Hergarden (1997) suggested that the maximum allowable mud pressures should be reduced further to account for inconsistencies in the soil deposits (heterogeneities), and concluded that the maximum allowable pressures should be restricted to that generating shear failure one half of the way to the ground surface ($R_{p,\text{max}}=1/2H$ in Equation 2.1) in purely cohesive materials and to two thirds of the way to the surface ($R_{p,\text{max}}=2/3H$ in Equation 2.1) in cohesionless materials. This suggestion for the maximum allowable mud pressure provided by Van and Hergarden (1997) was accepted by the Pipeline Research Council International (Staheli et al. 1998) and later by the USACE (Carlos et al. 2002).

### 2.2.2 Numerical Analyses

Kennedy et al. (2006) performed a series of finite element analyses to examine the effects of mud pressure during directional drilling in sand on circumferential stresses adjacent to the borehole in soil with a initial anisotropic geostatic stresses states (i.e. coefficient of lateral earth pressure at
rest, \(K_0 \neq 1\). This study considered the sand and a “filtercake (FC)” system, in which the filtercake is a composite layer of sand and drilling fluid formed under internal borehole mud pressure during drilling. The filtercake was assumed to exhibit undrained response with no volume change (i.e. purely cohesive material behavior). Its undrained shear strength \((S_u)\) was established based on the initial frictional strength of the sand particles, and was calculated as:

\[
(S_u)_{FC} = 0.5\sigma_0(1 + K_0) + \sin\phi'.
\]  

The elastic parameters (undrained elastic modulus, \(E_u\), and Poisson’s ratio, \(\nu_u\)) of the filtercake were estimated by assuming that its undrained shear modulus is equal to the shear modulus of the native sand. This study also investigated how the mud pressure, the thickness of filtercake and the undrained shear strength of the filtercake influence the development of hoop stress at the crown of the borehole. The possibility of occurrence of tensile fracture mainly depends on the undrained shear strength of the filtercake. If shear failure initiates first, the tangential stresses steadily become more compressive, thereafter, no tensile stress would develop in the filtercake. However, the filtercake behavior (undrained response) assumed here does not match the filtercake characteristics reported by Elwood et al. (2007), where the filtercake was observed to feature little or no changes in physical properties relative to the native sand, except that the moisture content increased (the moisture content ranges from 2% to 6% in native sand, and increased to the range of 20% to 24% in the filtercake samples), and the hydraulic conductivity decreased (the initial hydraulic conductivity of the sand was measured to be 3.2x10^{-3} \text{cm/s}, and this decreased to 7.0x10^{-6} \text{cm/s} in the filtercake samples). Elwood (2008) recommended strength measurements for filtercake using triaxial testing. While low quality filtercake (i.e. highly permeable, the stability effect of filtercake will be minimized due to the equilibrium between the pore pressure and the internal mud pressure) was formed due to the clear bentonite slurry used during the tests, stronger filtercake would form later in construction once the drilling mud become contaminated by silt and
other particles, and the assumption of zero filtercake thickness in numerical simulation should be conservative.

Wang and Sterling (2006) investigated the stability of a borehole wall during HDD through loose sand with an emphasis on the role of the filtercake in stabilizing the borehole. The studies assume that in addition to cavity expansion, pore pressures may develop within the sand during deformation and that these ultimately reduce the effective shear strength of the soil, resulting in liquefaction and consequently loss of borehole stability. Their studies were based upon a numerical model designed to determine the effect of filtercake thickness on the overall strength of the borehole. It was found that in cases with little or no filtercake, the effective stresses within the borehole would increase rapidly even at very low mud pressures and likely result in liquefaction. For the case where sufficient filtercake forms, they found that the development and growth of the zone of shear failure surrounding the borehole wall lead to the hydraulic fracture (i.e. tensile fracture), which may eventually lead to mud circulation loss. The numerical results indicated that the maximum mud pressure to ensure borehole stability is equal to the earth pressure at the level of the borehole in the soil. However, to date, there is no physical evidence to support the assertion that static liquefaction as the mechanism causing borehole instability.

Duyvestyn (2004) performed numerical studies using FLAC to investigate the maximum allowable mud pressure for borehole in loose and dense sand. The studies culminated in a design criterion based on displacement or strain control. The maximum allowable mud pressure is the pressure required to make the deformation of an open borehole reach half an inch (12.5mm). Duyvestyn (2004) concluded that the current state-of-the-art practice provides an overestimated maximum mud pressure value or unsafe maximum plastic radius as comparison indicated. His numerical studies did not consider the formation and influence of filtercake and non-hydrostatic initial stresses (i.e $K_0\neq1.0$).
2.2.3 Experimental Studies

Elwood et al. (2007) and Elwood (2008) described a series of laboratory tests to examine the physical nature of mud loss in frictional material, and investigate the soil deformation (surface displacement) during borehole expansion, and with the objective to quantify the maximum allowable mud pressure. The nature of the filtercake that formed in his experiments was investigated through comparisons between the changes in gradation, moisture content and hydraulic conductivity of the soil fully saturated with drilling mud using standard laboratory tests.

In these studies (Elwood et al. 2007, Elwood 2008), 10 small scale tests (denoted T1 to T10 in Table 2.1), and 6 large scale tests (denoted LS1 to LS6 in Table 2.2) were carried out. The large scale tests also included 2 (denoted LS5 to LS6 in Table 2.2) tests to examine the influence of a stiff surface layer on the mode of mud loss; a layer of well graded gravel material was placed and densely compacted over the top of the sand. The small scale tests were conducted in the testing system illustrated in Figure 2.1. The test cell is 0.78m high, 0.80m long, and 0.32m wide. Friction between the walls and backfill soil was minimized by placing three layers of polyethylene sheets lubricated with high-temperature silicone bearing grease along the walls. This arrangement can reduce the boundary friction to less than 5º (Tognon et al. 1999). These laboratory tests were carried out in four stages as follows: In stage 1, the soil was placed from a height approximately 1.5m above the base of the cell in 200mm loose lifts and manually compacted using a hand tamper until the soil reached the top of the cell. The overburden pressure was applied to establish the geostatic stresses condition in second stage by an MTS servo-controlled test system acting on a metal plate placed over three hardwood planks. A 0.6m long clean borehole was formed using a specially designed Shelby tube (the outside diameter is 0.045m) in stage 3. Since the first 0.2m of the borehole was sealed with an expandable packer, the total length of borehole where the mud
was in contact with the surrounding ground was 0.4m. The mud pressure was applied through a
displacement pump until the mud loss was observed at the surface of soil in the final stage.

The soil material used in each small scale test was uniform sand with a mean grain size \(D_{50}\) of
0.2mm, and a uniformity coefficient \(C_u\) of 3.4, which was obtained from a source pit located in
the Kingston area. This material is typically used for bedding material for lightweight buried
structures (Elwood et al. 2007). Details of the sand properties are listed in Appendix A. The
measured bulk densities of the sand range from 1684 to 1767 kg/m\(^3\) with a relative compaction,
\(R(\%)\), of 85%. The horizontal soil pressure acting on the vertical walls was monitored in some
tests using soil pressure sensors described by Talesnick et al. (2005). The coefficient of lateral
earth pressure at rest, \(K_0\), in the test cell ranged from 0.48 to 0.61 based on the measured
horizontal soil pressures, and an average \(K_0\) value of 0.55 will be assumed in subsequent
calculations in this chapter.

Four large scale tests with only sand (denoted LS1 to LS4 in Table 2.2) were performed in a
reaction pit with length 2m, width 2m, and height 1.5m. The same poorly graded medium sand
used in the small scale tests was employed, and was compacted using a vibratory plate (Wacker
Packers, WP1550) to an average bulk density of 1750kg/m\(^3\). The 55 mm diameter borehole was
carefully bored using a hand auger at a cover depth of 0.78m and length of 1.5m. An expandable
packer was placed in the borehole to seal it, and leave about 0.5m of the borehole available for
testing. The ground surface displacements during borehole expansion were calculated by
interpreting a series of high resolution pictures taken by two Cannon cameras oriented at right
angles using the image processing software Geo-PIV (White and Take, 2002). The borehole mud
pressure was measured via an ATS pressure transducer (AST M4000) in both the small scale and
large scale tests.
Tensile surface cracks were observed in the large scale tests, likely due to soil dilation (volume increase) and borehole expansion due to increased mud pressure. The open surface cracks likely weakened the soil strength and provided channels along which mud flow could occur up to the surface as shown in Figure 2.2 a. The surface heave exhibits a “bell” shape with an average heave of 26 mm above the centerline of the borehole as shown in Figure 2.3, and tapers down to near zero with distance from the centerline of the borehole (Elwood et al. 2008).

Based on the phenomenon observed in the large scale tests, one possible mud loss mechanism can be hypothesized as shown in Figure 2.4. After the initial yield occurred in the soil surrounding the borehole, the plastic zone continued to grow. Meanwhile, the sand in the plastic zone dilates and ground heaves occurs and continues to accumulate as mud pressure increases (Figure 2.4 a). When the mud pressure reaches the maximum allowable mud pressure that the soil can support, tensile cracks occur on the ground surface and propagate down forward (Figure 2.4 b). After that, the mud being injected accelerates the propagation of surface cracks to deeper locations (down to the boundary of the plastic zone), until the borehole can no longer contain the injected mud, and it begins to flow out of the fractures (Figure 2.4 c). No tensile surface cracks were found in the small scale tests, likely because the top surcharge applied at the surface and the relative small scale of the test (distance from springline of the borehole to the side wall boundary was 3.2 times diameter of the borehole) prevented their occurrence.

Two further large scale tests using sand overlain by granular “A” were performed to consider the effects of a stiff surface layer on the mode of mud loss and the maximum mud pressure. A 300 mm thick layer of the well graded gravel was compacted to a dense state with an average density of 2050kg/m³ over the uniform sand. In the first test (corresponding to LS5 in Table 2.2),
relatively small surface cracks developed in comparison to those found in the ‘sand alone’ tests. The mud flowed out from the side wall as shown in Figure 2.2 b, instead of from the surface cracks as observed in the ‘sand alone’ tests. Post-test excavation evidence indicated that mud gathered at the interface of the two different material layers and then flowed along the path of least resistance which would have been through the least dense material. The second of these large scale tests (corresponding to LS6 in Table 2.2) was reported by Elwood (2008) as a failure (unsuccessful) since mud flowed out from around the packer as shown in Figure 2.2 (c) and (d) as a result of poor compaction near the corner. Larger volumes of mud were injected in tests LS5 and LS6 (average of 32.4L) compared to tests LS1 to LS4 (average of 15.2L). Details of the small scale and large scale tests are summarized in Tables 2.1 and 2.2, respectively.

2.3 NUMERICAL SIMULATIONS OF TESTS (T1 to T10 and LS1 to LS4)

2.3.1 Development of the Numerical Model

Three dimensional numerical models were used to simulate the small scale tests, to evaluate the validity of the experimental modeling process, and to investigate the effects of the side boundaries of the test box. Finite element analysis software ABAQUS version 6.5 was employed, with 20-node reduced integration brick element (C3D20R) chosen to reduce the calculation time, and symmetry was used to reduce the mesh size. Two dimensional numerical analyses (i.e. based on plane strain condition) were also performed considering the same cross section dimensions as these laboratory tests, to examine the implications of assuming plane strain conditions. Six-node triangular plane strain element (CPE6) was used considering the accuracy and calculation time. Examples of the meshes used for the three dimensional and plane strain analyses are shown in Figure 2.5. In each case the mesh becomes finer near the borehole, and smooth, rigid boundaries were placed along the bottom and sides of the mesh. The surcharge was applied on the top surface.
to simulate the stress applied in the test to simulate greater cover depth. Shear failure and the elastic-plastic constitutive response were modeled using the Mohr-Coulomb failure criterion.

A simple comparison was made to evaluate the efficiency of three dimensional and plane strain analyses. The plane strain condition was simulated using three dimensional elements (i.e. C3D20R and all elements were removed along the borehole) and these results were compared with simulations using the two dimensional plane strain element (CPE6). The difference in calculated values of maximum mud pressure obtained using those two different simulations was around 4.5%, which indicates that the 2D and 3D elements chosen here provide reasonably close solutions.

Knight et al. (2001) and Ariaratnam et al. (2005) performed a series of field tests to assess the annular space after pipe installation by directional drilling. Post-excavation evidence indicated that the initial HDD borehole geometry is a relatively uniform circular cross section due to the type of reamer and multiple reaming stages as shown in Figure 2.6. Yanagisawa and Panah (1994) performed a laboratory tests in clay to investigate the effect of borehole geometry where these featured a small notch along the borehole. They concluded that the local imperfection in borehole geometry did not have any effect on the hydraulic fracturing pressure and even the direction of the cracks. Moreover, the maximum mud pressure which is the primary focus here mainly depends on the burial depth, the strength of neighboring soil and the maximum diameter of borehole, therefore, a clean and circular borehole using in the laboratory experiments and considered in the numerical simulation appears reasonable. An irregular borehole geometry could be considered in a simulation, though this would make the simulation more complex and less general.
2.3.2 Numerical Simulation Procedure

Mud pressure was not applied in the course of forming the borehole after the establishment of geostatic stresses in the laboratory experiments. Instead, the borehole is formed first, and the mud is pressurized second. When drilling mud is injected into a borehole in the field it is already pressurized when the borehole is excavated. The difference caused by these different load paths was studied through two numerical models; one involved application of mud pressure after the excavation of the borehole, while the other model featured mud pressure application at the same as soil was removed. The calculated results for the maximum mud pressure indicate that no noticeable difference (less than 1%) resulted. Therefore, it is concluded that the procedure used in the laboratory experiments where the borehole was formed first, and then mud pressure is applied subsequently, is a reasonable representation of the real drilling process.

The initial geostatic stress state was established under the self-weight of the sand and the surface surcharge in the first solution step. In the second step, the borehole was excavated (elements within the borehole were removed), and a surface pressure of 30kPa was applied around the packer sealing area to simulate the sealing pressure of the packer in the tests. The borehole mud pressure was applied as a surface pressure to the inner surface of the borehole, and increased from zero to the required value in each subsequent step. A pressure increment of 5 kPa was chosen to minimize the calculation divergence in each time step that results when a large pressure increment is used. More details of the simulation steps can be found in Appendix B (i.e. an example ABAQUS model input file). The simulation was terminated when the plastic zone (the region of shear failure) reached the surface. The maximum mud pressure was considered to be the pressure which caused the plastic zone to extend two-thirds of the way to the surface in the numerical simulation. The same strategies used in the three dimensional simulation were
implemented in the plane strain simulation except that the packer pressure is neglected in the two dimensional model (the borehole is effectively being modeled as having infinite length).

2.3.3 Simulation Results for Small Scale Tests (T1 to T10)

In the course of simulation, ten different top surface surcharges were applied based on the load applied in the small scale laboratory tests to simulate different total overburden pressure or greater cover depth. The soil properties used in the three dimensional and plane strain analyses are listed in Table 2.3.

Figures 2.7 and 2.8 show the development of the plastic zone surrounding the borehole area under typical mud pressure values in both the three dimensional and plane strain simulations. In this example, the borehole diameter is 0.045m, and top surcharge of 16.0kPa was applied to simulate the total overburden pressure of 22.9kPa (i.e. 1.2m cover depth), corresponding to T3 and T4 in Table 2.1. The plastic zone at the crown of the borehole initiated at a low mud pressure, and a very small plastic zone appears at the springline of the borehole as shown in Figures 2.7 a and 2.8a-b. After initial yield, the plastic zone grows steadily as the mud pressure increases until it reaches the vertical side walls of the test box as shown in Figures 2.7 b and 2.8 c. All the plane strain analyses were terminated once the plastic zone touched the vertical side walls as shown in Figure 2.8d due to difficulties obtaining numerical convergence. However, the three dimensional analyses continued after the plastic zone reached the side walls, and a small (partial) zone of plastic failure was calculated to occur at the end of the borehole as shown in Figure 2.7c. A contiguous plastic zone was calculated to develop from the end of borehole extending to the soil surface at a higher mud pressure. The mud pressure producing this zone of unconfined plastic flow can be used to interpret the mud loss phenomenon observed from the small scale physical model tests, in which injected mud spouted with force from the end of the borehole to the surface as shown in Figures 2.9 and 2.10 (Elwood 2008).
Figure 2.11 presents values of mud pressure leading to mud loss versus the magnitude of overburden pressures acting at the level of the crown of the borehole from the numerical studies, the small scale tests, and calculations using the Delft solution (Keulen et al. 2001). The mud pressures in the three dimensional analyses needed to generate a plastic zone reaching the vertical walls of the box are marked as L3, and the mud pressures required to make the plastic zone grow up to the ground surface are shown as L1. The maximum mud pressures from the plane strain simulation considering the box with finite side boundaries are shown as L4. Values calculated using the Delft solution when the radius of the plastic zone is sufficient to reach the box sides (i.e. $R_{p, max}/R_0 = 6.4$) are denoted as L2, and values calculated using the Delft solution when the radius of the plastic zone reaches the surface are denoted as L5. The peak mud pressures measured from the small scale laboratory tests are shown as the black points denoted as T1 to T10. Direct comparisons of critical mud pressures are then possible, to explore differences between results from the small scale laboratory tests, the plane strain and three dimensional numerical analyses, and the Delft solution.

As indicated by L3 and L4, there is close agreement between the mud pressure calculated using three dimensional numerical analysis with finite boundaries and finite length borehole and the plane strain simulation considering plasticity reaching the side boundaries of the test box (i.e. a plastic radius, $R_p$ of 0.1375m). Inclusion of L2 in this comparison indicates that the Delft solution provides slightly unsafe calculations for plastic zone touching the side walls of the test box for this specific physical and geometrical condition, (i.e. distance to the side wall of the test box that is 3.2 times the borehole diameter). The calculated mud pressure values (L5) using the Delft solution when the plastic zone reaches the surface overestimated the measured peak mud pressure (T1 to T10) by 30% to 80%. The maximum discrepancy between calculation L2 and L4 based on use of the Delft solution and the finite element calculations is 7.5%, which is considered
acceptable, given the uncertainties in soil parameters and the greater ease of calculation. This conclusion is similar to that made by Moore (2005) who suggested that the Delft solution may work well in sand materials with isotropic geostatic ground stress conditions (initial horizontal stress equals to initial vertical stress) and a plastic radius ratio that is small (i.e. one-third of cover depth, $R_p=1/3H$).

Statistical analysis (i.e. the Mann-Whitney test) is performed to compare the difference between the 2 sets of data (experiment and finite element analysis). The Mann-Whitney test is the non-parametric equivalent of the independent “t-test” and is used for testing differences between data groups (Field and Hole 2003). More details of the Mann-Whitney test can be found in Appendix C. Table 2.4 shows the results from the Mann-Whitney tests comparing the experimental data and the three dimensional finite element results. According to the Mann-Whitney U test, the statistical difference between these groups is insignificant ( $U_1=53$, $U_2=47$, and $U_{critical}=45$).

A comparison of the data from the small scale laboratory model tests and the three dimensional numerical studies indicates that three dimensional simulations provide mud pressures that effectively represent the physical processes, with the exception of mud pressures recorded for tests T9 and T10 (these featured differences between calculated and measured pressure of 52kPa and 56kPa respectively). The causes of the significant discrepancies for tests T9 and T10 are unclear, but it may be that these are within the normal scatter of tests results (due to factors like sand compaction variability, and the technique of forming the borehole, sealing it with the expandable packer, and applying mud pressure).

Comparison between the maximum mud pressures observed during the small scale tests and theoretical studies using plane strain analyses like the Delft solution and finite element simulations consistently demonstrate that the measured pressures (or ground strength) are higher.
Potential explanations include the more complex three dimensional stress conditions associated with the finite length borehole, a geometry that has some features of spherical or balloon shaped expansion rather than the cylindrical expansion associated with plane strain analysis of a borehole of infinite length. This observation is supported by the shape of the mud saturated sand observed during the sand excavation after the testing reported by Elwood et al. (2007) and by the success of the three dimensional finite element analyses.

The finite distance to the side walls of the test box limited the growth of the plastic zone horizontally, and side wall friction may also have contributed to the higher values of measured mud pressure. A similar observation (i.e. measured pressure that was higher than expected) was also reported from several small scale model tests performed at Delft University of Technology. Those small scale model tests were carried out to simulate spherical expansion within well compacted fine sand (Keulen et al. 2001) rather than cylindrical cavity expansion (plane strain condition), therefore, the Delft test data cannot be included in the above comparisons.

One conclusion from these comparisons is that the three dimensional numerical analysis better simulates the real stress conditions and geometry of the small scale tests, and produces better estimates of maximum mud pressure than the plane strain analyses. However, these comparisons also indicate that the measured mud pressures from the small scale tests should not be used as effective representations of “field” conditions, since it appears that the walls of the test box influenced the response. However, given that the three dimensional finite element simulations are effective; these can be employed to explore maximum mud pressure where there are distant boundaries (i.e. large scale or field) conditions.
2.3.4 Numerical Simulations of Large Scale Tests (LS1 to LS4)

Large scale tests were carried out to minimize the boundary effects associated with the small scale tests, and numerical and theoretical studies were also performed for comparison with the large scale tests. The same modeling procedure, sand properties and element type were used as in the small scale model development. The model featured dimensions equal to the large scale tests (i.e. 0.78m cover depth, borehole diameter of 0.055m, and 0.5m length of open borehole space during testing). Plane strain conditions were also simulated with the cross section dimensions as the large scale tests. The same mesh techniques used in the analyses of the small scale model tests were employed, as shown in Figure 2.5.

Figure 12 shows the development of the plastic zone surrounding the borehole at three different levels of mud pressure for the large scale tests (LS1 to LS4). The development of the plastic zone around the borehole exhibits characteristics similar to those observed from the numerical simulations of the small scale tests at low mud pressure (i.e. 60kPa). The plastic zone grows steadily out vertically and horizontally as mud pressure increases, and eventually it touches the ground surface as shown in Figure 2.12 (c) when the internal mud pressure reaches 123.5kPa.

The comparison of the mud pressures including peak mud pressure measured from the large scale tests and maximum mud pressure calculated from both three dimensional and plane strain simulations ($R_{\rho, \text{max}}=2/3H$) and cavity expansion theory (the “Delft solution” and $R_{\rho, \text{max}}=H$) is listed in Table 2.5. Direct inspection of peak mud pressure values (LS1 to LS4) measured from the large scale tests indicates that the four large scale tests are fairly close given uncertainties from sand properties and test procedures, with an average of 87.5kPa. As indicated in Table 2.5, the predicted maximum mud pressure according to the cavity expansion theory (the “Delft solution”) is higher than the measure peak pressures (the average peak pressure measurement is 87.5kPa, while the calculated pressure is 219kPa, which is about 2.5 times higher). Three
dimensional finite element simulations overestimate the maximum mud pressure by 16.6%. The maximum allowable mud pressure from the plane strain simulation is 70.5kPa, which underestimates the average peak pressure by 23%. Direct comparison of these maximum mud pressure values indicate that both three dimensional and plane strain simulations match the measured peak mud pressure well for these specific tests, and these calculations are closer than those obtained using the “Delft solution”. However, there is a difference of 43% (30kPa) between the maximum mud pressure from the three dimensional model and the two dimensional plane strain model for these specific tests. Theoretically, if soil response is under plane strain condition for these tests, then the maximum mud pressure from the three dimensional model should be the same or almost identical to that produced by the two dimensional plane strain model.

One possible reason for the difference in mud pressure calculations obtained using three dimensional and plane strain finite element analyses is that the borehole is pocket shaped, and/or it has a short length (0.5m net space left for test), and this is explicitly modeled in the three dimensional finite element model, and it may not exhibit two dimensional plane strain response. The different element types used in the finite element analysis may also contribute to this difference. The effects on the maximum mud pressure of shape and length of borehole will be examined further by carrying out two additional finite element analyses in the next section. The calculated mud pressure values from three dimensional and plane strain finite element analysis could be considered to provide a safe prediction in these specific tests when the plastic zone reaches up a distance of two-thirds of the way to the surface providing a suitable safety factor is applied to account for the effects of uncertainties such as the influence of ground surface, the effect of tensile cracks formed during expansion of the borehole, the soil density, the interaction between mud and host sand, the influence of the packer pressure, the effects from side wall friction, and the disturbance of the sand during borehole excavation. However, only one group of tests (all tests having the same test parameters) were executed, and additional testing and
analyses are needed to evaluate further the performance of the ‘Delft solution’ and quantify its difference in estimating the maximum mud pressure.

2.4 THE EFFECTS OF BOREHOLE DIMENSIONS

As discussed above, the shape and length of the test borehole may have influenced the prediction of maximum mud pressure. In this section, two additional three dimensional models for cover depths of 1.0m and 1.2m are reported to investigate the effects of the length of the borehole and its shape on the maximum mud pressure. Borehole lengths of 0.5m, 1.0m, 1.5m 2.0m, 2.5m and 3.0m are examined, corresponding to L/D ratio of 5, 10, 15, 20 and 25. The relationship between the maximum mud pressure calculated by finite element modeling when the plastic zone extends two-thirds of the way up to the surface, and the total overburden pressure is given for the five different L/D values on Figure 2.13. The maximum mud pressures from the “Delft solution” are also presented to assess the performance of the Delft solution further.

As shown in Figure 2.13, the numerical simulations provide higher maximum mud pressures if the length of the borehole is short (i.e. L/D=5) than for longer borehole geometry (the maximum mud pressures decrease with increasing the L/D ratio). The results from the simulations of shorter borehole (i.e. L/D=5) is higher than longer borehole (i.e. L/D≥25) or plane strain simulations (i.e. L/D=∞) by 60%. As the L/D ratio is increased to 25, the effects of borehole length become negligible, or can be neglected in this specific test configuration.

The results from the three dimensional simulations are then almost the same as those from the plane strain simulations. The Delft solution consistently overestimates the maximum mud pressure. It is approximately at least two times higher than the results from the plane strain finite element simulations. It appears that the net length of the borehole which interacts with the mud should be at least 25 times the borehole diameter for this specific test configuration and soil.
material, if the test configuration is to provide results equivalent to long boreholes drilled in the field.

2.5 SUMMARY AND CONCLUSIONS

Control of maximum mud pressure to prevent mud loss during HDD in sand is recognized as an important issue during design and execution of HDD installation. Previous research studies examining maximum mud pressure and mechanisms of mud loss were either based on closed form solutions or finite element calculations, and these were reviewed as well as new experimental investigations involving laboratory tests to simulate the mud loss process. Three dimensional finite element analyses were carried out to establish effective numerical simulation processes, and to investigate the effects of the boundaries in the small scale tests. Direct comparisons and results from Mann-Whitney U tests indicated that three dimensional numerical analyses can effectively simulate the stress conditions and geometry of shear failure and mud loss in the small scale tests. However, the analyses also indicate that mud pressure measured from the small scale tests should not be used as effective representations of ‘field’ conditions. Finite element models (both three dimensional and plane strain) were also created to simulate the large scale tests. The maximum mud pressures from finite element analyses when the plastic zone extended up to two-thirds of the way to the ground surface ($R_{p,max}=2/3H$) match the measured peak mud pressure relatively well in this specific test configuration (with differences of 16.6% and 23%, respectively) compared with the predictions from cavity expansion theory (where peak mud pressure is overestimated by 160% to 190%). This comparison indicates that use of a plastic radius of two-thirds of the distance to the ground surface is able to minimize the influence of the free ground surface and tensile cracks observed in the large scale tests, at least for these specific tests. Plastic radius of two-thirds of the cover depth ($R_{p,max}=2/3H$) will be used consistently in the
subsequent finite element analyses assuming that it is also suitable for all other soil conditions and test configurations.

Cavity expansion theory (the Delft solution) overestimates the maximum mud pressure by more than 150% as indicated by comparisons with both large scale tests and numerical studies. A safety factor of at least 2.5 is suggested based on these comparisons if calculating maximum mud pressure using the Delft solution, however, this safety factor still depends on the field conditions such as soil characteristics, initial stress condition and so on, good engineering judgment and geotechnical experience are needed when designing pipeline installation using the Delft solution.

Finite element analysis also indicates that the length of the test borehole contributes to an unconservative solution for maximum allowable mud pressure. Analysis for a pocket shaped borehole could be used to predict the soil response during the pilot bore stage where stresses are three dimensional rather than two dimensional (the plane strain condition does not apply). Additional tests for longer borehole (i.e. L/D≥25) are suggested to understand the mud loss mechanisms more effectively and to quantify the maximum allowable mud pressure expected in the field.

REFERENCES


Table 2.1 Summaries of the small scale tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Total overburden pressure, kPa</th>
<th>Equivalent cover depth, m, $H = \sigma / \gamma_{sand}$</th>
<th>Measured peak mud pressure, kPa, $P_{max}$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>14.85</td>
<td>0.77</td>
<td>134.76</td>
<td>A</td>
</tr>
<tr>
<td>T2</td>
<td>18.85</td>
<td>0.98</td>
<td>142.67</td>
<td>A</td>
</tr>
<tr>
<td>T3</td>
<td>22.85</td>
<td>1.19</td>
<td>164.69</td>
<td>A</td>
</tr>
<tr>
<td>T4</td>
<td>22.85</td>
<td>1.19</td>
<td>186.49</td>
<td>A</td>
</tr>
<tr>
<td>T5</td>
<td>24.85</td>
<td>1.29</td>
<td>189.90</td>
<td>A</td>
</tr>
<tr>
<td>T6</td>
<td>26.85</td>
<td>1.40</td>
<td>183.83</td>
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<td>T7</td>
<td>26.85</td>
<td>1.40</td>
<td>219.31</td>
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<td>T8</td>
<td>30.85</td>
<td>1.60</td>
<td>260.51</td>
<td>A</td>
</tr>
<tr>
<td>T9</td>
<td>34.85</td>
<td>1.81</td>
<td>215.31</td>
<td>B</td>
</tr>
<tr>
<td>T10</td>
<td>46.85</td>
<td>2.43</td>
<td>285.92</td>
<td>C</td>
</tr>
</tbody>
</table>

Note: A: Mud flowed out from the end of the borehole (east wall of the test box). B: Mud flow out from the flow tube. C: Mud flowed out from side wall (south wall of the test box).

Table 2.2 Summaries of the large scale tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Total overburden pressure, kPa</th>
<th>Equivalent cover depth, m, $H = \sigma / \gamma_{sand}$</th>
<th>Measured peak mud pressure, kPa, $P_{max}$</th>
<th>Notes</th>
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<tbody>
<tr>
<td>LS1</td>
<td>13.65</td>
<td>0.78</td>
<td>94.60</td>
<td>D</td>
</tr>
<tr>
<td>LS2</td>
<td>13.65</td>
<td>0.78</td>
<td>81.10</td>
<td>D</td>
</tr>
<tr>
<td>LS3</td>
<td>13.65</td>
<td>0.78</td>
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</tr>
<tr>
<td>LS4</td>
<td>13.65</td>
<td>0.78</td>
<td>77.85</td>
<td>D</td>
</tr>
<tr>
<td>LS5</td>
<td>14.33</td>
<td>0.78</td>
<td>150.5</td>
<td>E</td>
</tr>
<tr>
<td>LS6</td>
<td>14.33</td>
<td>0.78</td>
<td>N/A</td>
<td>F</td>
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</table>

Note: D: ‘Sand alone’ test and mud flowed up to surface from tensile surface cracks. E: Sand and gravel layer system test, which 300mm thickness well graded gravel was compacted over the poorly graded sand. Mud flowed out from interface between gravel and sand. F: A failed test which mud flowed up to ground surface resulting from poor compaction near the corner of test pit, $[\gamma_{gravel} = 20.5 \text{kN/m}^3]$.  

Table 2.3 Model parameters used in the three dimensional and plane strain analyses

<table>
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<tr>
<th>Parameter</th>
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<tr>
<td>Borehole diameter, $D$</td>
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<tr>
<td>Sand unit weight, $\gamma_{sand}$</td>
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<td>Elastic modulus, $E$</td>
<td>25MPa</td>
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<td>Friction angle of the soil, $\phi$</td>
<td>$33.8^\circ$</td>
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<tr>
<td>Dilation angle of the soil, $\psi$</td>
<td>$15^\circ$</td>
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<tr>
<td>Cohesion, $c$</td>
<td>1.65kPa</td>
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<tr>
<td>Poisson’s ratio, $\nu$</td>
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<tr>
<td>Coefficient of lateral earth pressure at rest, $K_0$</td>
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### Table 2.4 Mann-Whitney U test between experimental and three dimensional modeling results

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<th>N</th>
<th>Median</th>
<th>Mean Rank</th>
<th>Sum of Ranks</th>
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<tr>
<td>Group 1 (experiment)</td>
<td>10</td>
<td>189.92</td>
<td>11.36</td>
<td>102</td>
</tr>
<tr>
<td>Group 2 (finite element model-3D)</td>
<td>10</td>
<td>192.0</td>
<td>11.64</td>
<td>108</td>
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</tbody>
</table>

\( U_1 = 53, U_2 = 47, U_{\text{critical}} = 45, \text{ there is no significant difference} \)

### Table 2.5 Comparison of the large scale tests results with the numerical simulation and theory calculation

<table>
<thead>
<tr>
<th>Measure Description</th>
<th>Value</th>
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<tr>
<td>Peak mud pressure measured from large scale tests (LS1 to LS4: sand alone test, LS4: sand and gravel)</td>
<td>90.5 kPa, 85 kPa, 75.1 kPa</td>
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<tr>
<td>Maximum mud pressure (M1: the Delft solution)</td>
<td>219.3 kPa</td>
</tr>
<tr>
<td>Maximum mud pressure (M2: 3D_FEA)</td>
<td>102.5 kPa</td>
</tr>
<tr>
<td>Maximum mud pressure (M3: 2D_FEA)</td>
<td>70.50 kPa</td>
</tr>
</tbody>
</table>

\(^*\): Peak mud pressure measured from sand and gravel large scale test (LS4)
Figure 2.1 Schematic diagram of the setup for the small scale tests
Figure 2.2 Typical diagram of mud loss and surface cracks in the large scale tests of (a) sand alone test (LS3); (b) sand and gravel layer system (LS5); (c) and (d) sand and gravel layer system (LS6)
Figure 2.3 Ground heave monitored by PIV in large scale test-LS3 (after Elwood 2008)

Figure 2.4 Likely mud loss mechanism causing surface cracking and mud flow in the large scale tests
Figure 2.5 Example meshes from the three dimensional and plane strain analyses
Figure 2.6 Borhole geometry after postconstruction (Knight et al. 2001)
Figure 2.7 Development of plastic zone from three dimensional simulation (T3 and T4) for mud pressures of (a) 60kPa; (b) 130.5kPa; and (c) 178.5kPa \(\gamma_{\text{sand}}=17.25\text{kN/m}^3, K_0=0.55, \phi=33.8^\circ, \\psi=15^\circ, D=0.045\text{m and }c=1.65\text{kPa}\)
Figure 2.8 Development of plastic zone from plane strain simulation (T3 and T4) for mud pressures of (a) 12.5kPa; (b) 40kPa; (c) 79.5.kPa; and (d) 100kPa [$\gamma_{sand}$=17.25kN/m$^3$, $K_0$=0.55, $\phi$=33.8$^0$, $\psi$=15$^0$, $D$=0.045m and $c$=1.65kPa]

Figure 2.9 Mud flow pathway observed in the tests (after Elwood 2008)
Figure 2.10  Typical mud loss phenomenon found in the tests (after Elwood 2008)
Figure 2.11 Comparison between maximum mud pressures measured during the small scale laboratory tests with results from cavity expansion theory (the ‘Delft solution’) and finite element calculations \( \gamma_{\text{sand}}=17.25 \text{kN/m}^3, K_0=0.55, \phi=33.8^0, \psi=15^0, D=0.045 \text{m} \) and \( c=1.65 \text{kPa} \)
Figure 2.12 The development of plastic zones from numerical simulation of the sand alone tests (LS3) for drilling mud pressures of (a) 50; (b) 102.5; and (c) 120.5kPa \([H=0.78\text{m}, D=0.055\text{m}, L=0.5\text{m}, \gamma_{\text{sand}}=17.5\text{kN/m}^3, K_0=0.55, \phi=33.8^\circ, \psi=15^\circ, \text{and } c=1.65\text{kPa}]\)
Figure 2.13 The maximum mud pressure from three dimensional calculations featuring different borehole lengths \([H=0.8; 1.0, 1.2\text{m}, D=0.055\text{m}, \gamma_{\text{sand}}=17.5\text{kN/m}^3, K_0=0.55, \phi=33.8^\circ, \psi=15^\circ, \text{and } c=1.65\text{kPa}]\)
Chapter 3

LARGE SCALE HYDROFRACTURING/BLOWOUT TEST

3.1 INTRODUCTION

Horizontal Directional Drilling (HDD) is a trenchless construction technique that has been in practice for more than a decade. Since it is a trenchless technology, HDD allows buried conduits (such as sewer pipes) to be installed in areas with specific construction demands at a significantly reduced financial or environmental cost compared to traditional cut and cover trench construction. However there are still aspects of the practice that are not fully understood, such as inadvertent mud return or mud loss from the borehole to the ground surface. This phenomenon usually resulting from tensile failure (hydrofracturing) or shear failure (blowout), is affected by the pressures of the drilling mud used to stabilize the excavated zone, but is not fully understood especially in relation to the surrounding soils. The consequences of mud loss include reductions in drilling efficiency due to the loss of drilling mud pressure, and ground heave causing substantial disruption of existing buried and surface infrastructure. New techniques have been developed to monitor drilling mud pressures in the field (e.g. Baumert et al. 2004), but further work is still needed to quantify the maximum allowable mud pressure that leads to hydrofracturing or blowout especially when drilling through sand.

State-of-the-art practice (the “Delft solution”) to calculate the maximum mud pressure was introduced based on the belief that mud loss is the result of unconfined plastic flow (shear failure) which grows around the borehole when the internal mud pressure is increased after initial yielding of the soil adjacent to the borehole. The maximum allowable mud pressure that soil can support is the pressure required to make the plastic zone reach up two-thirds of the distance to the surface (i.e. $R_{p, max}=2/3H$) as suggested by Van and Hergarden (1997). While the derivation of the
“Delft solution” is based on several assumptions, it appears to apply to homogeneous infinite soils which neglected the influence of the ground surface and gradients of stresses and soil strength with depth, and also assumes that the soil medium is in initial isotropic stress state, where initial horizontal stresses in the ground equal the vertical stresses (corresponding to a unit value for the coefficient of lateral earth pressure at rest, $K_0$). These assumptions do not always adequately capture the soil characteristics and stress states in the field (i.e. for case of non-homogeneous ground and $K_0 \neq 1$).

Given these limitations of the “Delft solution”, a number of recent studies have sought to assess current practice by examining mud loss mechanisms using numerical simulations which can consider complex soil and different stress conditions, and even consider complex situations involving the coupling of stress and seepage (Wang and Sterling 2006, Tang et al. 2002). A number of investigations using finite element analysis of mud loss and maximum mud pressure have been documented in recent years within cohesionless material (e.g. Duyvestyn 2004, Wang and Sterling 2006, Kennedy et al. 2006, Elwood et al. 2007). These finite element analysis studies indicate that the “Delft solution” seems to be an unsafe approach to calculate the maximum allowable mud pressure.

Wang and Sterling (2006) and Kennedy et al. (2006) considered the effects of the filtercake (FC) in the finite element model. The thickness of the filtercake is assumed to be around 2cm, and strength parameters are largely based on the frictional strength of its sand component, with consideration of the undrained response given on the assumptions that the infiltration of drilling mud considerably reduces the hydraulic conductivity of the sand. Elwood et al. (2007) carried out a series of standard laboratory tests to investigate properties of the filtercake samples obtaining from small scale simulations of mud loss from boreholes in the uniform test sand. He concluded
that there is little or no change in the physical characteristics of the filtercake compared to the sand except for the water content and hydraulic conductivity (from $3.2 \times 10^{-3} \text{cm/s}$ to $7.0 \times 10^{-6} \text{cm/s}$) at least for his tests which were performed rapidly. Based on this observation, the effects of the filtercake will be neglected in the finite element analyses that follow.

Chapter 2 in this thesis reviewed and presented the mud loss mechanisms within sand considered by past researchers. A series of finite element analyses were carried out to evaluate the results from those scaled laboratory tests and the “Delft solution”. The results of the finite element analyses indicate that finite element analysis can provide a good prediction of the maximum allowable mud pressure in those specific laboratory tests; however, the “Delft solution” overestimated the maximum mud pressure (by a factor of about 2.5). Unfortunately, only one group of large scale tests (having the same test dimensions) was performed. More tests are needed to understand further the mud loss mechanisms and quantify the maximum mud pressure.

Two additional large scale laboratory tests are presented here. These two tests have deeper cover depth and longer test borehole than the tests reported by Elwood (2008). Finite element analyses are also carried out to evaluate the large scale tests and to compare the results with the ‘Delft solution’. This chapter finishes by drawing conclusions and making suggestions for further work.

### 3.2 EXPERIMENTAL PROCEDURE

The experimental methods employed here to study mud loss and quantify the maximum mud pressure within sand are similar to those reported by Elwood (2008). However, different cover depths and the lengths of the test borehole are used in these tests. The internal borehole mud pressure and ground surface displacements were monitored. The details of test preparation and instrumentation are discussed in the following sections.
3.2.1 Scale of Test Pit and Pit Preparation

The pit is 4m long, 2m wide, and 2m deep as shown in Figure 3.1. The dimensions of the pit were altered to 2m length, 2m width, and 1.5m depth by Elwood using 6 piece of removable concrete retaining wall to reduce its size to limit the time and effort required to fill and empty the pit between each test. In the new tests, the following steps were taken:

- The 6 pieces of concrete retaining walls were moved 1m backward to set the pit dimensions to length of 3m, width of 2m and height of 1.5m.
- The location of the borehole was lowered by switching the steel plates used by Elwood to block the gap left between the concrete end walls. The borehole is about 30 cm above the base, leading to cover depth of 1.2 m.
- A layer of non-woven geo-textile (GT) was placed along the face of the concrete wall to seal any gaps between the blocks and to provide a uniform vertical wall surface.

3.2.2 Sand Backfill

The test sand was dumped into the pit, and compacted in 300mm thick layers to obtain a uniform density distribution. Compaction was performed using a light duty vibratory plates (Wacker Packers, WP1550) and a hand tamper as shown in Figures 3.2 and 3.3 respectively. These compaction methods can achieve relative density of at least 95% reported by Elwood (2008). The average density of the compacted sand is 1754kg/m³, based on measurements at 6 different locations using a nuclear density/moisture gauge (MC-1DR-PORTAPROBE) as shown in Figure 3.4. The same poorly graded, medium sand with initial water content of 3.5% used in the previous tests by Elwood et al. (2007) was again employed. After the required height was reached, the surface of the sand was carefully levelled, and a grid spaced at intervals of 0.5 m was painted on the surface of the sand, providing the control points for the surface displacement (heave) measurements.
3.2.3 Borehole Preparation and Instrumentation Installation

The borehole was drilled carefully using a hand auger as shown in Figure 3.5, attempting to minimize disturbance. The diameter of the test borehole is 0.055 m and the borehole was terminated at a position 2.5 m from the beginning of the borehole. The expandable packer manufactured and marketed by RocTest Canada (LP/102-190) shown in Figure 3.6 was inserted into the borehole about 1 m to seal the test borehole such that there was a minimum of 1.5 m of net space available interacting with the injected bentonite mud. The packer was inflated to prevent mud flow from the space between the packer and the surface of the borehole. The displacement pump shown in Figure 3.7 was connected to the inflated packer to inject the pressurized bentonite mud into the borehole. To measure the internal mud pressure in the borehole, the American Sensor Technologies (AST 4000 model) pressure transducer shown in Figure 3.8 was placed into the borehole through the packer. The pressure transducer was connected to the data acquisition system working with the software “Labtech” to collecting data as required.

Cubic wooden blocks, measuring 1.0 x 1.0 x 1.0 inch, were placed along the painted grid at 0.25 m intervals, staggered in a way so that the first line blocks should not block the view of the second line blocks. Two Cannon digital cameras (Eosdigital Rebel XT, 8.0 Megapixels) were placed with axes oriented horizontal and normal to each other to capture the vertical and horizontal movement of the target blocks. High resolution pictures (3458×2304 pixels and image size: 22.2×14.8 mm) were taken every 4 seconds and automatically stored into the connected PC. Two special lights were provided to minimize errors resulting from movement of shadows or shifting in light causing errors in target tracking. The specific configuration of those target blocks and the displacement measurement program set-up is shown in Figure 3.9, and a schematic diagram of the complete set-up for the large scale test is shown in Figure 3.10.
3.3 LARGE SCALE TEST 1

3.3.1 Maximum Mud Pressure

The total cover depth to the crown of the borehole was 1.15m (for a calculated total overburden pressure of 19.95kPa), and the length of the test borehole which interacts with the injected mud was 1.5m. The test was stopped due to depletion of the mud supply although no mud flowed up to the surface along the surface cracks at that time. The internal mud pressure measured by the ATS transducer was interpreted and plotted as shown in Figure 3.11.

As shown in Figure 3.11, the peak mud pressure in the borehole recorded in test 1 is 112.7kPa. The phenomenon reported by Elwood (2008), that is, ground surface cracks produced due to the borehole expansion under the increasing mud pressure and/or the dilation of the dense sand. Unfortunately, the test was stopped before the mud flowed up to the ground surface along the cracks due to loss of mud supply. However, inspection of the internal borehole pressure time history indicates that the internal mud pressure passed the peak mud pressure value and started to drop, which means the peak mud pressure for test 1 was obtained even without mud flowing to surface. The mud pressure time history can be divided into 3 stages. In stage 1, mud pressure in the test borehole accumulates with time, and this corresponds to the growth of the plastic zone around the borehole. After the mud pressure reaches a critical value corresponding to the occurrence of tensile cracks, mud pressure fluctuates in a small range denoted stage 2-fluctuation in Figure 3.11. This stage implies that plastic zone grows very slowly after the internal mud pressure passes the critical value, and further injection of mud creates and accelerates the propagation of the tensile surface cracks. The surface cracks extend deeper as mud volume increases, and the mud pressure attenuates during stage 3, when the bottom tip of the surface
cracks contacts the highest points of the plastic zone. This pressure history implicitly explains the hypothesized mode of mud loss from Chapter 2, which concluded that mud transport through sand results from a combination of plastic shear failure radiating out from the borehole and tensile cracks propagating down from the ground surface.

A series of finite element analyses were performed to simulate large scale test 1. The same procedures and sand properties described in Chapter 2 were used. Comparisons between the results from the experiments measured, the finite element analyses (plane strain and three dimensional simulations) and the ‘Delft solution’ are presented in Table 3.1.

The maximum mud pressure calculated from the Delft solution is 323.8kPa which overestimates the measured peak mud pressure by 187%. The maximum mud pressure obtained from these three dimensional and plane strain simulations are 125.5kPa and 118.5kPa respectively, which provides a good prediction of the measured peak mud pressure value (i.e. 112.7kPa) in test 1. Differences are just 11.6% and 5.1% respectively.

The development of the plastic zones calculated around the borehole is shown in Figure 3.12 at mud pressure of 40kPa, 125.5kPa and then 155.4kPa. Figure 3.13 shows the growth of the plastic zone surrounding the borehole from the plane strain simulation at the mud pressures of 50kPa, 118.4kPa, and then 143.5kPa. The development of the plastic zone surrounding the borehole for an isotropic initial stress condition (the coefficient of lateral earth pressure at rest, $K_0=1$) is shown in Figure 3.14. The shear strain contour and vertical displacement contour are shown in Figure 3.24 (a) and (b) respectively.

As indicated in Figures 3.12 and 3.13, the development of the plastic zone surrounding the test borehole from three dimensional and plane strain analyses is similar. When the mud pressure is
relatively low (e.g. 40kPa in the three dimensional simulation), the plastic zone exhibits sub-ellipse shape with the long axis oriented in the vertical direction. When the mud pressure in the borehole is increased, the plastic zone grows steadily, and eventually touches the ground surface. In these simulations, the plastic zone touched the bottom boundary due to the close distance (i.e. 0.295m) between the invert of the borehole and the base of the test pit, and this is verified by evidence from the post test excavation where the mud was found to have accumulated at the interface between the bottom of the sand and the base of the test pit. However, the development of the plastic zone when the soil has isotropic initial stressed exhibits a different shape. The plastic zone develops and grows in an approximately circular shape as shown in Figure 3.14 a and b. When the mud pressure approaches the maximum mud pressure, the plastic zone extends diagonally from the shoulder and haunches of the borehole and accelerates from two-thirds of the distance to the ground surface during an internal mud pressure increment of just 25 to30kPa as shown in Figure 3.14 c and d. The angle of the axis of the plastic zone for $K_0=1$ corresponds to an angle oriented at approximately $60^\circ$ to the horizontal.

Figure 3.15 presents calculations for the extent of the plastic zone. Results are shown based on the “Delft solution” and the finite element analysis for large scale test 1, where the latter considers coefficient of lateral earth pressure at rest values, $K_0$ of 0.55 and 1.0. Comparison of the extent of the plastic zone between finite element analysis for anisotropic stresses ($K_0=0.55$) and the Delft solution ($K_0=1$) indicates that $K_0$ has a significant effect on the prediction of maximum mud pressure. As shown in Figure 3.15, at small plastic radius ratio (the ratio of the furthest distance to a plastic point to the radius of the test borehole) the results from the Delft solution and finite element analysis for isotropic initial stresses ($K_0=1.0$) exhibits are almost identical (the difference is 2.5% when $R_p/R_0=10$). However, as mud pressure increases, the “Delft solution” overestimates mud pressures to produce the same value of plastic radius ratio. This discrepancy
likely caused by the assumptions associated with the Delft solution that the soil surrounding the borehole is homogenous isotropic, and soil of infinite size which neglects the influence of the ground surface and gradients of strength and stress with depth. Therefore, it can be concluded that all three of these limitations (i.e. $K_0$, the ground surface, and gradient of strength and stress with depth) associated with the Delft solutions contribute the overestimation of the maximum mud pressure.

### 3.3.2 Surface Displacement Measurements

Particle image velocimetry (Geo-PIV) was used to monitor the ground heave during the mud loss experiment, since traditional deformation measurement instrumentation (e.g. Linear Potentiometers, LP) would provide more limited measurement on the large surface area. In the test, two digital Cannon cameras were placed at right angle to take photographs of pre-installed targets. The Geo-PIV program developed by White and Take (2002), a MATLAB module which implements particle image velocimetry in a manner suited to geotechnical testing, was used to interpret these photographs. The Geo-PIV program gathers displacement data by tracking the fixed dimensional targets from a sequence of digital images, and the displacements are reported in horizontal (u) and vertical (v) directions in pixels.

Interpretation of the deformations is performed in two stages. Firstly, the displacement field between a pair of images is constructed. Since the distance between the cameras and the targets is not equal, the displacement field is obtained row by row; that is, several horizontal lines are defined on which all the targets are at approximately the same distance from the camera. The targets on the first row are tagged, and the displacement field on the line is interpreted from the subsequent images. The same procedures are repeated to obtain the displacement field for the rest of the rows. In the second step, a scale factor is calculated from each row by studying a specific
cubic target. The displacement fields obtained during this step are then converted from image space (i.e. coordinates in terms of pixels in the image) to objective-space (i.e. coordinates in terms of mm or m) using the calculated scale factors.

The surface displacement (heave) captured by cameras during test 1 is shown in Figure 3.16. The centre line of the test borehole is located around the vertical line (x=1.0). As we can see from Figure 3.16, the maximum ground heaves occur above the middle of the borehole, and the displacement tapered down to near zero with distance from the centerline of the borehole. The ground heave exhibits a “bell” shape, and the maximum ground heave of approximately 26 mm was observed in test 1. The ground heave volume of the displaced material was calculated based on the displacement data obtained from the Geo-PIV program. The total ground heave volume is 56.9 L. If it is assumed that the occurrence of tensile cracks on the surface corresponds with the maximum observed mud pressure, the surface displacement when the mud pressure is at its maximum value can be obtained from the Geo-PIV program as shown in Figure 3.17. The maximum ground heave is 11.2 mm, and the ground heave volume at this typical moment is 22.6 L. These indicate that 40% of the total ground heave occurs when the mud pressure reaches the maximum value, and further injected mud will act to deepen the tensile cracks, until mud flows out along those tensile cracks. Meanwhile, the surface displacement is accumulating to its maximum value.

The displacement from the two-dimensional and three-dimensional finite element analyses also exhibits a bell shape, but with the vertical ground displacement of 3 mm in this specific test as shown in Figure 3.24 (b). This is about 10% of the observed ground heave of 26 mm. This discrepancy likely results from the fact that the numerical model features mud pressure applied as a pressure on the boundary of the borehole, and it does not model any mud flow into and through the sand. Therefore, further investigation (more advanced model which can consider the mud
volume and seepage) is needed to accurately represent the surface displacement during directional drilling.

3.4 LARGE SCALE TEST 2

3.4.1 Introduction

The 6 pieces of concrete retaining walls were removed from the pit so that the whole pit (i.e. 2m high by 4m long by 2m wide) was available for testing, and the borehole was relocated to a position 75cm above the base of the test pit (after some simple modification of the steel end plates). The same procedure used in test 1 was repeated. The sand was compacted to a dense state with an average bulk density of 1781 kg/m³. The borehole was advanced used the hand auger, and terminated at a location of 3.0m. The packer was inserted into the test borehole, leaving a net length of 2.0m for interaction with the injected mud. The surface displacement and internal mud pressure were monitored using the same technology discussed in Section 3.3, however, only the maximum mud pressure will be discussed in the following section due to the poor quality of images for test 2.

3.4.2 Maximum Mud Pressure for Test 2

The total cover depth from the crown of the borehole is 1.18m giving a total overburden pressure of 21.0kPa. The mud pressure in the test borehole recorded by the pre-placed ATS pressure transducer is presented in Figure 3.18. Table 3.2 presents the comparison between the results measured from the test, numerical simulations, and the “Delft solution”.

As seen from the Table 3.2, peak mud pressure measured in test 2 is 121.5 kPa. The maximum mud pressure from the Delft solution \( R_{p,max} = 1.18 \text{m} \) is 338.5kPa. The percent differences between the Delft solutions and the peak mud pressure measured in test 2 are 180%. The
maximum mud pressure calculated from the three dimensional simulation and the plane strain simulation are 128.5kPa and 115.2kPa respectively. These results are closed to the peak mud pressure measured in test 2 with the percent differences of 5.8% and 5.0%, respectively.

To evaluate the efficiency of the Delft solution, the comparisons between all available results from large scale tests (though the length of the borehole likely influenced the test results for Elwood’s tests), the finite element analysis, and the Delft solution are provided in Figure 3.19. The comparisons indicate the Delft solution overestimates the maximum mud pressure by 160% to 190%. The results from finite element analysis are close to the peak mud pressure measured in these large scale tests with differences ranging from -5% to 11.6%. These finite element calculations are considered acceptable, given the experiment and numerical uncertainties such as soil properties, the influence of the packer and the formation of filtercake. This level of agreement between the test measurements and numerical calculations indicates that the choice of the maximum plastic radius to be two-thirds of the distance up to the surface is able to minimize the effects of free ground surface and tensile cracks formed during borehole expansion is reasonable, at least for the conditions associated with the large scale laboratory experiments.

Figure 3.20 shows comparisons of the extent of the plastic zone calculated using the Delft solution and the finite-element analysis for large scale test 2. As can be seen from Figure 3.20, at very small mud pressure values \( P/P_0 < 8 \), the development of the plastic zone calculated from the finite element analysis for an isotropic initial stress condition is close to calculations obtained using the Delft solution. As the internal mud pressure increases, the discrepancy in size of the plastic radius becomes more distinct. Together with the results from Figure 3.15, the growth of plastic zone calculated using finite element analysis for isotropic initial stress \( K_v=1 \) corresponds to pressures lower than these calculated using the Delft solution. The similar conclusion could be made that the overestimation of maximum mud pressure by the Delft solution is due to the
limitations of cavity expansion (neglecting the effect of $K_0$, the ground surface, and gradient of strength and stress with depth).

The development of the plastic zone surrounding the borehole is shown in Figure 3.21 at mud pressures of 60kPa, 128.5kPa and 158.5kPa. Figure 3.22 shows the growth of the plastic zone surrounding the borehole from the plane strain simulation at the mud pressures of 40kPa, 80kPa, 115kPa, and 143.5kPa. The development of the plastic zone surrounding the borehole for an isotropic initial stress condition (the coefficient of lateral earth pressure at rest, $K_0=1$) is shown in Figure 3.23.

As shown in Figures 3.21 and 3.22, the 2D and 3D calculations for the development of the plastic zone surrounding the test borehole exhibit similar behaviors. The plastic zone exhibits sub-ellipse shape with the long axis oriented in the vertical direction. When the mud pressure in the borehole increases to a critical value, the plastic zone grows steadily, and eventually touches the ground surface. The development of the plastic zone when the soil is in an isotropic initial stress condition exhibits a different shape. The plastic zone develops and grows in an approximately circular shape, as shown in Figure 3.23. When the mud pressure approaches the maximum mud pressure level, the plastic zone extends diagonally from the shoulder and haunches of the borehole and accelerates through the remaining distance to the ground surface over an internal mud pressure increment of just 25kPa.

3.5 SUMMARY AND CONCLUSIONS

Two large scale laboratory tests were performed within a poorly graded sand to study the mud loss mechanism and maximum mud pressure that the sand can support under different total overburden pressures. Plane strain and three dimensional finite element analyses were carried out
modelling the same dimensions as the large scale laboratory tests. The sand was modeled using the Mohr-Coulomb failure criterion, and considering dilation of the dense sand.

The results from the large scale laboratory tests, finite element analyses and state-of-the-art practice (“the Delft solution”) were compared. The maximum plastic radius is selected as two-thirds of the way to the ground surface \( R_{p,max} = \frac{2}{3}H \) as Van and Hergarden (1997) suggested, and this also provided a close prediction of the maximum allowable mud pressure in these specific tests based on the conclusions from Chapter 2. The comparison indicates that state-of-the-art practice provides unsafe maximum mud pressure, overpredicting maximum mud pressure by more than 100% (differences range from 160% to 190%). The finite element analyses match the results from the large scale laboratory tests more closely with differences ranging from -5% to 11.6%. These are considered acceptable given the uncertainties and other factors such as the interaction between the mud and the surrounding sand (the formation of ‘filtercake’ and the seepage of drilling mud).

Particle image velocimetry (Geo-PIV) was used to measure ground surface deformations. The measured peak ground heave was around 26 mm located above the middle of the borehole, and tapered down to near zero with distance from the centerline of the borehole. Surface cracks were found in the course of the tests caused by both the cavity expansion of the borehole under the internal mud pressure and dilation of the sand undergoing shear failure. If it is assumed that the occurrence of tensile cracks on the surface corresponds with the maximum observed mud pressure, the ground heave volume at this moment was 40% of total ground heave volume. Further injected mud accelerated the propagation of the tensile cracks, until these contact the plastic zone, and then mud can flow out along this complete flow path. These tensile cracks at the surface are believed to weaken the strength of the layer of soil near the surface, which lowers the ability of the soil to contain the pressurized drilling mud, and therefore reduces the maximum
allowable mud pressure. Unfortunately, the numerical model does not correctly predict the surface heave since the volume of mud and mud flow in sand did not take into account during the simulation.

Finite element analysis did not capture the tensile cracking phenomenon, since the Mohr-Coulomb model is used. However, predictions of maximum mud pressure from finite element analysis were effective when considering plastic radius of two-thirds of the borehole depth ($R_{p,max}=2/3H$) in these specific tests. It appears that this brings the zone of shear failure sufficiently close to the surface, so that finite element analysis based on shear failure can be used to estimate the maximum mud pressure.

Comparison of the extent of the plastic zone illustrated the limitations of the Delft solution (neglecting the effect of $K_0$, the ground surface, and gradient of strength and stress with depth). Different shape and rate of growth of the plastic zone occurs as a result of these complexities, and this explicitly influences the value of the maximum mud pressure.

A parametric study is needed to explore the effect of the lateral earth pressure coefficient, $K_0$, different soil properties (friction angle, dilation angle and cohesion) and different initial soil conditions (loose and dense state), as well as a more complete study of the affects of the borehole diameter and construction depth. Further research is needed to consider formation of the filtercake and its influence, as well the possible seepage of drilling mud through the sand (to provide credible calculations of surface movement). More complex modeling using ABAQUS could be pursed, which includes the effects of surface tensile cracks, and this is suggested for future investigation.
REFERENCES


Table 3.1 Comparison of the large scale tests results with the numerical simulation and theory calculation [Test 1]

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<td>Peak mud pressure measured from test 1</td>
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<td>Maximum mud pressure (M1: the Delft solution)</td>
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<tr>
<td>Maximum mud pressure(M2: 3D_FEA)</td>
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<tr>
<td>Maximum mud pressure(M3: 2D_FEA)</td>
<td>118.5 kPa</td>
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Table 3.2 Comparison of the large scale tests results with the numerical simulation and theory calculation [Test 2]

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<td>Peak mud pressure measured from test 1</td>
<td>121.5 kPa</td>
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<tr>
<td>Maximum mud pressure (M1: the Delft solution)</td>
<td>338.5 kPa</td>
</tr>
<tr>
<td>Maximum mud pressure(M2: 3D_FEA)</td>
<td>128.5 kPa</td>
</tr>
<tr>
<td>Maximum mud pressure(M3: 2D_FEA)</td>
<td>115.5 kPa</td>
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</table>
Figure 3.1 The test pit used in the large scale tests seen from above (after Elwood 2008)

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Figure 3.4 Nuclear density moisture gauge (MC-1DR-PORTAPROBE)
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Figure 3.14 Development of plastic zone from plane strain simulation for mud pressures of (a) 90kPa; (b) 170kPa; (c) 210kPa; and (d) 217.6kPa [$\gamma_{\text{sand}}=17.35\text{kN/m}^3$, $K_0=1.0$, $\phi=33.8^0$, $\omega=15^0$, $D=0.055\text{m}$ and $c=1.65\text{kPa}$]
Figure 3.15 The comparisons of the extent of the plastic zone calculated using the Delft solution and the finite-element analysis for large scale test 1 \( [\gamma_{\text{sand}}=17.35\text{kN/m}^3, K_0=0.5 \text{ and } 1.0, \phi=33.8^\circ, \psi=15^\circ, D=0.055\text{m}, \text{ and } c=1.65\text{kPa}] \)
Figure 3. 16 Total ground heave measured in test 1 [maximum heave of 25.8mm, total ground heave volume of 56.9L]
Figure 3.17 Ground heave corresponding to the moment of tensile cracks initiation in test 1 [maximum heave of 11.2 mm, ground heave volume of 22.6 L]
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Figure 3.20 The comparisons of the extent of the plastic zone calculated using the Delft solution and the finite-element analysis for large scale test 2 [$\gamma_{\text{sand}}=17.81\,\text{kN/m}^3$, $K_0=0.55$ and 1.0, $\phi=33.8^\circ$, $\psi=15^\circ$, $D=0.055\,\text{m}$, and $c=1.65\,\text{kPa}$]
Figure 3.21 Development of the plastic zone from three dimensional simulation for mud pressures of (a) 60kPa; (b) 128.5kPa; and (c) 158.5kPa [$\gamma_{sand}=17.81\text{kN/m}^3$, $K_0=0.55$, $\phi=33.8^0$, $\psi=15^0$, $D=0.055\text{m}$ and $c=1.65\text{kPa}$]
Figure 3.22 Development of the plastic zone from plane strain simulation for mud pressures of (a) 40kPa; (b) 80kPa; (c) 115kPa and (d) 145.5kPa [$\gamma_{sand}=17.81$ kN/m$^3$, $K_0=0.55$, $\phi=33.8^0$, $\psi=15^0$, $D=0.055$m and $c=1.65$kPa]
Figure 3.23 Development of the plastic zone from plane strain simulation for mud pressures of (a) 85kPa; (b) 120kPa; (c) 217kPa and (d) 264.5kPa \([\gamma_{sand}=17.81\text{kN/m}^3, K_0=1.0, \phi=33.8^\circ, \psi=15^\circ, D=0.055\text{m} \text{ and } c=1.65\text{kPa}]\)
Figure 3.24  Shear strain contour and vertical displacement contour for large scale test 2.

(a): shear strain contour for mud pressure of 158kPa; (b) Vertical displacement contour for mud pressure of 158kPa
Chapter 4

PARAMETRIC STUDY OF MAXIMUM MUD PRESSURE WITHIN SAND

4.1 INTRODUCTION

The potential for significant cost savings has increased the acceptance and use of trenchless construction techniques over the past 25 years. Traditional “cut and cover” installation methods are often abandoned in favor of these newer construction techniques that are considered to be less disruptive and more cost effective (Allouche et al. 2000). Horizontal Directional Drilling (HDD), an outgrowth of the oil well drilling technology, was reportedly first developed in the early 1970s. The first civil application of HDD involved the installation of approximately 180m of four-inch diameter steel pipe under the Pajaro river near Watsonvill, California (Ariaratnam et al 2001). Since 1979, the method has progressed to the point where long lengths of crossings and a wide variety of pipe sizes can be installed in soil or rock under natural or manmade obstacles. However, installation engineers and contractors still face problems with borehole instability such as blowout or hydrofracturing resulting in mud loss which can affect adjacent infrastructure or have an environmental impact when drilling under environmentally sensitive areas and densely populated urban areas.

Mud loss usually results from hydrofracturing (tensile failure) and/or unconfined plastic flow (e.g. blowout or shear failure) as a result of high internal mud pressure in borehole during drilling processes (pilot bole drilling or pipe pullback). To address these mud loss concerns, theoretical (e.g. Arends 2003), numerical (e.g. Mathew et al 2003, Wang and Sterling 2006, Duyvestyn 2004, Moore 2005, Kennedy et al. 2006) or experimental

\footnote{A version of this chapter is being prepared for submission to the Journal of Geotechnical and Geoenvironmetal Engineering (ASCE) for publication: Xia, H.W. and Moore, I.D. 2008. Parametric Study of Maximum Mud Pressure during HDD within Sand}
(e.g. Elwood et al. 2007, Elwood 2008) studies have been conducted to investigate the mud loss mechanisms and quantify the maximum allowable mud pressure which soil can support. The numerical simulations and large scale laboratory tests have focused on specific conditions (i.e. specific soil properties, borehole diameter, cover depth), and simplifications have been introduced in closed form solution such as the assumption that the soil is in an initially isotropic stress condition (i.e. coefficient of lateral earth pressure at rest, $K_0=1$), and/or that the influence of filtercake is neglected.

Chapters 2 and 3 described laboratory tests and finite element analyses conducted to study mud loss in sand, and to gain an understanding of the ability of state-of-the-art practice (the “Delft solution”) to estimate maximum mud pressure deduced from measured response, and therefore the validity of the modeling assumptions associated with that solution in these chapters. It was demonstrated that finite element analysis can provide close predictions (within 12%) of test measurements of the maximum mud pressure provided the pressure that produces the plastic radius to two-thirds of the distance way to the ground surface (i.e. $R_{max}=2/3H$) is used, while the Delft solution overestimated the maximum allowable mud pressure by more than 100% (differences ranged from 160% to 190%). Therefore, the objective of this chapter is to perform a numerical parametric study based on the procedures established in Chapters 2 and 3. The classical cavity expansion theory of Yu and Houlsby (1991) is firstly reviewed, and several comparisons between the results from finite element analysis and Yu and Houlsby (1991) are used to verify the effectiveness of the finite element analysis for the isotropic initial stress conditions and remote boundaries assumed in the Yu and Houlsby (1991) solution. A parametric study is then reported in the rest of the chapter, considering anisotropic initial stress and finite burial depth. Simplified design equations are developed by curve fitting to the finite element results, so that design calculations for maximum mud pressure can be readily made for a variety of sand strength, dilation angle, initial stresses, and burial depth. The chapter finishes with conclusions regarding the use of the computational results and suggestions for future research.
4.2 REVIEW OF CAVITY EXPANSION THEORY

This section examines the ability of ABAQUS to model cavity expansion problem in a Mohr-Coulomb material. Finite element analyses are compared with analytical solutions from Yu and Houlsby (1991). A brief summary of work completed by Yu and Houlsby (1991) is presented in the following subsection prior to the comparisons.

Cavity expansion theory was first developed for application to metal indentation problems (Bishop et al. 1945 and Hill 1950). It has been progressively refined by for example Gibson and Anderson (1961), Vesic (1972); Yu and Houlsby (1991); Yu (2000), and applied to a variety of Geotechnical problems by Fernando and Moore (2002), Yu and Cater (2002). Vesic (1972) extended Hill’s cavity solution to compressible soils by assuming that the soil volumetric strain is not zero but a finite value and presented an approximate solution for limiting internal pressure. He applied this solution to determine bearing capacity factors for deep foundations. Hergarden and Laugar (1988) extended Vesic’s analytical solution and used it to estimate the maximum mud pressure in cohesive and cohesionless soil materials during directional drilling. This contribution is referred to as the Delft solution and is the basis of current “state-of-the-art” design practice in HDD. Carter, Booker and Yeung (1986) presented an analytical solution for limiting pressures for cavity expansion in a non-associated Mohr-Coulomb material by assuming that a pseudo steady state deformation is approached at a very large deformation. Yu and Houlsby (1991) suggested that this assumption is not completely right, and the limiting pressure solution in Carter et al. (1986) can only be treated as an approximate solution. All these analytical solutions mentioned above are either large strain analysis in an incompressible soil or small strain analysis in a dilatant soil. Yu and Houlsby (1991) presented a new large strain analytical solution for the expansion of both cylindrical and spherical cavities in a dilatant elastic-plastic soil using the Mohr-Coulomb yield criterion and a non-associated flow rule. The limiting internal pressure
can be determined analytically with no additional assumptions about the mode of deformation.

In the Yu and Houlsby’s solution, the elastic soil properties are defined by Young’s modulus, $E$, Poisson’s ratio, $\nu$, and the plastic properties by the cohesion $c'$, and angles of friction, $\phi$, and dilation, $\psi$. The initial stress field is assumed to be uniform and isotropic, the soil medium is assumed to be spatially infinite, and longitudinal effects are ignored (i.e. cavity response is modeled using plane strain theory). A tension positive notation is adopted in Yu and Houlsby’s cavity expansion theory.

As the cavity pressure increases from its initial value, $P_0$, the response of the soil is at first purely elastic. Tangential and radial stresses within the elastic region are shown to be:

$$\sigma_r = -P_0 - (P - P_0) \left( \frac{R_0}{R} \right)^{1+k} \quad [4.1]$$

$$\sigma_\theta = -P_0 + \left( \frac{P - P_0}{k} \right) \left( \frac{R_0}{R} \right)^{1+k} \quad [4.2]$$

where $\sigma_r$ is the radial stress component, $\sigma_\theta$ is the tangential stress component, $R_0$ is the initial borehole radius, $R$ is the radius to the point at which the stresses are calculated, $P_0$ is the initial isotropic stress in the soil medium, $k$ is a parameter used to distinguish between cylindrical analysis ($k = 1$) and spherical analysis ($k = 2$), and $P$ is the current internal cavity pressure on the cavity wall.

The initial yield stress can be found when the stress conditions satisfy the Mohr-Coulomb yield criterion:

$$P = 2kG\delta + P_0 \quad [4.3]$$

After initial yield takes places at the cavity wall, a plastic zone will form around the cavity wall as applied internal cavity pressure $P$ increases. The stress distributions in the plastic
zone can be determined using the equilibrium equation and the yield condition, and are defined by:

\[ \sigma_r = \frac{\gamma}{\alpha-1} (Aa)^{k(1+\alpha)/\alpha} \]  \[4.4\]

\[ \sigma_\theta = \frac{\gamma}{\alpha-1} \left( \frac{A}{a} \right)^{k(1+\alpha)/\alpha} \]  \[4.5\]

The stress components in the elastic zone can be obtained from the equilibrium equation and the elastic stress-strain relations as follows:

\[ \sigma_r = -P_0 - B\alpha R^{1+k} \]  \[4.6\]

\[ \sigma_\theta = -P_0 + \left( \frac{\beta}{k} a \right)^{1+k} \]  \[4.7\]

where A and B are integration constants which can be determined by considering the continuity of stress components at the elastic-plastic interface:

\[ A = -\frac{(1+k)\alpha[Y+(\alpha-1)P_0]}{(\alpha-1)(k+\alpha)} \left( \frac{\xi}{\alpha} \right)^{k(\alpha-1)/\alpha} \]  \[4.8\]

\[ B = \frac{k[Y+(\alpha-1)P_0]}{(k+\alpha)} \xi^{1+k} \]  \[4.9\]

To account for large strain in the plastic zone, an explicit expression for the pressure-expansion relation can be derived by integrating the governing equation with the aid of a series expansion. The expression for the pressure-expansion relation is:

\[ \frac{a}{a_0} = \left( \frac{R^{-\gamma}}{(1-\delta)\beta/(\beta+m)} \right)^{\beta/(\beta+m)} \]  \[4.10\]

where \( \Lambda_1(x, y) = \sum_{n=0}^{\infty} A_n \), in which \( A_n^1 = y^n/n! \ln x \) if \( n=y \); and \( A_n^0 = y^n(x^n-y-1)/n!(n-y) \) otherwise. \( W = (1 + \alpha)(Y + (\alpha - 1)P)/2\alpha(Y + (\alpha - 1)P_0) \), and the elastic-plastic interface radius can be expressed as \( \frac{b}{R} = W^{\alpha/(\alpha-1)} \), where \( a = (1 + \sin\phi)/(1 - \sin\phi) \), \( \beta = (1 + \sin\phi)/(1 - \sin\phi) \), \( \delta = [Y + (\alpha - 1)P_0]/2(1 + \alpha)G \), \( Y = 2\cos\phi/(1 - \sin\phi) \), \( \gamma = a(\beta + 1)/(\alpha - 1)\beta \).
The limiting internal pressure can be obtained by setting $R/R_0 = \infty$ into the above equation and together with $\Lambda_1(W_\infty, \zeta) = \left(\frac{p}{\mathcal{Y}}\right)(1 - \delta)^{(\beta+1)/\beta}$ where $W_\infty = \frac{(1+\sigma)|\mathcal{Y}|+(\alpha-1)r_0}{2\sigma|\mathcal{Y}|+(\alpha-1)r_0}$.

4.3 PERFORMANCE OF THE FINITE ELEMENT MODEL

A series of finite element solutions were conducted for comparison with the Yu and Houlsby (1991) solution. The soil medium is assumed to be linear elastic and perfectly plastic with a failure surface defined by the Mohr-Coulomb criterion with a non-associated flow rule. Plane strain conditions are assumed to exist and the initial stresses are assumed to be uniform and isotropic where coefficient of lateral earth pressure at rest, $K_0=1.0$. To compare the stress distributions around the borehole with the calculated results from cavity expansion theory, the finite element model considers the uniformly distributed loads on all sides of the mesh with no fixed boundaries (boundary stresses of 18kPa are applied to simulate construction depth of 1m). Unit weight of soil was set to zero to remove any gradients with depth. The FE model featuring 8m height and 8m width is employed to minimize the effect of side boundaries on the stresses calculation. This stress state illustrates the limitations of analytical cavity expansion solutions, where initial stresses corresponds to finite burial depth even though boundaries are remote; it reinforces the importance of explicitly modeling the ground surface in the finite element calculations introduced later in the chapter when the parameters solution for maximum mud pressure is being developed. The stress development of the element at the crown of the borehole is also studied and compared with the elastic and plastic solution. The soil properties used in the comparison are summarized in Table 4.1.

The initial geostatic stress condition was achieved in the first step of the analysis using the ABAQUS commands “Initial Condition” and “Geostatic, which is used to verify that the initial geostatic stress field is in equilibrium with applied loads and boundary conditions”. In the following steps, the borehole was excavated and the internal borehole pressure was
increased from zero to the target borehole pressure (100kPa). The distributions of radial stress and tangential stress along the horizontal line (the borehole springline) from finite element analysis and from Yu and Houlsby (1991) are shown in Figure 4.1.

The maximum difference in radial and tangential stress obtained using the finite element analysis and the cavity expansion theory is 4.1%. These differences likely result because the finite element mesh does not have integration points along the horizontal axes of the cavity. Stresses calculated at the integration points nearby will not then exactly match the Yu and Houlsby solution. Given this issue, the difference between the analytical and finite element solutions is considered small (less than 5%), and it is concluded that ABAQUS is providing solutions that are very acceptable approximations of the exact solution.

The stress development at the crown of the borehole is also investigated and compared with the traditional elastic and plastic theory in Figure 4.2. It examines the stress development at the crown in sand, having a friction angle of 40°. The numerical solution is close to the plastic solution, but falls somewhat below the elastic solution shown in Figure 4.2. This small discrepancy results from shear failure during borehole excavation (release of initial stress around the borehole). As internal borehole pressure then increases, the soil element directly at the crown of the borehole exhibits elastic response, but some other points around the borehole remain in the plastic state, and this affects the stress at the crown and elsewhere around the borehole.

From this comparison it is concluded that the finite element analysis provides results that are close to the closed form solutions for uniform isotropic initial stress and remote boundaries. Together with the success of the finite element calculations of the laboratory experiments seen in Chapters 2 and 3, it is assumed that ABAQUS analysis can be used to study the cavity expansion problem and calculate maximum mud pressure for anisotropic initial stress ($K_0\neq1.0$) and finite borehole depth.
4.4 PARAMETRIC STUDY

4.4.1 Introduction

A set of numerical calculations were carried out using the plane strain finite element analysis, to investigate the effect of various parameters on the maximum mud pressure when drilling horizontal boreholes through sand. Analyses considering both non-associated and associated plastic flow were performed using the finite element program ABAQUS. The sand was modeled as a Mohr-Coulomb material, which follows Hooke’s Law in the elastic state. Parameters which can potentially influence the maximum mud pressure are Young’s Modulus, $E$, unit weight, $\gamma$, the frictional angle, $\phi$, dilation angle, $\psi$, the depth of burial, $H$, the borehole diameter, $D$, and the coefficient of lateral earth pressure at rest, $K_0$. These are studied in subsequent section, and normalization techniques established for the problem. Numerical convergence problems occurred if the dilation angle of the sand was set to zero ($\psi=0^\circ$). To avoid this numerical difficulty, the minimum dilation angle of the sand used is $0.1^\circ$ for solution featuring non-associated plastic flow. As demonstrated in Chapter 3, finite element models provide a close match to the measured peak mud pressure when the pressure corresponding to a maximum plastic radius of two-thirds of the distance to the ground surface is taken (i.e. $R_{p,\text{max}}=2/3H$), since the mode of mud loss inferred from the experiments involved growth of the plastic zone to this position, followed by propagation of tension cracks down from the ground surface. The parametric study is concluded, assuming that maximum mud pressure occurs when plastic radius reaches $2/3H$ for all cases (i.e. for a range of different initial stress conditions, soil strengths, borehole diameters, and cover depths).

4.4.2 Effect of Young’s Modulus, $E$

The stress-strain characteristics of soil are highly non-linear, and dependent on confining pressure. The effects of confining pressure have been studied by numerous researchers
(Kulhawy et al. 1969, Duncan and Chang 1970, Janbu 1963). In this section, two finite element models were created to investigate how Young’s Modulus affects the maximum mud pressure. Constant Young’s modulus (independent of depth) is used in the first model. Elastic modulus that increases with depth was also studied. Surface modulus $E_0$ of 10.8MPa and modulus gradient with depth of 2.2MPa/m were chosen, to provide an elastic modulus of 20MPa at the springline of the borehole when the burial depth is 4m.

The effect of these different soil moduli on maximum mud pressure is shown in Figure 4.3. Borehole diameter for these calculations is 0.4m, and cover depth of 4m is simulated. Three friction angles, 40°, 35°, and 30°, are considered, and an associated flow rule (i.e. $\psi/\phi=1.0$) is modeled. It can be seen in Figure 4.3 that the soil modulus has little influence on the maximum allowable mud pressure (the difference between upper and lower solutions is less than 1.5%). Since this comparison indicates that the maximum mud pressure is almost independent of elastic soil modulus, only one value of constant elastic modulus (20MPa) is employed in the rest of the parametric study.

### 4.4.3 Effect of Unit Weight of Sand, $\gamma_{sand}$

The effect sand unit weight on the maximum mud pressure is shown in Figure 4.4. Results are presented for two different unit weights, 18kN/m$^3$ and 22kN/m$^3$. The maximum mud pressures are normalized by the total overburden pressure, $\sigma_{total}=\gamma_{sand} \times H$, where $H$ is the distance from the crown of the borehole to the ground surface. It can be seen in Figure 4.4 that the variation of sand unit weight does not influence the ratio of maximum allowable mud pressure to the total overburden pressure. The difference between solutions for the two unit weight values is less than ±1.0%. It is concluded, therefore, that the total overburden pressure can be used to normalize the finite element results, and a single, constant value of total unit weight can be used to develop the parametric solution ($\gamma_{sand}=18$ kN/m$^3$).
4.4.4 H/D Normalization

Figure 4.5 shows the relationship between the mud pressure ratio, \( P_{\text{max}}/P_0 \), and dilation angle normalized by friction angle, \( \psi/\phi \) for three different model geometries featuring the same H/D ratio (i.e. borehole diameter and total cover depth are increased in proportion). It can be seen from Figure 4.5 that the results of maximum mud pressure ratio for three cases are almost identical (difference is less than 2%). This comparison indicates that the results calculated for one specific borehole diameter can be applied to cases with different borehole diameter but with the same dimensionless geometry (i.e. the same H/D ratio). One specific borehole diameter (0.6m) will be used in the subsequent finite element study, together with four different H/D ratios (5, 10, 15, and 20). The results from these specific geometries can then be applied to other problems with the same dimensionless geometries.

4.4.5 Effect of Dilation Angle, \( \psi \)

Figure 4.5 also demonstrate that pressure ratio, \( P_{\text{max}}/P_0 \), varies almost linearly as dilation angle increase from \( \psi = 0^\circ \) and \( \psi = \phi \). The slope of this linear line in Figure 4.5 is 0.7, which confirms that the results for \( \psi = 0^\circ \) and \( \psi = 0.1^\circ \) are almost identical (the difference is less than 0.05%). This demonstrates that setting dilation angle equals to 0.1 rather than zero is a reasonable approach to avoid numerical convergence problems mentioned earlier (when \( \psi = 0^\circ \)). It is also concluded that the results for all other angle ratios \( (0 < \psi/\phi < 1) \) can be interpolated from the results for those two limits \( (\psi/\phi \approx 0 \text{ and } \psi/\phi = 1) \). To simplify the calculation process, only two dilation angles \( (\psi = 0.1 \text{ and } \psi = \phi) \) will be used in the subsequent calculations.

Figure 4.6 shows the influence of sand dilation on the maximum mud pressure in each H/D case when the coefficient of lateral earth pressure at rest, \( K_0 \), is 0.36, and the maximum allowable mud pressure is normalized by the total overburden pressure. The results for full dilation (\( \psi=\phi \)) are plotted as solid lines, and the results for zero dilation (dilation angle set to
0.1°) are presented using dashed lines. Figure 4.6 clearly shows that sand with associated flow ($\psi/\phi=1$) is stronger than sand with non-associated flow ($\psi/\phi=0$). The maximum difference between maximum mud pressure for $\psi/\phi=1$ and $\psi/\phi=0$ is about 25%. It is always conservative to calculate the maximum mud pressure based on sand assumed to have zero dilation angle.

In general, dense sand starts to dilate when shear failure initiates, and the rate of volume increase reaches a maximum as the peak strength is mobilized. Following peak strength, the rate of dilation decreases to zero and the critical state strength is mobilized. Using a constant, nonzero dilation angle in finite element calculations could therefore provide somewhat unconservative solutions. It might be preferable to consider the angle of dilation as a function of density and stress level. However, this makes the finite element modeling both more complex and less practical. Therefore, the use of a fixed angle of dilation is considered an acceptable simplification.

Bolton (1986) suggested that dilation angle $\psi$ can be estimated using the equation $\phi = \phi_{crit} + 0.8\psi$. Most natural sand deposits have value of $\phi_{crit}$ greater than 30-33°, though it may be as low as 27° when the silt content is high. This indicates that the dilation angle of most natural sand is 15° or less (based on the assumption that the peak friction angle of sand is 45°). Schanz, T. and Vermeer P. A. (1996) suggested that a dilation angle of 15° is suitable for dense sand and this value was broadly suggested in geotechnical computer software like FLAC 3D and PLAXIS. Therefore, a single value of $\psi=15$° or an estimation using Bolton’s equation could be used, though further assessment based on measurements or judgment may be needed to choose a suitable value as a result of complexities in stress history or soil stratigraphy.
The effects of sand dilation on maximum mud pressure for anisotropic \((K_0=0.5, 0.75)\) and isotropic initial stress \((K_0=1.0)\) are presented in Figures 4.7 to 4.9 respectively. It can be seen in these figures that the effect of sand dilation is similar for various initial stress conditions. The maximum difference between full dilation \((\psi/\phi=1)\) and zero-dilation \((\psi/\phi=0)\) is 20\%, 15\%, and 18\% when \(K_0\) is 0.5, 0.75, and 1.0 respectively.

4.4.6 Effects of Coefficient of Lateral Earth Pressure, \(K_0\)

A series of finite element analyses were carried out in this section to study the effects of coefficient of lateral earth pressure at rest, \(K_0\), on maximum drilling fluid pressure. Jaky et al. (1948) introduced the following equation, \(K_0=1-\sin \phi\), to estimate the value of coefficient lateral earth pressure at rest for normally consolidated soils, where \(\phi\) is the effective stress friction angle obtained from a triaxial compression test with a confining pressure of approximately the same magnitude as the horizontal stress to be found in the ground. Several measurements from oedometer tests on granular soils as well as clays and silts in the compression range, lead to a good match with estimates from Jaky’s equation (Terzaghi et al. 1995). The coefficient of earth pressure at rest for overconsolidated soils, \((K_0)_{OC}\), may be estimated from (Schmidt 1966): \((K_0)_{OC} = (K_0)_{NC} \times (OCR)^{\sin \phi}\) where \((K_0)_{NC}\) is the value of \(K_0\) for the soil in the normally consolidated condition, OCR is the overconsolidation ratio, which can be determined from other sources (laboratory or in-situ tests), and \(\phi'\) is the friction angle in triaxial compression. This approximate formula has been shown to give reasonable estimates of \(K_0\) for many soils (Mayne and Kulhawy 1982).

For this parametric study, \(K_0\) values are estimated based on the Jaky’s equation, and solutions for \(K_0=1\) are selected to consider the case for lightly overconsolidated soils. Figure 4.10 presents the effect of \(K_0\) for friction angle of 40 degrees. As can be seen from Figure 4.10 that influence of initial stress on pressure ratio exhibits similar characteristic, the maximum mud pressures increase as the coefficient of lateral earth pressure at rest increases. That is logical.
because higher internal pressure is required to initiate the shear failure for soil in an isotropic initial stress, and the rate of growth of the plastic zones are also different (the growth speed of the plastic zone in anisotropic soil is faster than in isotropic soil) as reported in Chapters 2 and 3. Similar solution sets are presented in Figures 4.11 and 4.12 for friction angles of 35° and 30°. Clearly, similar conclusions can be drawn as these obtained from Figure 4.10.

### 4.5 APPROXIMATION FOR USE IN DESIGN

An approximate formula is now developed to facilitate design estimates of the maximum mud pressure within sand based on the results from the parametric study. A regression using an exponential growth equation was performed for the relationship between the pressure ratio and the H/D ratio for different $K_0$ values (0.36, 0.5, 0.75 and 1.0) when the friction angle is 30° and dilation angle is 0.1°. The format of the exponential growth equation is:

$$\left( \frac{P_{\text{max}}}{P_0} \right)_{30^\circ} = A + \log (B) \times B^{(H/D)}$$  \[4.11\]

where $A$ and $B$ are coefficients of regression. The four regressions are presented in Figure 4.13 with a minimum $R^2$ of 0.97.

Another two regressions were carried out to determine the values of $A$ and $B$ which are functions of coefficient of lateral earth pressure at rest, $K_0$ as shown in Figures 4.14 and 4.16. The simplified expression for $A$ and $B$ are:

$$A = 17.57 - \frac{12.9K_0}{K_0 - 0.084}$$  \[4.12\]

$$B = \frac{1.23K_0}{K_0 + 0.022}$$  \[4.13\]

To estimate the maximum mud pressure within the sand with a different value of friction angle and $K_0$, a scale factor is introduced which is a function of the friction angle and the coefficient of lateral earth pressure at rest, $K_0$. The equation for estimating the maximum mud pressure is:
\[(P_{\text{max}}/P_0)_{\phi} = (P_{\text{max}}/P_0)_{\phi=30°} \times F(\phi, K_0)\]  \[4.14\]

The scale factor function \(F\) in Figure 4.16 is expressed as:

\[F(K_0, \phi) = 1.46 \times e^{K_0^{-0.346}} \times \left(\frac{\phi-30°}{10°}\right) + \left(\frac{40°-\phi}{10°}\right)\]  \[4.15\]

Summarizing the above regression functions, the approximate solution used to estimate the maximum mud pressure is:

\[(P_{\text{max}}/P_0)_{\phi} = A + \log(B) \times B\left(\frac{H}{D}\right) \times F(\phi, K_0)\]  \[4.16\]

where \(A = 17.57 - \frac{12.9K_0}{K_0-0.084}\), \(B = \frac{1.23K_0}{K_0+0.022}\), and \(F(K_0, \phi) = 1.46 \times e^{K_0^{-0.346}} \times \left(\frac{\phi-30°}{10°}\right) + \left(\frac{40°-\phi}{10°}\right)\).

Figures 4.17 and 4.18 present comparisons of the finite element results and the interpolation results calculated from equation 4.16 for \(K_0=0.5\) and 1.0. The comparisons indicate that use of the approximate solution is able to predict the FE mud pressure ratio within 10%.

Moreover, equation 4.16 can be easily programmed or used in a spreadsheet, reducing the effort to calculate the maximum allowable mud pressure relative to state-of-the-art practice, and providing safer values or creating new finite element models for calculation. To illustrate how this approximate solution is used, an example calculation is presented in Appendix D.

Naturally, given the uncertainties associated with determination of sand properties, the variability of soil profile, and the simplifications associated with the finite element model relative to real field conditions (which can involved stratified profiles, for example). Sound engineering judgment and prudence are needed to ensure that the approximate equation is used effectively, and physical tests data may well be needed to support the design estimates of maximum mud pressure.
4.6 CONCLUSIONS

Mud loss as a result of tensile failure or unconfined shear failure has become a critical issue for engineers and contractors responsible for buried pipe installation using directional drilling through sand. Consequently, a series of finite element analyses were performed to study the factors influencing the maximum allowable mud pressure. The effects of Young’s modulus, unit weight of sand, friction angle, dilation angle, and coefficient of lateral earth pressure at rest were studied. Finite element results for maximum mud pressure when drilling through sand were presented corresponding to boreholes through sand with specific friction angles, dilation angles, burial depths and initial stress conditions. An equation approximating the finite element results was also developed to facilitate design estimates of the maximum allowable mud pressures.

The finite element analysis indicates that the maximum mud pressure is essentially independent of elastic modulus, and the effect of the unit weight of sand can be eliminated by normalizing the maximum mud pressure by the total overburden pressure. A specific borehole diameter (i.e. 0.6m) and four different total cover depths (i.e. 3m, 6m, 9m, and 12m) were considered in the course of the finite element study, and it was demonstrated that the results from these specific dimension configurations can be used to assess other configurations with the same H/D ratios. This study also demonstrated that the maximum mud pressure ratio, \( P_{\text{max}}/P_0 \), varies linearly between solutions for dilation angles of \( \psi = 0 \) and \( \psi = \phi \), so interpolation for all other \( \psi/\phi \) values can be undertaken based on these two limiting cases.

Finite element analysis considering high dilation angle for dense sand (associated plastic flow when dilation angle is equal to the friction angle) produced higher maximum mud pressure. The difference of maximum mud pressure using full dilation and zero-dilation is between 18% and 25%, which indicates that the use of a fixed dilation angle in the course of finite element calculations may provide a somewhat unconservative maximum mud pressure.
Considering the uncertainties in estimating dilation angle for dense sand, a constant value between 0 and 15 degrees can be used in the mud pressure calculation. Results capturing the effect of the coefficient of lateral earth pressure at rest, $K_0$, indicated that initial stress (isotropic or anisotropic) has a significant influence on the maximum mud pressure. Typical values of $K_0$ for sand range from 0.35 to 0.65 in the field, and current design practice (the “Delft solution”) overestimates the maximum mud pressure since the soil is assumed to be in isotropic initial stress($K_0=1$).

Approximate expressions were introduced to fit the results from the finite element calculations. Two simple equations are therefore available for use interpolating results for other friction and dilation angles. All results presented here are based on the assumption that the soil is isotropic, homogeneous, and that it responds under plane strain conditions. This does not capture the complexities of field conditions, so good judgment is necessary when selecting reasonable soil parameters, and using these equations. More laboratory and field test data is needed for future investigations to understand further the effects of soil parameters and initial stress on mud loss and evaluate the performance of the new design equations.

REFERENCES


Fernando, V. and Moore, I.D. 2002. *Use of cavity expansion theory to predict ground movements during pipe bursting*, Proc. of pipelines, American Society of Civil Engineers, Cleveland, OH.


Table 4.1 The soil properties used in the verification of finite element analysis and parametric study

<table>
<thead>
<tr>
<th>Property</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand unit weight, $\gamma_{\text{sand}}$ kN/m$^3$</td>
<td>18 kN/m$^3$</td>
</tr>
<tr>
<td>Diameter of borehole, $D$, m</td>
<td>0.2m 0.6m</td>
</tr>
<tr>
<td>Cover depth, $H$, m</td>
<td>1m 3m 6m 9m 12m</td>
</tr>
<tr>
<td>Friction angle of sand, $\phi$, $^\circ$</td>
<td>40$^\circ$, 35$^\circ$, 30$^\circ$</td>
</tr>
<tr>
<td>Dilation angle of sand, $\psi$, $^\circ$</td>
<td>$\psi=\phi$, $\psi=0.01^\circ$</td>
</tr>
<tr>
<td>Young’s modulus, $E$, MPa</td>
<td>10MPa 20MPa</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.33</td>
</tr>
<tr>
<td>Coefficient of lateral earth pressure at rest, $K_0$</td>
<td>Calculated from 1-sin$\phi$, 0.75, and 1.0</td>
</tr>
</tbody>
</table>

$^a$ Soil properties used in the verification of finite element model
Figure 4.1 Comparison of stresses distribution along horizontal axes between finite element analysis and Yu and Houlsby (1991) cavity expansion theory at borehole pressure of 100kPa 
\[\gamma_{\text{sand}}=18\text{kN/m}^3, K_0=1.0, \phi=35^0, \psi=0.1^0, D=0.2\text{m}, H=8.0\text{m}, \text{and } c=0.001\text{kPa}\]
Figure 4.2 Stresses development at the crown of the borehole with increasing internal borehole pressure; comparison between finite element analysis, elastic solution and plastic solutions (Kennedy et al. 2006) \([\gamma_{\text{sand}}=18 \text{kN/m}^3, K_0=0.55, \phi=40^\circ, \psi=0.1^\circ, D=0.2 \text{m}, H=1.0 \text{m}, \text{and } c=0.01 \text{kPa}]\)
Figure 4.3 The effect of Young’s Modulus, $E$, on the maximum mud pressure [$H=4.0\text{m,}$ $D=0.4\text{m,} \gamma_{\text{sand}}=18\text{kN/m}^3$, $K_0=0.5$, $\phi=40^\circ$, $35^\circ$, and $30^\circ$, $\psi/\phi=1.0$, and $c=0.05\text{kPa}$]
Figure 4.4 The effect of sand unit weight on the maximum mud pressure \([H=2.0\text{m}, D=0.4\text{m}, \gamma_{\text{sand}}=18 \text{and} 22 \text{kN/m}^3, K_0=0.5, \phi=30^0, 35^0 \text{and} 40^0, \text{and} \psi/\phi=1.0]\)
Figure 4.5 H/D normalization and dilation angle on the mud pressure ratio \( \gamma_{\text{sand}}=18 \text{kN/m}^3 \),

\( K_\phi=0.5, \phi=40^0 \), and \( \psi/\phi=0, 0.25, 0.5, 0.75, 1.0 \)
Figure 4.6 The effect of sand dilation on maximum mud pressure \([\gamma_{\text{sand}}=18\text{kN/m}^3, K_0=0.36, D=0.6\text{m}, \psi/\phi=1, \text{and } \psi=0.1^\circ]\)
Figure 4.7 The effect of sand dilation on maximum mud pressure [$\gamma_{\text{sand}}=18\text{kN/m}^3$, $K_0=0.5$, $D=0.6\text{m}$, $\psi/\phi=1$, and $\psi=0.1^\circ$]
Figure 4.8 The effect of sand dilation on maximum mud pressure \([\gamma_{\text{sand}}=18\text{kN/m}^3, K_0=0.75, D=0.6\text{m}, \psi/\phi=1, \text{and } \psi=0.1^\circ]\)
Figure 4.9 The effect of sand dilation on maximum mud pressure $[\gamma_{\text{sand}}=18\text{kN/m}^3$, $K_0=1.0$, $D=0.6\text{m}$, $\psi/\phi=1$, and $\psi=0.1^\circ]$
Figure 4.10 The effect of coefficient of lateral earth pressure, \( K_0 \), on maximum mud pressure

\[ \gamma_{\text{sand}} = 18 \text{kN/m}^3, \; D = 0.6 \text{m}, \; \phi = 40^0, \; \text{and } \theta = 0.1^0 \]
Figure 4.11 The effect of coefficient of lateral earth pressure, $K_0$, on maximum mud pressure

$[\gamma_{sand}=18\text{kN/m}^3, D=0.6\text{m}, \phi=35^0, \text{and } \psi=0.1^0]$
Figure 4.12 The effect of coefficient of lateral earth pressure, $K_0$, on maximum mud pressure

$[\gamma_{sand}=18\text{kN/m}^3$, $D=0.6\text{m}$, $\phi=30^\circ$, and $\psi=0.1^\circ]$
Figure 4.13  Best-Fit curve (exponential growth equation) for relation between dimension ratio, H/D and maximum mud pressure ratio, \( \frac{P_{\text{max}}}{P_0} \) [\( \gamma_{\text{sand}}=18 \text{kN/m}^3, D=0.6 \text{m}, \phi=30^\circ \), and \( \psi=0.1^\circ \)]
Figure 4.14 Best-Fit curve for coefficient of regression, A
\[ B = \frac{1.23K_0}{(K_0 + 0.022)} \quad R^2 = 0.98 \]

$\phi = 30^\circ$, $\psi = 0.1^\circ$

Figure 4.15 Best-Fit curve for coefficient of regression, B
Figure 4.16 The expression of F as a function of friction angle and $K_0$.

\[ F(K_0, \phi) = 1.46 \times e^{0.001K_0^{0.346}} \times \left(\frac{\phi - 30^0}{10^0}\right) + \left(\frac{40^0 - \phi}{10^0}\right) \]
Figure 4.17 Comparison between the results from finite element calculation and interpolation based on equation 4.12 for non-associated flow \([\gamma_{\text{sand}}=18\text{kN/m}^3, D=0.6\text{m}, \text{and } K_0 = 0.5]\)
Figure 4.18 Comparison between the results from finite element calculation and interpolation based on equation 4.12 for non-associated flow \([\gamma_{\text{sand}}=18\text{kN/m}^3, D=0.6\text{m}, \text{and } K_0 = 1]\)
5.1 INTRODUCTION

Trenchless technologies are gradually gaining recognition by municipal, provincial and federal authorities as providing valuable installation methods for the renewal, replacement and installation of buried services especially in densely populated urban areas. Horizontal Directional Drilling (HDD) as a main element of trenchless technologies (TT) has been developed to improve upon traditional “cut and cover” methods for installation and replacement of pipelines and other underground conduits. Motivating factors for the use of directional drilling installation include reduced cost, greater flexibility of operation, or greatly reduced impact on the adjacent environment, especially when crossing crowded urban areas, rivers, and sensitive or protected environments.

The directional drilling process includes three main stages: pilot hole boring, pilot hole reaming, and product pipe pull back (Allouche et al. 1998). High pressurized drilling fluid is introduced at the drill head from the beginning of the pilot boring process and is present in all stages of the drilling process. The main functions for the drilling fluid are to transport excavated drill cuttings back to the ground surface, to clean and cool the drill bit, to reduce sidewall resistance when the pipe is pulled back through the borehole, and to prevent the borehole from collapsing (Ariatatnam et al. 2000). Low mud pressure may cause borehole contraction or borehole collapse, and these can increase the friction resistance or block the...
borehole and make the pilot hole reaming and pull back operations more difficult. High mud pressures may result in mud flow from the borehole following tensile failure (hydraulic fracturing or ‘hydrofracture’) or shear failure (blowout) of the soil in the vicinity of the borehole. Engineers and contractors responsible for directional drilling projects need guidelines for limiting mud pressures to prevent these difficulties.

Past investigations of allowable mud pressure and two different mud loss mechanisms within purely cohesive clays are reviewed in this chapter. A new approximate closed form solution is subsequently developed to estimate the development of shear failure around the borehole and the maximum allowable mud pressures. The new approach is compared with the blowout solution reported by Arends (2003) as well as finite element analyses. The study concludes with a review of limiting mud pressures and the mechanisms controlling mud loss for both normally consolidated and overconsolidated clays.

5.2 LITERATURE REVIEW

This review examines different approaches in the literature to calculate the allowable mud pressure during directional drilling. Two mechanisms have been identified as having the potential to cause ground failure and mud loss. The first is associated with generalized shear failure in the soil and loss of confinement around the borehole, the so called “blowout” mechanism (Figure 5.1a). For clay, Van and Hergarden (1997) suggests that the maximum safe mud pressure to prevent blowout is the pressure that brings the zone of shear failure in the soil (the plastic zone) halfway to the ground surface. Alternatively, “hydraulic fracturing” (Figure 5.1b) or tensile failure of the soil can occur when the minor principal stress mobilizes the full tensile strength of the soil (which is conservatively assumed to be zero), followed by propagation to the ground surface, Kennedy et al. (2004a).
5.2.1 Shear Failure Mechanism

Carter et al. (1986) derived closed form solutions for the expansion of cylindrical cavities in an ideal cohesive-frictional material. The material is assumed to be isotropic but could be elastic and plastic. It obeys Hook’s law until onset of yield, which is determined by the Mohr-Coulomb criterion. The maximum pressure is taken as the pressure when large radial displacement or plastic expansion occurs. Mori et al. (1987) proposed the shear failure mechanism as the cause of hydraulic fracturing in soils. They believed that the horizontal and inclined fractures observed in laboratory experiments indicated shear failure. Panah and Yanagisawa (1989) concluded that fracturing in soil occurs when the stress of a soil element around the borehole intercepts the Mohr-Coulomb strength envelope based on the analysis of the laboratory tests results. Both Mori et al. (1987) and Panah and Yanagisawa (1989) assert that fracturing is initiated when shear failure is induced by the difference in radial and tangential stresses adjacent to the borehole. They indicate that maximum borehole pressure should be set as the pressure that initiates shear failure in the soil. These procedures lead to very safe solutions since they do not make use of the strength of the surrounding soil; significant increases in the mud pressure are required to expand the radius of the plastic zone up to the ground surface.

Lugar and Hergarden (1988) advanced the cavity expansion theory derived by Vesić (1972), and applied this closed form equation (called the “Delft solution”) to estimate the maximum allowable mud pressure in purely cohesive clays. A full scale experiment was also carried out to assess the effectiveness of the closed form solution and their finite element analysis. The test was carried out in a purely cohesive soil deposit with good results assuming that pressure is limited to 90% of the maximum pressure calculated by the “Delft solution”, whereby the plastic radius equals the depth of the soil cover. Van and Hergarden (1997) suggested that instead of limiting pressure to 90% of that maximum, internal borehole pressure should be reduced further to account for inconsistencies in the soil deposits and the need to estimate into
the strength of the native soil throughout the length of the drill path. Keulen (2001) extended
the work of Lugar and Hergarden by conducting laboratory experiments designed to
determine the effectiveness of their assumptions. Keulen determined that at great depths,
some form of cavity expansion or ‘ballooning’ of the borehole indeed occurs; similar to the
theories derived by Vesić where the flow was blocked to simulate the spherical cavity
expansion in a compacted soil (Keulen 2001). However, both Lugar and Hergarden (1988)
and Keulen (2001) report that at shallow depths, these theories do not always hold true and
some form of hydrofracture or tensile cracking occurred.

Arends (2003) reported on the closed form solution for maximum pressure developed by
Lugar and Hergarden (1998), and this solution was considered to be the “state-of-the-art”
practice to estimate the maximum mud pressure in clay and sand by the US Army Corps of
Engineers (USACE), (Carlos et al. 2002). The assumptions of this theoretical solution can be
summarized as follows:

- The borehole is axially symmetric, and the medium is homogeneous, isotropic, and of
  infinite size.
- The medium is approximated to have an isotropic initial stress condition (i.e. $K_0=1$) as
  shown in Figure 5.2a, it’s response is modeled as elastic until the onset of shear
  failure, defined using the Mohr-Coulomb failure criterion as a function of cohesion
  and friction angle.
- Increments of elastic deformation follow Hooke’s law.
- Elastic deformations in the plastic zone are neglected.
- Volume change in the plastic zone is assumed to be zero.

The maximum mud pressure was then expressed as:

$$p_{\text{max}} = p'_{\text{max}} + u$$  \[5.1\]
\[ P_{\text{max}} = \left( \sigma_0 (1 + \sin \phi) + c \cos \phi + c \cot \phi \right) \left( \frac{R_0}{R_{\text{p,max}}} \right)^2 + \frac{\left( \sigma_0 \sin \phi + c \cos \phi \right)}{G} \right)^{-\frac{\sin \phi}{1 + \sin \phi}} - c \cot \phi \] 

[5.2]

where \( P_{\text{max}} \) is the maximum allowable mud pressure, \( u \) is the initial in-situ pore pressure, \( P'_{\text{max}} \) is the maximum allowable effective mud pressure, \( \sigma_0' \) is the initial effective stress, \( \phi \) is the internal friction angle, \( c \) is the cohesion of the surrounding material, \( R_0 \) is the initial radius of the borehole, \( R_{\text{p,max}} \) is the maximum allowable radius of the plastic zone, and \( G \) is the shear modulus of the surrounding soils.

### 5.2.2 Tensile Failure Mechanism

Bjerrum et al. (1972) presented a closed form solution based on the theory of expansion of a cylindrical cavity. The fracture was assumed to initiate when the tensile effective circumferential stress exceeds the tensile strength of the material. Anderson et al. (1994) employed Bjerrum’s work, and proposed an analysis procedure which emphasises the nonlinearity of the stress-strain properties of soil, and the pore pressure changes in the soil induced by changes in total mean normal stress and shear stress. They tried to conclude that the failure pressure is a function of overconsolidation ratio (OCR), however, the relationship between the failure pressure and overconsolidation ratio is difficult to express due to the scattered pressure data, but it can be observed from the scattered data that the fracture pressure exceeds the overburden stress.

Kennedy et al. (2004a, 2004b) used finite element analysis to examine the response of an elastic-plastic soil during boring under a range of different drilling mud pressures. They concluded that the initiation of a tensile fracture occurs at the crown of the borehole where \( K_o < 1 \), and at the springline where \( K_o > 1 \). The limiting mud pressure causing fracture initiation is estimated by considering how the tangential stress in the soil at the crown or springline of the
borehole reduces down to zero from the initial compressive stress of $P_0$ (making the conservative assumption that the tensile strength of the soil is zero). Elastic continuum theory was used to determine that fracture initiates when mud pressures reach $P_{frac}$ given by:

$$P_{frac}/P_0 = (3K_0 - 1) \text{ for } K_0<1,$$  \[5.3a\]

and

$$P_{frac}/P_0 = (3 - K_0) \text{ for } K_0>1$$  \[5.3b\]

provided

$$P_{frac} < 2C_u$$  \[5.4\]

For soils where

$$C_u < P_{frac}/2$$  \[5.5\]

blowout can be expected before brittle fracture.

Kennedy et al. (2006) also provided upper and lower mud pressure limits where hydraulic fracturing can occur:

$$P_{lower} = \frac{1}{2} \cdot P_0(3K_0 - 1) - C_u$$  \[5.6a\]

$$P_{upper} = \frac{1}{2} \cdot P_0(3K_0 - 1) + C_u$$  \[5.6b\]

Murdoch (1993a, 1993b, 1993c) used laboratory tests to study the character of the fracture face and fracture propagation from a vertical borehole. Criteria were established based on conventional linear fracture mechanics, and fracture toughness was introduced as a factor to estimate the fracture pressure. While fracture toughness is very difficult to measure in practice, Murdoch (2002) demonstrated that the fluid pressures to propagate the fracture reduce significantly after fracture initiation. During directional drilling, the mud pressures can be sustained by the column of mud going back to the ground surface, so fracture propagation
is likely to be rapid. Prevention of fracture initiation is therefore the best approach to control hydrofracture and mud loss.

5.3 STATEMENT OF PROBLEM

Each of the two ground failure mechanisms reviewed above control the maximum allowable mud pressure for a specific range of circumstances. Moore (2005) used finite element analysis to investigate the performance of the current design practice, and found that it provides good estimates of the radius of the plastic zone if the coefficient of lateral earth pressure at rest \( K_0 \) is equal to or close to unity (i.e. \( K_0 > 0.85 \)), but also demonstrated that for lower \( K_0 \) the solution overestimates mud pressure. On the other hand, the Kennedy solution (2004a, 2004b) does not provide estimates of fracture pressures for all values of \( K_0 \) (it does not apply when shear failure occurs instead of tensile fracture). It is necessary to perform a study to investigate the maximum allowable mud pressure in those regions where lateral earth pressure coefficient \( K_0 \) is not covered by either Kennedy et al. (2004a, 2004b) or the current design practice which assumed \( K_0 = 1 \).

5.4 NEW APPROACH TO ESTIMATE MAXIMUM MUD PRESSURE

A new approximate closed form solution for shear failure in purely cohesive material has therefore been developed and is reported here through studying the traditional cavity expansion theory. The new approach can approximately consider the effects of coefficient of lateral earth pressure at rest, \( K_0 \) on maximum allowable mud pressure.

A number of simplifications and assumptions are made to facilitate development of the new approach:

(1) Plane strain conditions in the axial direction are assumed
(2) For a radius of borehole \( R_0 \), initial stresses around the borehole obey the Kirsch elastic solution for stresses around a circular hole in an elastic plate, under non-hydrostatic initial stress condition as shown in Figure 5.2 b.

(3) The soil exhibits linear elastic-perfect plastic behavior, governed by the Mohr-Coulomb shear failure criterion.

(4) The plastic zone around the expanded borehole is assumed to be axisymmetric (shaped like a doughnut) its outer radius is interpreted as the distance between the center of the borehole and the furthest shear failure point.

(5) For each point at the interface between the plastic zone and elastic region, the displacement has the same value as the furthest plastic point.

(6) Displacement is independent of the position angle, but is dependent in the maximum plastic radius \( R_{p, \text{max}} \).

(7) No volume change occurs in the plastic zone under undrained condition.

(8) The convention is adopted that compressive stresses and strains are positive, as is usual in soil mechanics.

The radial, tangential, and shear stresses in a plane of elastic material can be calculated relative to the known horizontal, vertical and shear stresses (\( \sigma_x, \sigma_y, \) and \( \tau_{xy} \), respectively; Obert and Duval 1967). When these stresses are applied to the problem of an infinite plate with a circular hole and applied stresses in the x- and y-directions of \( \sigma_x \) and \( \sigma_y \), respectively (Figure 5.3), Obert and Duval (1967) showed that the resulting stress distribution (radial stress, tangential stress, and shear stress) becomes:

\[
\sigma_r = \frac{1}{2} \cdot P_0 \cdot \left\{ (1 + K_0) \cdot \left( 1 - \frac{R_0^2}{R^2} \right) + (1 - K_0) \cdot \left( 1 - \frac{4R_0^2}{R^2} + \frac{3R_0^4}{R^4} \right) \cos 2\theta \right\} \tag{5.7a}
\]

\[
\sigma_\theta = \frac{1}{2} \cdot P_0 \cdot \left\{ (1 + K_0) \cdot \left( 1 + \frac{R_0^2}{R^2} \right) - (1 - K_0) \cdot \left( 1 + 3 \frac{R_0^4}{R^4} \right) \cos 2\theta \right\} \tag{5.7b}
\]
\[
\tau_{r\theta} = \frac{1}{2} P_0 \left\{ (1 - K_0) \cdot \left(1 + 2 \frac{R_i^2}{R^2} - 3 \frac{R_i^4}{R^4}\right) \sin 2\theta \right\}
\]  

[5.7c]

For soil with specific strength and particular geometry, shear failure initiates at the crown of the borehole once internal mud pressure reaches a limit. The plastic zone grows as the mud pressure increases, and the furthest shear point in the plastic zone appears over the crown when lateral earth pressure at rest \(K_0\) is less than unity. Therefore, this approximate closed solution considers only those stresses that lie above the crown:

\[
\sigma_r = \frac{1}{2} P_0 \cdot \left\{ (1 + K_0) \cdot \left(1 - \frac{R_i^2}{R^2}\right) + (1 - K_0) \cdot \left(1 - \frac{4R_i^2}{R^2} + 3 \frac{R_i^4}{R^4}\right) \right\} + P_i \cdot \frac{R_i^2}{R^2}
\]  

[5.8a]

\[
\sigma_\theta = \frac{1}{2} P_0 \cdot \left\{ (1 + K_0) \cdot \left(1 + \frac{R_i^2}{R^2}\right) - (1 - K_0) \cdot \left(1 + 3 \frac{R_i^4}{R^4}\right) \right\} - P_i \cdot \frac{R_i^2}{R^2}
\]  

[5.8b]

At the internal wall, the stresses become \(R=R_0\):

\[
\sigma_r = P_i
\]  

[5.9a]

\[
\sigma_\theta = (3K_0 - 1)P_0 - P_i
\]  

[5.9b]

Before initial yielding occurs, hydraulic fracturing will develop if the tangential stress becomes negative \(\sigma_\theta < 0\) based on the conservative assumption that the tensile strength of purely cohesive material is zero. The limiting mud pressure is then \(P_{\text{max}} / P_0 = (3K_0 - 1)\), that is the solution of Kennedy et al. (2004a) for maximum allowable mud pressure.

Otherwise, a plastic zone develops around the borehole. Shear failure and plasticity develops in accordance with the Mohr-Coulomb failure criterion:

\[
\sigma_r - \sigma_\theta = 2C_u
\]  

[5.10]

Based on this failure criterion, the critical mud pressure for initial yielding at the crown of the borehole \(P_i\) is:
At the interface between the plastic zone and the elastic region, the stress distributions are given by:

\[ \sigma_r = P_e \]  \hspace{1cm} [5.12a]

\[ \sigma_\theta = (3K_0 - 1)P_0 - P_e \]  \hspace{1cm} [5.12b]

where \( P_e \) is used to denote the radial stresses across the interface. In the plastic zone, the stresses also satisfy the equilibrium equation

\[ \frac{\partial \sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \]  \hspace{1cm} [5.13]

Substitution of Equation 5.11 into Equation 5.14 and integration provides

\[ \sigma_r = -2C_u \ln r + C. \]  \hspace{1cm} [5.14]

The integral constant \( C \) in Equation 5.14 is determined by the boundary condition at the internal wall of the borehole.

The radial stress above the crown in the plastic zone can then be expressed as:

\[ \sigma_r = P_i + 2C_u \ln\left(\frac{r}{r_i}\right) \]  \hspace{1cm} [5.15]

At the interface between the plastic and elastic zones, the interface is in equilibrium, so:

\[ \sigma_r^e = \sigma_r^p \text{ at } R = R_e \]

where the superscripts \( e \) and \( p \) denote the elastic and plastic regions respectively.

Radial pressures at the interface \( P_e \) can then be determined considering the boundary condition:

\[ P_e = C_u + \frac{1}{2}(3K_0 - 1)P_0 = P_i + 2C_u \ln\left(\frac{R}{R_i}\right) \]  \hspace{1cm} [5.16]

This equation can be rewritten as:
\[ P_i = C_u + \frac{1}{2} (3K_0 - 1)P_0 - 2C_u \ln \left( \frac{R_i}{R_e} \right) \] [5.17]

At the interface between the plastic zone and the elastic region, the displacement can be obtained using elasticity:

\[ u_{(e)} = \frac{(P_i - P_0) \cdot R_e^2}{2GR_e} = \frac{\left( C_u + \frac{1}{2} (3K_0 - 1)P_0 - P_0 \right)R_e}{2G} = \frac{\left( C_u + \frac{3}{2} (K_0 - 1)P_0 \right)R_e}{2G} \] [5.18]

No volume change (\( \Delta V = 0 \)) is expected in the plastic zone during the deformation for this purely cohesive material responding in an undrained condition, so that:

\[ \pi (R^2_e - R^2_i) = \pi (R^2_e - u^2_{ue}) - R^2_0 \] [5.19]

where:

- \( R_e \) plastic radius,
- \( R_i \) current borehole radius,
- \( u_{ue} \) displacement at the interface of the plastic and elastic zones, and
- \( R_0 \) initial borehole radius.

Simplifying this equation and neglecting the higher order terms gives:

\[ \frac{R_i}{R_e} = \left( \frac{R^2_e}{R^2_i} + \frac{(C_u)}{G} \right)^{\frac{1}{2}} \] [5.20]

Now, substitution of Equation 5.20 into Equation 5.17 provides:

\[ P_i = C_u + \frac{1}{2} (3K_0 - 1)P_0 - C_u \ln \left( \frac{R^2_i}{R^2_e} + \frac{(C_u)}{G} \right) \text{ for } K_0 < 1 \] [5.21]
For the case of coefficient of lateral earth pressure $K_0 > 1$, the maximum plastic zone develops adjacent to the springline. A similar approach can be used to obtain the equivalent limiting pressure equation:

$$P_i = C_u + \frac{1}{2} (3 - K_0) P_0 - C_u \ln \left( \frac{R_0}{R_e} \right)^2 + \frac{C_u}{G}$$

for $K_0 > 1$ \[5.22\]

### 5.5 FINITE ELEMENT EVALUATION

The effectiveness of Equations 5.21 and 5.22 for estimating the relationship between maximum plastic zone and applied mud pressure has been evaluated using AFENA (Carter, 1992), a non-linear finite element analysis that can consider non-hydrostatic initial geostatic stress conditions (i.e. $K_0 \neq 1$).

The problem is modeled using an assumption of plane strain and undrained soil parameters. The analysis employs six-node triangular elements. Shear failure and the elastic-plastic constitutive response are modeled using the Mohr-Coulomb failure criterion, total stresses, and undrained strength and deformation parameters. Figure 5.4 shows the finite element mesh and scaled sample of applied mud pressure through node reaction on the cavity wall.

Mud pressure is applied to the inside surface of the borehole in a number of load increments, and the growth of the plastic zone around the borehole is examined as the internal applied mud pressures are increased. A borehole of 0.4 m diameter at depth of 2 m and 4 m is studied for 6 different lateral earth pressure coefficients at rest (i.e. $K_0 = 0.75, 0.85, 1, 1.2, 1.4, 1.6$).

Parameters used in the analysis are listed in Table 5.3. Mud pressures are adjusted to simulate an increasing column of mud over the borehole (local gradient within the borehole is kept fixed).
Figure 5.5 shows the development of the plastic zone around the borehole as applied mud pressure increases for coefficient of lateral earth pressure at rest that is smaller than unity (i.e. $K_0=0.85$). As mud pressure increases, the plastic zone grows around the borehole after initial yield. When the mud pressure is about 1.5 times the initial stress, shear failure initiates around the borehole. With further increases in mud pressure, the plastic radius grows steadily around the borehole. Up to mud pressures of 3 times the initial ground stress, the maximum plastic radius occurs at a point directly over the crown. Once mud pressure reaches 3.5 times the initial ground stress, the point of shear failure which is furthest from the cavity deviates from above the crown and accelerates towards the ground surface.

Figure 5.6 shows the development of the plastic zone around the borehole as applied mud pressure increases for coefficient of lateral earth pressure at rest that is larger than unity (i.e. $K_0=1.4$). The development of the plastic zone around the borehole exhibits characteristics similar to the case for $K_0 = 0.85$. After initial yield, the plastic zone grows steadily as the mud pressure increases, and the maximum plastic zone appears adjacent to the springline. Once the mud pressure $P_i$ exceeds $3.5 \, P_0$, the furthest shear failure point moves from opposite the springline and rapidly extends up to the ground surface.

Figures 5.7 and 5.8 show the radius of the zone of shear failure $R_{p,\text{max}}$ normalized using initial borehole radius $R_0$, versus the mud pressure $P_i$ normalized using $P_0$, the mean initial ground stress at the crown of the borehole. Results are given for cover depths of 2 m and 4 m and different $K_0$ values.

Table 5.2 presents comparisons of results from the three different solution methods where the plastic zone extends to three different distances away from the borehole:

- When it extends halfway to the ground surface, $R_{p,\text{max}}/R_0 = 5$ (the design limit recommended by Arends, 2003)
When it extends right to the ground surface, $R_{p,max}/R_0 = 11$ (the point where blowout would be expected)

When the closed form solutions indicate that the plastic zone diverges rapidly (since the finite element solution explicitly models the ground surface, it cannot provide solutions for plastic zone reaching beyond that point); $P_\infty$ is also called the ‘limiting mud pressure’.

Figure 5.7 indicates that the new solution works better than the Delft equation at lower load levels (where the mud pressure is less than 80% of the limiting mud pressure), providing improved estimates of maximum plastic radius as a function of borehole mud pressure wherever earth pressure coefficient $K_0$ is less than 1 (values of 0.75 or 0.85). When the earth pressure coefficient at rest $K_0$ is equal to unity, the new approximate closed form solution provides results identical to the Delft equation, and they both fit the finite element results well with a maximum difference of 1.2% when the plastic zone extends up to the ground surface.

For the case where $K_0$ is larger than unity, the new solution also provides a more accurate estimate of the relationship between mud pressure and the extent of the plastic zone (with a small difference of 2.5% compares to the large difference of 20% between the results from finite element analysis and the Delft solution). The new solution also avoids the excessive (unsafe) estimates of mud pressure that arise from the Delft equation once plastic radius extends beyond 2/3 of the distance towards the ground surface (for both $K_0<1$, and $K_0>1$).

This deviation is due to the assumption of cavity expansion theory which neglects the influence of $K_0$, the free ground surface, and gradient of strength and stress with depth. At 2/3 of the distance towards the ground surface, the Delft solution only differ 0.5% from the finite element analysis, which indicates the most important of these factors is $K_0$ for clay.
The comparisons in Table 5.2 also demonstrate that the new solution gives lower (more conservative) values of limiting mud pressure than the Delft solution when \( K_0 = 0.75 \) or less. For \( K_0 > 0.75 \), both solutions provide essentially the same limiting mud pressure (when the plastic radius diverges). The pressures obtained using the new solution are up to 3% conservative relative to the finite element results. For cases involving \( K_0 < 1 \), the Delft solution overestimates mud pressures by up to 15%. Stress factor is defined as a measure of the additional pressure capacity available once the plastic zone stretches halfway to the ground surface. Values of between 1.2 and 1.29 (a function of \( K_0 \)) imply 20% to 30% reserve capacity once the recommended mud pressure limit is reached.

Figure 5.8 shows that for deeper boreholes (i.e. depth of 4 m), the new solution also produces estimates of the radius of the plastic zone that are more consistent with the finite element calculations. For problems with anisotropic stress \((K_0 \neq 1)\), the results from the new solution are very similar to the finite element results when applied mud pressure is less than 85 percent of the limiting pressure. However, once mud pressures exceed that pressure, the results from both closed form solutions do not fit the finite element results. This may be due to the effect of the ground surface, since it is explicitly considered in the finite element calculation. In each case, the new solution gives exactly the same results as the Delft equation when \( K_0 = 1 \), and they fit the finite element results well. For the case of \( K_0 > 1 \) and \( c_d/P_0 = 0.75 \), the new solution also performs better than the Delft equation.

Figures 5.9, 5.10 and 5.11 show the relationship between maximum allowable mud pressure and lateral earth pressure coefficient \( K_0 \). Each figure provides maximum allowable mud pressure based on the tensile failure (Equation 5.4, Kennedy et al., 2004) and blowout cases (Equations 5.21 and 5.22), which are collectively denoted the “Queen’s solution”. The range of \( K_0 \) values are marked where these blowout or hydrofracture cases apply. Figures 8, 9 and
For the specific material in Figure 5.10 (i.e. $c_u/P_0=0.5$), the lateral earth pressure coefficient at rest (and therefore the stress history of the clay deposit) controls whether hydraulic fracturing or blowout is expected. When the lateral earth pressure coefficient at rest is less than 0.67 or larger than 2.0, tangential stresses at the crown or springline, respectively, become negative (tensile), and hydraulic fracturing is expected to be the main cause of mud loss during drilling. In this range, maximum allowable mud pressure is estimated using the approach of Kennedy et al. (2004a, 2004b), Equations 5.21 and 5.22.

However, when the lateral earth pressure coefficient at rest is between 0.67 and 2.0, shear failure occurs before the tangential stresses become tensile, and the tangential stress subsequently increases with increasing mud pressure (discussed in detail by Kennedy et al, 2004). Hydraulic fracturing is not then expected to occur. Rather, blowout develops at mud pressures that can be estimated using the new approximate closed form solution. Except when $K_0=1$, the Delft equation overestimates the allowable mud pressures.

The maximum allowable mud pressure and the controlling failure mechanism are dependent on the initial stress condition, the undrained soil shear strength, and the overburden pressure. Kennedy et al, (2004b) developed criteria for judging which type of failure criterion is critical, as depicted in Figure 5.12 for a given value of $K_0$ and a specific strength ratio (i.e. $c_u/P_0$, the ratio of undrained shear strength to total vertical stress). As presented in Figure 5.12, soil elements at certain depths having specific undrained shear strengths and initial horizontal ground stresses defined by $K_0$, have mud loss mechanism dependent on the strength ratio defined by the dashed line or the dashed-dot line. Hydrofracture is critical at strengths
greater than those defined by these lines, and unconstrained shear failure (or blowout) is the mechanism controlling mud loss at lower shear strengths.

The strength ratio $C_u/P_0$ for different type of soil materials has been well-documented based on numerous experimental studies. Ladd et al. (1971) reported that the ratio of shear strength to effective vertical stress ($c_u/P_0$) is between 0.2 and 0.28 for normally consolidated (NC) clays, between 0.5 and 1.0 for lightly overconsolidated (LOC) clays, and may be larger than 1.2 for heavily overconsolidated (HOC) clays. Assuming that the groundwater table is located at the ground surface and using typical values of soil unit weight, the values of $c_u/P_0$ (total overburden stress) will be between 0.1 and 0.14 for NC materials, from 0.25 to 0.5 for lightly overconsolidated soils, and over 0.6 for heavily overconsolidated clay. Now, typical values of $K_0$ for NC clay range from 0.5 to 0.6, for LOC clay the $K_0$ value is close to unity, and for heavily overconsolidated clay, $K_0$ can be 3 or more. It appears, therefore, that NC and LOC clays are most susceptible to blowout. Hydraulic fracturing is expected in HOC clay once $K_0$ exceeds 1.8, while blowout is expected at lower $K_0$ values.

5.6 SUMMARY AND CONCLUSIONS

Control of maximum allowable mud pressure to prevent mud loss during directional drilling is recognized as an important issue during project design and construction. Two mechanisms causing ground failure have been reviewed, namely the tensile failure and shear failure mechanisms. A new approximate closed form solution has been reported considering growth of maximum plastic radius with increasing mud pressure. The solution explicitly considers coefficient of lateral earth pressure that is not equal to unity. Finite element calculations were used to examine the effectiveness of the solution. Comparisons were undertaken between the size of the plastic zone surrounding the borehole (where shear failure has occurred) obtained from the new approximate closed form solution, the Delft equation, and the finite element
calculations. These demonstrate that the new solution provides estimates of mud radius that more closely match finite element calculations than do estimates obtained using the Delft equation when initial ground stress conditions are not isotropic. The new solution still neglects interaction between the borehole and the ground surface, and this eventually influences the solution when the plastic zone approaches the ground surface.

Maximum mud pressure to prevent mud loss due to hydraulic fracturing can be estimated using the procedure described by Kennedy et al. (2004a, b). However, if shear failure develops in the vicinity of the borehole, blowout is the likely failure mechanism and the dependence of the maximum mud pressure on $K_0$ can be estimated using the new closed form solution. In the absence of contrary experimental or field evidence, allowable borehole pressures can be estimated for the situation where the plastic zone stretches halfway to the ground surface (providing between 20% and 30% reserve capacity). Projects where mud loss would be particularly hazardous should likely employ lower mud pressures.

Parameters for use in these calculations should be estimated by experienced geotechnical engineers. Preliminary values for cohesive strength and the coefficient of lateral earth pressure at rest indicate that blowout is the dominant failure mechanism in normally consolidated and lightly overconsolidated clays. Tensile fracture is expected in heavily overconsolidated clays where the coefficient of lateral earth pressure exceeds 1.8. While the solutions presented here have been derived through careful consideration of the relevant soil behavior, they are theoretical in nature, and both field and laboratory studies would provide valuable guidance on the performance of these analytical methods.
REFERENCES


Table 5.3 Soil parameters used in the course of finite element analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole diameter, $D$</td>
<td>0.4 meters</td>
</tr>
<tr>
<td>Borehole depth, $H$</td>
<td>2 and 4 meters</td>
</tr>
<tr>
<td>Soil unit weight, $\gamma_{soil}$</td>
<td>20kN/m$^3$</td>
</tr>
<tr>
<td>Drilling mud unit weight, $\gamma_{mud}$</td>
<td>13kN/m$^3$</td>
</tr>
<tr>
<td>Coefficient of lateral earth pressure at rest, $K_0$</td>
<td>0.75, 0.85, 1, 1.4 ,and 1.6</td>
</tr>
<tr>
<td>Undrained cohesion, $C_u$</td>
<td>20, 24, 40, and 60 kPa</td>
</tr>
<tr>
<td>Undrained elastic modulus, $E_u$</td>
<td>5 and 15MPa</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.499</td>
</tr>
</tbody>
</table>

Table 5.4 Comparisons of stress ratios estimated using the three different solutions (borehole depth of 2m and diameter of 0.4m)

<table>
<thead>
<tr>
<th>$K_0$</th>
<th>$P^5_{\text{Delft}}/P^5_{\text{New}}$</th>
<th>$P^{11}<em>{\text{Delft}}/P^{11}</em>{\text{New}}$</th>
<th>$P^\infty_{\text{Delft}}/P^\infty_{\text{New}}$</th>
<th>$P^5_{\text{New}}/P^5_{\text{FEA}}$</th>
<th>$P^{11}<em>{\text{New}}/P^{11}</em>{\text{FEA}}$</th>
<th>Stress Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>1.15</td>
<td>1.04</td>
<td>0.95</td>
<td>0.97</td>
<td>1.0</td>
<td>1.29</td>
</tr>
<tr>
<td>0.85</td>
<td>1.1</td>
<td>1.04</td>
<td>1.0</td>
<td>0.99</td>
<td>0.97</td>
<td>1.25</td>
</tr>
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<td>1.0</td>
<td>0.99</td>
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<td>1.2</td>
</tr>
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<td>1.2</td>
<td>1.1</td>
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<td>0.88</td>
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<td>0.98</td>
<td>1.26</td>
</tr>
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<td>1.4</td>
<td>1.1</td>
<td>1.02</td>
<td>0.93</td>
<td>1.0</td>
<td>0.98</td>
<td>1.28</td>
</tr>
</tbody>
</table>

$P^B_A$: subscript $A$ = Delft, New or FEA denoting the three different solutions; superscripts $B=5$, 11 and $\infty$ denote the distance the plastic zone extends (corresponding to halfway and all the way to the surface, and a very large distance).

Stress Factor = $P^5_{\text{New}}/P^{11}_{\text{New}}$ (pressure ratio for plastic zone halfway and all the way to the surface).
a. Shear failure - blowout  
b. Tensile failure - hydraulic fracturing

Figure 5.1 Two mechanisms of soil failure leading to mud loss.

a. Delft equation considering hydrostatic stresses  
b. Non-hydrostatic stress for new solution

Figure 5.2 Boundary stresses and plastic zones
Figure 5.3 Circular hole in an infinite plate (modified from Obert and Duval 1967).
Figure 5.4  Example mesh and nodal loads equivalent to the applied mud pressures.
Figure 5.5 Plastic zone development with increasing mud pressure [diameter of 0.4m, depth of 2m, $K_0$ of 0.85, $c_u$ of 20 kPa, $\gamma$ of 20kN/m$^3$]

Figure 5.6 Plastic zone development with increasing mud pressure [diameter of 0.4m, depth of 2m, $K_0$ of 1.4, $c_u$ of 24 kPa, $\gamma$ of 20kN/m$^3$]
Figure 5.7 Comparison of extent of plastic zone calculated using the Delft Equation, the new solution, and finite element analyses for five different initial stress conditions: $K_0$ values of 0.75, 0.85, 1, 1.2 and 1.4 [borehole diameter $D$ of 0.4m, depth $H$ of 2m, $\gamma$ of 20kN/m$^3$, $c_u$ of 20 kPa and $c_u$ of 24 kPa for $K_0=1.4$]
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\[ 0.5(3 - K_0)P_0 - C_u \]

\[ 0.5(3K_0 - 1)P_0 - C_u \]
Chapter 6

PREPARATION FOR HYDROFRACTURING TEST

6.1 INTRODUCTION

Numerous studies (Leach and Bures 1994, Frank and Barkley 1995, Wong and Alfaro 2000) have been performed to take advantage of hydrofracturing to increase hydraulic conductivity in the gas and oil production industry and to facilitate contaminated soil remediation. The studies mentioned above focused on a vertical borehole, which is a configuration of interest for oil and gas recovery. However, directional drilling for pipe installation involves horizontal boreholes. Mud pressure in the borehole must be high enough to prevent the borehole collapse, suspend the cuttings, and transport the cuttings back to the ground surface. However, if the mud pressure in the borehole exceeds the maximum mud pressure that the soil can withstand, then mud loss or inadvertent mud return resulting from either hydrofracturing (tensile failure) or unconfined plastic flow (shear failure or blowout) could occur depending on type of soil and initial stress, mud properties, and the interaction between mud and soil. Mud loss during directional drilling can have a substantial effect on infrastructure and the environment in the area surrounding the construction.

Chapter 5 discussed mud loss due to soil failure in clays; and presented an approximate solution for estimating of maximum mud pressure the case where $K_0 \neq 1.0$ to be made for a range of coefficient of lateral earth pressure ($K_0$). This work indicated that the mud loss mechanism is controlled by $K_{oc}$ and the mechanism of soil failure (hydrofracture or blowout) is dependent on overconsolidation ratio.

However, little physical data are available to evaluate and calibrate the new procedures being proposed or existing “state-of-the-art” design equation used in practice (the “Delft solution”).
Moreover, field evidence is difficult to use to evaluate or calibrate these findings, because the magnitude of soil strength and lateral earth pressure will be difficult to establish precisely, and it would be difficult to determine whether differences in measured and calculated pressures results from errors in the methods themselves, or the parameters used in the models. Therefore, it is considered both reasonable and useful to obtain evidence from hydrofracturing experiments in the laboratory testing cohesive soils (clays) to directly evaluate or calibrate the theoretical and numerical solutions, and their use in HDD installation design.

Preparatory work for laboratory tests within purely cohesive soils (clay) is presented in this chapter. This preparation includes the design of a top extension box for clay consolidation, selection of the soil sample, measurement of horizontal soil pressure, and the study of the possibility of performing hydrofracturing tests within purely cohesive soils in the small biaxial test cell through three dimensional finite element analysis. It finishes with some conclusions and suggestions for the hydrofracturing tests. While it did not ultimately prove possible to conduct these experiments as part of this doctoral research study, the preparations are described here so they can guide future efforts.

6.2 DESIGN OF THE TOP EXTENSION BOX

In order to perform hydrofracturing tests in clay, a Kaolin clay slurry can be consolidated in the small biaxial test cell as shown in Figure 6.1. The test cell is 320mm wide, 780mm long and 800mm high, and made with 8.5 mm thick steel plate to limit the side wall deformation. Three Lexan plates of 12 mm thickness are mounted inside each wall of the test cell. These are prepared to facilitate measurement of horizontal pressure and drilling of the test borehole as explained later in this section. Large deformations of the soil surface are expected to occur during slurry consolidation due to the high water content of slurry, so accurate estimation of initial slurry height is important to ensure that the final height of the consolidated sample is sufficient. For this investigation, the final height of the specimen should be equal to or
slightly lower than the height of the bottom box, so the total cover depth will be high enough
to minimize the effects from the surface, and surcharge can be applied by an MTS loading
system.

The initial height \( (H_i) \) of fully saturated slurry is:

\[
H_i = \frac{V_s}{A_s} (1 + W_i G_s)
\]  

[6.1]

where \( V_s \) is the volume of Speswhite Kaolin Powder, \( A_s \) is the cross section area of the test
cell, and \( W_i \) is the initial water content of the slurry (i.e. 120%, or around two times the liquid
limit).

The ratio of initial to final height of the sample \((H_i/H_f)\) can be obtained from the following
expression:

\[
\frac{H_i}{H_f} = \frac{1 + W_i G_s}{1 + W_f G_s}
\]  

[6.2]

where \( H_f \) and \( W_f \) are final height and water content at the end of consolidation. As numerous
research results (Tabbaa, 1987, Mair 1979, Kim 1993, Smith 1993, Martin 1994) indicate, the
typical value of liquid limit for Kaolin clay is around 65%, the plastic limit is around 34%,
and specific gravity is 2.61. If the Kaolin powder is mixed with pure water with water content
around 120% and final water content after consolidation is around 40% (assumed), to
generate a 770 mm high consolidated soil sample, the ratio of initial to final height of the
sample \((H_i/H_f)\) is 2. Kim (1996) also reported that the ratio of initial to final height of the
sample in his tests for the normally consolidated and overconsolidated clay samples were 1.8
to 1.9. Therefore, the top extension box should have the same size as the bottom test cell.

To limit the deformation of the side walls, the maximum top load is assumed to be 100kN
which can produce a maximum top surcharge of 400kPa. The coefficient of lateral earth
pressure at rest, \( K_0 \) was assumed to be one, to ensure the design is conservative (lateral earth
pressure equals vertical pressure at the beginning of consolidation of the slurry, and it will change during the process of consolidation). Steel plates of 13mm thickness are needed to fabricate the top extension, and four S shaped beams are welded to the middle part of these steel plates to limit the deflection of the box under the maximum consolidation pressure. The final design of the top extension box is shown in Figure 6.2.

6.3 SOIL SPECIMENS

Speswhite Kaolin clay power mined by the English China Company in St. Austel, Cornwall, UK is suggested for use in preparing the consolidation slurry. Speswhite Kaolin has been widely used in laboratory research for various small scale modeling tests, especially in tunneling studies (Tabbaa, 1987, Mair 1979, Kim 1993, Smith 1993, Martin 1994). The principal reasons for the popularity of Speswhite Kaolin are that, for a clay, it has a relatively high coefficient of permeability, of the order $10^{-6}$mm/s (Tabbaa, 1987, Smith, 1993) which allows rapid consolidation of large clay specimens from reconstituted slurry.

As a widely used physical testing material, the physical and mechanical properties of Speswhite Kaolin have been well documented by many researchers such as Steenfelt, Randolph and Wroth (1981), Martins (1983), Santa Maria (1988), and Smith (1993). The index properties and mechanical properties of Speswhite Kaolin are listed in Table 6.1.

6.4 MEASUREMENT OF HORIZONTAL SOIL PRESSURE

6.4.1 Introduction

Work by Kennedy et al. (2004) and in Chapter 5 demonstrate that the occurrence of mud loss resulting from hydrofracturing depends on the soil properties (undrained shear strength of
clay) and initial stress conditions (coefficient of lateral earth pressure at rest, $K_0$), so the determination of $K_0$ values during the tests is very important.

### 6.4.2 Novel Soil Pressure Sensor

The commercial earth pressure cell is relatively large, and causes measurement error due to the interaction between the earth pressure cell and the clay. Moreover, mud flowing out from the buried position of the earth pressure cell should be avoided in the course of hydrofracturing tests. All these limitations imply that conventional commercial earth pressure cells cannot be used in the hydrofracturing test. A novel soil pressure sensor as shown in Figure 6.3 is introduced to avoid these limitations mentioned above. The sensor consists of two parts; a membrane housing, which is mounted flush to the soil structure interface, and a housing back which provides a hermetic seal of the cylindrical volume behind the membrane face. The membrane face is 0.25mm in thickness and 13.6mm in diameter (Talesnick 2005). The design assumes that the soil contact stress over the dimension of the membrane is uniform. A full bridge diaphragm configuration is adhered to the under sides of the membrane face. Three sets of three holes are equally distributed around the flange of the membrane housing. One set of thread holes is used to align the outer face of the membrane flush with the solid boundary. A second threaded set seals the housing back to the membrane housing, and the final, unthreaded set is employed to fix the sensor to the solid boundary. Two holes in the central area of the housing back allow for a sealed electrical feed-through and a sealed pneumatic feed-through.

The concept of the sensor is based on the null method (Doebelin 1990). The diaphragm deforms under the earth pressures, a response that is monitored by the diaphragm’s strain gage bridge. To maintain infinite stiffness (i.e. negligible deformation), air pressure is applied to the cylindrical volume behind the membrane face, and it is regulated until the outer signal of the diaphragm strain gage bridge is returned to its initial, undeformed value. This
correction is repeated 50 times per second in a tightly controlled PID loop. The pressure required to null the signal of the diaphragm bridge can be calibrated to the pressure applied to the outer membrane face: elastic theory dictates that they are equal with a calibration factor of 1.0. To understand the operation of this novel soil pressure sensor and evaluate the effectiveness of soil pressure measurement, laboratory tests were carried out to measure the soil pressure acting on the interaction surface of the buried HDPE pipe. More details can be found from Talesnick et al. (2005) and Appendix F.

6.4.3 Calibration of the Novel Soil Pressure Sensor

The calibration of the novel soil pressure sensor includes two steps:

- Step 1 (Not Nulling): The soil pressure sensor is mounted into the calibration cell as shown in Figure 6.4 which can apply air pressure on the sensor via a thin rubber membrane, two pressure transducers as shown in Figure 6.5 are connected parallel to the calibration cell measuring inside pressure, internal pressure is then increased to 150kPa with an increment of 15kPa in each step, and then decreased to zero. An inverse calibration factor is obtained which will be input to the traction cell calibration blank in the second calibration step. A typical result for the first step of calibration is shown in Figure 6.7.

- Step 2 (Apply Nulling): To applying the internal pressure to return the strain to zero, the program “Labview” was a tightly maintained PID (Proportional-Integral-Derivative) control loop which runs at a frequency of 10Hz. The “Null Pressure” (internal pressure to restore membrane strains to zero) is applied by an electro-pneumatic controller (BB1) shown in Figure 6.6, which receives an analog command (0-10Vdc) directly from the controlling computer and outputs a scaled, regulated air pressure to the soil pressure sensor. Relations between the calibration cell pressure and soil pressure sensor chamber pressure are recorded by the pressure transducer as
depicted in Figure 6.8. As shown in the plots, the best fit for those two pressures are 1.001975 for the pressure sensor that was tested, which indicates that the inside and outside pressures match well.

### 6.4.4 Data Acquisition System

As described above, novel soil pressure sensors are employed to measure the horizontal pressure acting on the vertical box walls. To work with the novel soil pressure sensor, a new data acquisition system interacting with Labview software is needed. The data acquisition (DAQ) system consists of the national instrumental (NI) SCXI system and Plug-In DAQ board installed in a personal computer bus slot. The National Instruments SCXI-1000 is a four slot chassis that houses the standard NI SCXI modules. There SCXI modules (SCXI 1520, SCXI1180 and SCXI 1125) are installed into the SCXI-1000 to measure the electrical signal and process signal conditioning before it can be transformed to the personal computer for analysis and presentation. A SCXI-1520, 8 channel universal strain gauge input module is inserted into slot 1 of the SCXI-1000 chassis, and the SCXI-1314, a universal strain gauge terminal block is connected to the SCXI-1520. Traditional transducers which generate electrical signals to measure physical phenomena (i.e. force, pressure and position) can be connected to SCXI-1314. A two side’s connector SCXI-1349 (68pin/50 pin connector) is installed at the back of the SCXI-1000 chassis. The 68-pin back connector is for attach the data cable coming from the Plug-In DAQ device in the personal computer. The SCXI-1180 (slot 2 in the SCXI chassis) is a feed through panel which contains a cable that connects to the side connector of the SCXI-1349 cable adapter and allows you to access the unused channels on the Plug-In DAQ device via a general purpose terminal block SCXI-1302.

The hardware and I/O channels are managed using the Measurement and Automation Explorer (MAX), a high level program used to configure National Instruments hardware and software (LabVIEW).
6.5 POSSIBILITY OF THE HYDROFRATURING TEST IN THE SMALL TEST CELL

The test cell was modified by cutting one narrow side wall and replacing the steel plate with the Lexan sheet to advance the test borehole in the course of the hydrofracturing/blowout test within sand performed by Elwood et al. (2007) and Elwood (2008). The test borehole was advanced from one side by using a thin wall Shelby tube, and was terminated around 0.2m (a pocket shape test borehole was formed). One sealing packer was inserted into the borehole to seal the borehole, and bentonite mud was injected into the borehole to expand the borehole. Since one end of the borehole is blocked by the initial intact soil, if the length of the test borehole where interaction with the drilling mud is relatively short, the stress around the middle of the borehole may exhibit three dimensional stress conditions, and moreover, tensile fractures may occur around the corner of the end of the borehole, rather than initiating at the crown of the borehole for the initial stress condition with $K_0<1$ like that predicted by Kennedy et al. (2004a). Considering the effort of preparing the soil sample (one sample’s consolidation may take 3 or more months) and the time limits, it is necessary to ensure that hydrofracture will initiate at the middle part of the test borehole, rather than the corner of the end of the borehole.

A finite element model has been created to simulate the hydrofracturing test within a purely cohesive clay with a pocket shaped borehole. The soil is modeled with elastic-perfect plastic material under undrained conditions. Top surcharge is applied to simulate the construction depth of 5m, and undrained shear strength of the clay is set to be 50kPa. Table 6.2 summarizes the soil properties used in the finite element analysis. As concluded by Kennedy et al. 2004a, hydrofracturing will initiate at the crown of the borehole when the tangential stress is becoming less than the tensile strength of the soil, usually conservatively assumed to
be zero, the developments of minor principal stress in two elements located at two typical locations (the middle of the borehole and the corner of the borehole) are studied. The plots of the relations between minor principal stress and mud pressure are presented in Figure 6.9. As can be seen from Figure 6.9, the development of minor principal stress for crown element at the middle of the borehole closely matches the results calculated from elastic theory and plasticity theory. The mud pressure that initiates hydrofracturing is 72kPa. However, for the crown element located at the corner of the borehole, the development of minor principal stress does not follow the theoretical prediction in the elastic stage, which is logical because stress concentration occurs at the corner of the borehole after the borehole excavation, and it also exhibits three-dimensional stress conditions. After initial yield, the minor principal stress follows the plasticity theory well. As can be determined, the maximum mud pressure required to initiate hydrofracturing at the corner of the borehole is 92.2 kPa, which is higher than the elastic prediction by only 20kPa. The comparison indicates that hydrofracturing is not likely to occur during the hydrofracturing test, since the pressure needed to initiate hydrofracture at the corner of the borehole is higher than that required to initiate hydrofracturing at the middle of the borehole. Furthermore, once the fracture initiates, it begins to propagate and mud pressure in the borehole is expected to drop. However, this finite element results did not consider other influence factors such as the disturbance of the borehole around the corner during borehole drilling, and the effects of seepage of the drilling mud. Considering the above aspects and the efforts and time to consolidate clay, an open borehole with two packers sealing at each end would be a good choice in future experiments.

6.6 SUMMARY

Hydraulic fracturing is a consequence of Horizontal Directional Drilling that has often been encountered and can lead to loss of drilling efficiency, damage to nearby infrastructure, and costly environmental damage. The “Delft solution” was introduced to predict the maximum allowable mud pressure, and is currently being used in the design of many drilling projects.
This solution is based on cavity expansion theory, and it does not consider the coefficient of lateral earth pressure at rest ($K_0$). Kennedy et al. (2004a) considered tensile failure as a cause of mud loss in purely cohesive soils, by examining soil response in the elastic and plastic state after internal mud pressure is increased. It was concluded by Kennedy et al. (2004a) that the maximum allowable mud pressure is a function of the tensile strength of soil, the initial total overburden pressure, and the initial stress condition (values of $K_0$). A new shear-failure solution was developed in Chapter 5 to calculate maximum pressures considering the initial stress state (in effect extending the Delft solution to consider $K_0$).

So far, little physical data is available for clay to evaluate and calibrate the “Delft solution”, the modified solution from Chapter 5, and the solution for tensile fracture (hydrofracture) introduced by Kennedy et al. (2004a). Physical evidence is needed like that from scale laboratory tests both to calibrate these theoretical studies and pipeline installation design.

Several points relative to future hydrofracturing tests were discussed including the design of the top extension box, the choice of soil sample and its relevant properties, and the system used to measure the horizontal soil pressure. A finite element model was produced to study the possible effectiveness of a hydrofracturing test using the small biaxial test cell. The development of minor principal stress at two different locations was studied, and compared with the results calculated from elastic and elastic-plastic theory. The results indicated that the hydrofracturing test can be performed in the small biaxial scale tests using a “pocket shape” borehole. However, considering other influence factors and the effort needed to complete clay consolation, an open borehole shape with packers sealing at each of the two ends is suggested.
REFERENCES


Mair, R.J., 1979, *Centrifugal modeling of tunnel construction in soft clay*. Ph.D.Thesis. Cambridge University, UK.


Table 6.1 Typical values for index properties and mechanical properties of Speswhite Kaolin (after E.T.R. Dean et al. 1996)

<table>
<thead>
<tr>
<th>Some Material Properties of Speswhite Kaolin</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>MC at Plastic and Liquid Limits</td>
<td>2.64</td>
<td>Al-Tabbaa (1984)</td>
</tr>
<tr>
<td>$K_0$, normally consolidated</td>
<td>38%, 69%</td>
<td>Clegg (1981), Airey (1984)</td>
</tr>
<tr>
<td></td>
<td>31%, 64%</td>
<td>Phillips (1989)</td>
</tr>
<tr>
<td>Critical state stress ratio, $M$</td>
<td>0.69</td>
<td>Airey (1984), Al-Tabbaa (1984)</td>
</tr>
<tr>
<td></td>
<td>0.9 (comp), 0.68 (ext)</td>
<td>Al-Tabbaa (1984)</td>
</tr>
<tr>
<td></td>
<td>0.82 (comp)</td>
<td>Elmes (1986)</td>
</tr>
<tr>
<td></td>
<td>0.3-0.21</td>
<td>Airey (1984),</td>
</tr>
<tr>
<td>Critical state model, $\lambda$</td>
<td>0.14</td>
<td>Elmes (1986)</td>
</tr>
<tr>
<td></td>
<td>0.187</td>
<td>Al-Tabbaa (1984), Phillips (1989)</td>
</tr>
<tr>
<td></td>
<td>3.44</td>
<td>Clegg (1981)</td>
</tr>
<tr>
<td>Critical state model, $\Gamma$</td>
<td>2.87</td>
<td>Elmes (1986)</td>
</tr>
<tr>
<td></td>
<td>3.00</td>
<td>Al-Tabbaa (1984),</td>
</tr>
<tr>
<td>$c_u$ isotropic consolidated</td>
<td>0.23P$_{max}$</td>
<td>Clegg (1981)</td>
</tr>
<tr>
<td>Permeability of 1D consolidated samples at specific volume $V=1$+voids ratio</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical direction</td>
<td>0.53(V-1)$^{3.16} \times 10^{-6}$ mm/s</td>
<td>Al-Tabbaa and Wood (1987)</td>
</tr>
<tr>
<td></td>
<td>0.34-2.9x$10^{-6}$ mm/s</td>
<td>Springman (1993)</td>
</tr>
<tr>
<td>Horizontal direction</td>
<td>1.49(V-1)$^{2.03} \times 10^{-6}$</td>
<td>Al-Tabbaa and Wood (1987)</td>
</tr>
</tbody>
</table>

Table 6.2 Soil properties used in the finite element analysis

| Cover depth, $H$, m                        | 5m |
| Diameter of the borehole, $D$, m           | 0.045m |
| Unite weight, $y_{soil}$, kN/m$^3$         | 20 kN/m$^3$ |
| Young’s modulus, $E$, MPa                  | 15MPa |
| Undrained cohesion, $c'$, kPa              | 50kPa |
| Poisson’s ratio, $\nu$                     | 0.499 |
| Coefficient of lateral earth pressure at rest, $K_0$ | 0.6 |
Figure 6.1 Schematic of the biaxial cell construction (after Elwood 2008)
Figure 6.2 Top extension box for the hydrofracturing test (after Elwood 2008)
Figure 6.3  Novel soil pressure sensor to be used in the hydrofracturing tests

Figure 6.4 Soil pressure sensor calibration cell
Figure 6.5 Pressure transducer used in the pressure measuring system

Figure 6.6 Electro-pneumatic controller used in the pressure measuring system
Figure 6.7 Step 1 in the calibration of the soil pressure sensor (Not nulling)
Figure 6.8 Step 2 in the calibration of the soil pressure sensor (Apply nulling)
Figure 6.9 Plot of minor principal stress (middle part and end part of the borehole) at different mud pressures for elastic plate theory, plasticity theory, and finite element analyses of firm clay soil \([H_{soil} = 5 \text{ m}, \gamma_{soil} = 20 \text{ kN/m}^3, K_0 = 0.6, c_u = 50 \text{ kPa}]\)
Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

7.1.1 Key Findings from the Literature Review

Inadvertent mud return or mud loss resulting from high internal mud pressure is a problem that can often be encountered by installation engineers and contractors during Horizontal Directional Drilling (HDD). The consequences of mud loss can be substantial, affecting surrounding infrastructure and the environment. An industry survey in 1997 revealed that loss of drilling slurry (also called mud loss) was the top problem identified by HDD contractors from the United States and Canada (Allouche et al. 2000). In recent years, concern about mud loss effects rose in the public given the remarkable growth of HDD use to install underground facilities. A clear understanding of the mud loss problem is necessary to advance the application of underground infrastructure installation, minimizing social impacts, and to enable prediction, control or prevention of mud loss during directional drilling projects.

Mud loss can occur through hydraulic fracturing (Kennedy 2004a, 2004b) and/or blowout or shear failure (Arends 2003) mainly depending on the type of ‘host’ soil and the initial stress condition. The current “state-of-the-art” design practice (the “Delft solution”) used to predict the maximum allowable mud pressure to control mud loss accounts for unconfined shear failure around the borehole (blowout) for both cohesive and frictional soils (Lugar and Hergarden 1988, Keulen 2001, Arends 2003). This current “state-of-the-art” design practice was initially based on the cavity expansion theory derived by Vesic (1972), which neglects the effect of ground surface, gradient of stress (unit weight of soil) and strength with depth, and assumes that initial stress is isotropic (coefficient of lateral earth pressure at rest, $K_0=1$).
This closed form solution was adopted as a design solution by the US Army Corps Engineers (Carlos et al. 2002 a, b). However, little physical data have been available to evaluate this design practice, and most numerical studies made assumptions like those employed in deriving the Delft equation which does not necessarily effectively capture field condition, where the soil is heterogeneous, and in an anisotropic initial stress.

The objectives of this research have been to study the mud loss through finite element analysis and scaled laboratory tests, to quantify the maximum mud pressure within frictional and cohesive materials, and to evaluate the “Delft solution”. The recent laboratory studies (both small scale and large scale tests) on mud loss within sand performed at Queen’s are reviewed. The small scale hydrofracturing/blowout tests were performed in a biaxial test cell with 0.78m height, 0.32m width and 0.8m length. A medium poorly graded sand with a mean grain size ($D_{50}$) of 0.2mm, and uniformity coefficient $C_u$ of 3.4 was used in the tests. A clean circular borehole with diameter of 0.045m was used to represent the real borehole geometry in drilling practices based on the evidence from Knight et al. (2005), Ariaratnam et al. (2001), and Panah and Yanagisawa (1994). Physical evidence during post-excavation indicated a flow path (flow tube) was formed at the end of the test borehole in almost all of the tests (80%). It was concluded that plane strain conditions do not apply in these tests. The physical configuration can be considered to be similar to the conditions encountered at the end of the borehole during the advancement of the pilot bore. It was reported that the results obtained from a two-dimensional numerical model should not accurately represent the conditions of these experiments (Elwood, 2008). In order to accurately represent the stresses developed during these specific experiments, it is necessary to carry out a three-dimensional numerical analysis to provide better representation of the geometry than that associated with a two-dimensional plane strain approximation.
7.1.2 Finite Element Analysis of Small Scale Laboratory Tests

A series of three-dimensional finite element analyses were carried out in Chapter 2 to interpret the small scale laboratory tests reported by Elwood et al. (2007). To simplify the modeling process, several assumptions were made:

1. No filtercake was considered in the course of finite element analysis based on the observation reported by Elwood et al. (2007) that little or no changes occurred to the sand as the mud flowed into it, in practice, there was no change in strength with the exception of water content and hydraulic conductivity.

2. The effects of mud seepage and interaction between mud and the sand were neglected in the course of the modeling.

3. The coefficient of lateral earth pressure at rest, \( K_0 \) was set to 0.55 (average values of measurement from the small scale laboratory tests) in all cases based on the measured horizontal earth pressure.

4. The plastic failure was modeled using the Mohr-Coulomb shear failure criterion, and the soil was assumed to be an isotropic uniform medium with an average total density of 1750kg/m\(^3\).

5. The properties of the sand were set to constant based on the measurements of direct shear test (friction angle of 33.8\(^0\), dilation angle of 15\(^0\), and cohesion of 1.65kPa).

6. The geometry and layout of the borehole was assumed to be symmetric, and only half of the system geometry model was modeled.

7. The gradient of the mud column was neglected, and the borehole pressurization process was applied uniformly, and was assumed to occur simultaneously throughout the entire borehole.

The principal purposes of the finite element analysis were to evaluate the effectiveness of these small scale laboratory tests, to understand better the mud loss phenomenon, and to investigate the effects of the side wall of the test box. Finite element results indicated that the
side walls at a distance of 3.2 borehole diameters from the borehole influenced the prediction of maximum mud pressure values, and the mud flowing out through the unconfined shear failure zone (i.e. blowout) formed at the end of the borehole is the major mud loss form. The comparisons implied that the measured mud pressure from these small scale laboratory tests cannot be used to quantify the maximum allowable mud pressure since the side walls seem to strengthen the soil capacity to resist internal mud pressure. However, the three-dimensional finite element analyses captured the mud loss phenomenon; these calculations explain how the injected mud flowed up to the top surface from the end of the test borehole. As mud pressure was increased during the finite element analysis, the plastic zone reached the side walls first, and then a partial plastic zone was developed at the end of the borehole; this eventually extended up to the top surface at an angle of 45 degrees.

Two-dimensional plane strain finite element analyses which can be used to simulate the real field condition during the product pipe pull back process (considering directional drilling as a plane strain problem given the length of drilling path and the relatively small diameter of the borehole) were also carried out to compare with the results from the three-dimensional finite element analyses. The plane strain finite element analyses were terminated due to convergence problems when the plastic zone touched the side walls. The results from plane strain finite element analyses implied the plane strain condition did not apply to these small scale laboratory tests and the potential influence of side walls on the maximum allowable mud pressure. A statistical study using of the Mann-Whitney U test was performed to compare the results from the three-dimensional numerical analyses and the small scale laboratory tests. The results from Mann-Whitney U test indicated there was no significant difference between the results from three dimensional finite element analyses and the experimental measurements, which implied that the three dimensional finite element analyses effectively simulated those small scale laboratory experiments.
7.1.3 Finite Element Analysis of Large Scale Laboratory Tests

Six large scale laboratory tests were performed in a test pit 2m high, 2m wide, and 2m long, in which four tests were with only sand, and two tests using sand overlain by well graded granular material (a typical road base material) to consider the effects of a stiff surface layer on the mode of mud loss and the maximum mud pressure. The purpose of these large scale laboratory tests was to eliminate the influence of side walls from the small scale laboratory tests, and further investigate mud loss problem within sand. In these tests, besides internal mud pressure, the surface displacement was also measured using the particle image velocimetry (PIV) technique. Finite element analyses were also performed to help better understand these large scale laboratory tests. Finite element analysis results indicated that the “Delft solution” overestimates the capacity of soil to resist internal borehole pressure by more than 100%, and finite element analysis provides an effective estimate of maximum mud pressure value when the maximum plastic radius reaches up two-thirds of the distance to the ground surface ($R_{p,max} = 2/3H$). The effects of the length of the borehole were also investigated using finite element modeling. The results indicated that when the ratio of the length of the borehole and the diameter of the borehole, L/D, is larger than 20, the borehole exhibits plane strain response, and it can be used to simulate the mud loss problem during the product pipe pull back process.

7.1.4 Additional Large Scale Laboratory Tests

7.1.4.1 Maximum Mud Pressure

Two additional large scale laboratory tests were reported in Chapter 3 to extend the comparison with finite element analysis and the “Delft solution” by considering deeper cover and longer test borehole. Three-dimensional and two-dimensional finite element analyses were performed to simulate these additional large scale laboratory tests. Again, finite element analyses provided an effective calculation of the maximum mud pressure value (difference
less than 12%), that was superior to calculated pressure from the “Delft solution”. However, this finite element model did not capture the tensile surface cracking phenomenon observed in the course of test. This surface cracking explains the need to limit the plastic radius to two-thirds of the cover depth (i.e. \( R_{p,\text{max}} = 2/3H \)), since the cracks weaken the capacity of the top third of the soil to resist the borehole pressure.

Finite element analysis indicated that the initial stress affects the shape of the plastic failure zone and the value of the maximum mud pressure. When coefficient of lateral earth pressure at rest, \( K_0 \), is unity, the plastic zone has a circular shape. The shape of plastic zone changes in the last part of the expansion process (when the plastic zone passes beyond the two-thirds of the total cover depth, and approaches the ground surface). If the coefficient of lateral earth pressure at rest, \( K_0 \), is less than one (e.g. \( K_0 = 0.55 \)), a different shape of plastic zone (elliptic or subelliptic shape with the long axis orientated vertically) develops, the plastic zone extends diagonally from the shoulder and haunches of the borehole and once it extends two-thirds of the way to the top surface, it accelerates up to the surface over an increment in just 25 to 30kPa. In both tests, the “Delft solution” overestimated the maximum allowable mud pressure by 160% to 180%.

Comparison of the extent of the plastic zone illustrated the effects of approximations associated with the Delft solution (which neglects the effect of \( K_0 \), the ground surface and gradient of strength and stress with depth) on prediction of maximum mud pressure. Different shape and rate of growth of the plastic zone occurs as a result of these factors, and this directly influences the maximum mud pressure.

### 7.1.4.2 Surface Displacement

The surface displacements of the sand were measured using Particle Image Velocimetry PIV (White and Take 2002). Targets placed on the surface of the soil at known locations were
monitored by capturing a series of images throughout the course of the tests and tracking known pixel locations placed on the surface of the targets. The accuracy of the displacement monitoring was confirmed by comparing the data of two synchronized cameras positioned at 90 degrees to one another. The vertical displacements of identical targets were compared and confirmed that the measurements were consistent when monitored from two different positions.

The interpreted surface displacement exhibited a bell shape with the maximum surface displacement located around the center line of the borehole, and decreasing with increasing distance from the center of the borehole. The average value of surface displacement is around 26mm. The surface displacement calculated from finite element analysis did not match the measured surface displacement. That is reasonable because finite element analysis only considers the development of the plastic zone as stress is applied to the inside surface of the borehole; and it does not consider the injected mud volume and mud flow within sand during the borehole expansion.

7.1.5 Parametric Study

7.1.5.1 Overview of Parameters Considered
A set of numerical calculations were carried out in Chapter 4 to provide an complete set of maximum mud pressure for a range of borehole geometries, cover depth, and soil strength, and dilation characteristics. This parametric study also considers the value of lateral earth pressure coefficient.

7.1.5.2 Effect of Young’s Modulus, $E$
Both constant Young’s modulus (independent of depth, 10MPa and 20MPa) and varied Young’s modulus (linear increasing with depth) were studied to investigate how Young’s
Modulus affects the maximum allowable mud pressure. Borehole diameter of 0.4m, and cover depths of 4m were simulated. Three typical friction angles, 40°, 35°, and 30°, were evaluated, and associated flow (i.e. $\psi/\phi=1.0$) was modeled. The results indicate that the soil modulus has negligible influence on the maximum allowable mud pressures (the difference is less than 1.5%).

7.1.5.3 Effect of Unit Weight of Sand, $\gamma_{\text{sand}}$

Two different unit weights of sand (18kN/m$^3$ and 22kN/m$^3$) were employed in the finite element analysis. The maximum allowable mud pressures are normalized by the total overburden pressure, $\sigma_{\text{total}}=\gamma_{\text{sand}}\times H$, where $H$ is the distance from the crown of the borehole to the ground surface. The results of the parametric study indicate that the variation of sand unit weight has negligible effect on the ratio of maximum allowable mud pressure to the total overburden pressure. The difference between normalized maximum mud pressures for the two unit weight values is less than ±1.0%. Normalization based on the total overburden pressure can therefore be used to generalize the results from the finite element analysis to materials with a range of different soil unit weights.

7.1.5.4 Effect of Dilation Angle, $\psi$

Two limits for dilation angle ($\psi/\phi=1$ and $\psi/\phi=0$) were used in the course of parametric study to investigate the effect of dilation angle on the maximum allowable mud pressure. The comparison between the results from full dilation and zero dilation analyses indicated that associated flow ($\psi/\phi=1$) is stronger than sand with non-associated flow ($\psi/\phi=0$). The maximum difference is about 25% as dilation angle increase from zero to the angle of internal friction. Using a constant dilation angle in the finite element calculations could provide somewhat unconservative solutions, but a more complex approach is not warranted given the
small acceptable level of error, therefore the use of a fixed angle of dilation (a single value of 15° or an estimation using Bolton’s equation) is considered an acceptable simplification.

7.1.5.5 Effect of Coefficient Lateral Earth Pressure at Rest, $K_0$

The effects of coefficient of lateral earth pressure at rest, $K_0$, on maximum allowable drilling fluid pressure were investigated. The $K_0$ values for normally consolidated soils are often estimated based on the Jaky equation, $K_0 = 1 - \sin \phi$, and solutions for $K_0 = 1$ are selected to consider the case in lightly overconsolidated soils. The maximum allowable mud pressure that soil can support depends on the initial soil stress. The maximum allowable mud pressure increases as the initial stress change from anisotropic to isotropic conditions. This is logical because higher internal pressure is required to initiate shear failure in soil with an isotropic initial stress (zero initial shear stress), and the rate of growth of the plastic zones are also different (the growth speed of the plastic zone in anisotropic soil is much faster than in isotropic soil).

7.1.5.6 Estimating Maximum Mud Pressure within Sand

An approximate equation was developed to facilitate estimation of the maximum mud pressure within sand. An exponential growth equation was introduced to provide the best-fit to the relationship between the pressure ratio and the H/D ratio in Mohr-Coulomb soil with a friction angle of 30 degrees. A scale factor, a function of friction angle and coefficient of lateral earth pressure at rest, $K_0$, was then introduced to calculate the maximum mud pressure for other soil friction angles. The approximate function is:

$$\left( \frac{P_{\text{max}}}{P_0} \right)_\phi = A + \log \left( B \right) \times B^{(H/D)} \times F(\phi, K_0)$$

where $A = 17.57 - \frac{12.9K_0}{K_0 - 0.084}$, $B = \frac{1.23K_0}{K_0 + 0.022}$, and $F(K_0, \phi) = 1.46 \times e^{0.001 \frac{K_0 - 0.346}{10^{0.06}}} \times \left( \frac{\phi - 30^0}{10^6} \right)$. 

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This approximate solution can be easily programmed or used in spreadsheet to calculate the maximum mud pressure within sand. However, given uncertainties of determination of sand properties, good experience and judgment are still required to use the above method for installation design, and more physical data are also needed to evaluate this approximate solution.

7.1.6 Maximum Mud Pressure within Purely Cohesive Clay

Current “state-of-the-art” practice adopted by U.S. Army Corps of Engineers for purely cohesive soils also assumes that the soil is in isotropic initial stress state ($K_0=1$), which has obvious shortcoming with respect to real stress situations in the field. Kennedy (2004a) introduced consideration of mud loss associated with hydraulic fracture (tensile failure), when the minor principal stress becomes smaller than the tensile strength of soil (conservatively assume to be zero) before plastic yielding occurs. However, that solution can only be used to estimate the fracture pressure for some values of $K_0$, and it does not apply when shear failure occurs instead of tensile fracture. A new approach was developed to consider the effects of coefficient of lateral earth pressure at rest on the blowout solution. This considers anisotropic initial stress but approximates the plastic zone as circular.

Expressions for borehole pressure as a function of radius of the plastic zone ($R_p$) were developed as:

$$ P_i = C_u + \frac{1}{2}(3K_0 - 1)P_0 - C_u \ln \left( \frac{R_0}{R_p} \right)^2 + \left( \frac{C_u}{G} \right) \quad \text{for } K_0 < 1 $$

$$ P_i = C_u + \frac{1}{2}(3 - K_0)P_0 - C_u \ln \left( \frac{R_0}{R_p} \right)^2 + \left( \frac{C_u}{G} \right) \quad \text{for } K_0 > 1 $$

A series of finite element analyses were performed to evaluate the above approach within purely cohesive soils with undrained cohesion ($C_u$) values of 20 and 40kPa, respectively.
These soils were modeled with lateral earth pressure coefficient at rest, $K_0$ values of 0.7, 0.85, 1.0, 1.2, and 1.4, with borehole diameter of 0.2m and with construction depths of 2.0 and 4.0m.

Comparisons were undertaken between the size of the plastic zone surrounding the borehole (where shear failure has occurred) obtained from the new approximate solutions, the “Delft solution”, and the results calculated from finite element analysis. These comparisons demonstrated that the new solution provides estimates of mud radius that effectively approximate the finite element calculation (mud pressure that is within 3.5% for the same plastic radius). The “Delft solution” performs poorly when initial ground stress conditions are not isotropic. The new solution still neglects interaction between the borehole and the ground surface, and this eventually influences the solution at the end of the blowout process. However, at 2/3 of the distance towards the ground surface, the Delft solution only differs 0.5% from the finite element analysis, which indicates the biggest effect is associated with $K_0$ in the clay.

Maximum allowable mud pressure to prevent mud loss due to hydraulic tensile fracturing can be estimated using the procedure described by Kennedy et al. (2004a, b). However, if shear failure develops in the vicinity of the borehole, blowout is the likely failure mechanism and the dependence of the maximum mud pressure on $K_0$ can be estimated using the new approach. Since no physical data is available to assess the performance of these procedures, the solution should be used with caution.

Preliminary studies for typical values of cohesive strength and the coefficient of lateral earth pressure at rest in purely cohesive soils indicates that blowout is the dominant failure mechanism in normally consolidated (NC) and lightly overconsolidated (LOC) clays. Tensile
fracture is expected in heavily overconsolidated (HOC) clays where the coefficient of lateral earth pressure exceeds 1.8.

7.2 RECOMMENDATIONS

Current research findings indicate that the ground heave is an important factor needing further investigated. A more complex finite element model would be valuable, considering the influence of mud flow within soil, as well as non-uniform soil profile and surface cracks produced during the borehole expansion. Various field monitoring data including the internal mud pressure and ground surface heave during pilot bore and product pipe pull back processes are needed to calibrate these findings.

A better understanding of the mud loss mechanism in cohesive and frictional material such as silt is necessary through laboratory tests, field study and finite element analysis. More studies should be carried out within layered systems to understand further the mud loss mechanism when the drilling crosses, or goes beneath stiff layers such as pavement. None of the tests and finite element analyses reported here considers the influence of subsurface water, so more complex testing is necessary using fully saturated silt or sand materials to explore the influence of the ground water.

More comprehensive laboratory studies should be undertaken to complement the research. Ideally, real field experiments capturing the characteristics of the soil, and other field conditions when mud loss was encountered should be compared with the conclusions of this research. A laboratory study in purely cohesive soils could be used to control all the relevant parameters, and to examine the propagation of tensile fractures and the relative importance of tensile fracture and unconfined plastic cavity expansion.
REFERENCES


Appendix A

DETERMINATION OF SAND PROPERTIES

A medium poorly graded sand with a mean grain size ($D_{50}$) of 0.2mm, uniformity coefficient $C_u$ of 3.4 is used to study the mud loss problem and quantify the maximum allowable mud pressure during horizontal directional drilling. The grain size distribution curve is shown in Figure A-1. The Mohr-Coulomb parameters (cohesion and internal friction angle) of this poorly graded sand were obtained through a series of direct shear tests. The sample cross section area is 36 cm$^2$, and the mass of upper plate is 0.47kg. Table A-1 list the corresponding normal stress applied on the test sample through the mass applied to the hanger. Table A-2 summarizes the test samples. Figure A-2 shows the relations between the horizontal displacement and the horizontal shear stress on the cross section. Figure A-3 shows the relations between the normal stress applied in the course of the direct shear test and the shear stress.

As can be seen from Figure A-3, the internal friction angle of test sand is around 33.8 degrees, and the test sand exhibits some cohesion because the test sand had average moisture content of 5.4%.
Figure A-1 Grain size distributions of test soil (after Elwood 2008)
Table A-1 Normal stresses applied to test sample

<table>
<thead>
<tr>
<th>Normal stresses applied to test sample, kPa</th>
<th>Mass applied to the Hanger, kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>0.5</td>
</tr>
<tr>
<td>30</td>
<td>1.1</td>
</tr>
<tr>
<td>35</td>
<td>1.3</td>
</tr>
<tr>
<td>40</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table A-2 Summary of test samples

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross section area</td>
<td>36 cm²</td>
<td>36 cm²</td>
<td>36 cm²</td>
</tr>
<tr>
<td>Sand Volume</td>
<td>55.6 cm³</td>
<td>55.6 cm³</td>
<td>55.6 cm³</td>
</tr>
<tr>
<td>Mass of sand</td>
<td>99g</td>
<td>97.89g</td>
<td>100.4g</td>
</tr>
<tr>
<td>Density of sand</td>
<td>1780 kg/m³</td>
<td>1760 kg/m³</td>
<td>1805 kg/m³</td>
</tr>
<tr>
<td>Moisture content of test sand</td>
<td>4.9%</td>
<td>6.2%</td>
<td>5.2%</td>
</tr>
</tbody>
</table>
Figure A-2 The relationship between the shear stress and horizontal displacement.
Figure A-3 The relationship between normal stress and shear stress of the test sand

\[ \tau = 0.667\sigma + 1.6525 \]

\[ R^2 = 0.986 \]

\[ \phi = 33.8^0 \quad c' = 1.652 \text{kPa} \]
Appendix B

EXAMPLE INPUT FILE OF ABAQUS MODEL

*HEADING
Simulation of Laboratory Test
UNITS: kN and Meter, PRESSURE kPa,
*PREPRINT, ECHO=YES, HISTORY=YES, MODEL=YES
*RESTART, WRITE, FREQ=10
*FILE FORMAT, ZERO INCREMENT

******************************************************************************
** MESH GENERATION **
******************************************************************************
*NODE, NSET=NALL, input=node2m2.inp------------->node data input
*ELEMENT, type=CPE6M, input=element2m2.inp------>element data input
*ELSET, elset=borehole, generate
  1,880, 1
*Elset, elset=eall, generate
  1, 5424, 1
*Elset, elset=m1
  1255, 1311, 1367, 1423, 147, 1759....
*NSET, nset=BOTTOM
  6,  7, 268, 269, 281....
*NSET, nset=RIGHT
  2,  3,  4,  6,  46.....
*NSET, nset=LEFT
  5,  7,  8,  113, 116.....

******************************************************************************
** MATERIALS **
**define material
*SOLID SECTION, MATERIAL=Sand, ELSET=EALL
*MATERIAL, NAME=Sand
*ELASTIC, TYPE=ISOTROPIC
  2.0e4, 0.33
*Mohr Coulomb
  33.8, 0.0001
*Mohr Coulomb Hardening
  1.65, 0.0

******************************************************************************
** INITIAL CONDITIONS, TYPE=STRESS, GEOSTATIC
EALL, -79.2, 0.0, 0.0, 4.4, 0.55, 0.55

******************************************************************************
** AMPLITUDE, NAME=MUD1, TIME=TOTAL TIME
  0.0, 0.0, 1.0, 0.0, 2.0, 0.0, 3.0, 10.0
  4.0, 20.0, 5.0, 30.0, 6.0, 40.0, 7.0, 50.0
  8.0, 60.0, 9.0, 70.0, 10.0, 80.0, 11.0, 100.0

******************************************************************************
** Step 1. GEOSTATIC
*****************************************************/

*STEP, NLGEOM=yes, UNSYMM=YES
Step 1: add initial stress state
*GEOSTATIC
*Dload
EALL, by, -17.5

** BOUNDARY CONDITIONS
**
** Name: BC-1 Type: Symmetry/Antisymmetry/Encastre
*Boundary
BOTTOM, 2
LEFT, 1
RIGHT, 1
*EL PRINT, elset=BB1, position=integration points
S11,S22,SP,SINV,
*EL FILE, elset=BB1, POSITION=AVERAGED AT NODES
E, EP,
*OUTPUT, FIELD,FREQ=2
*NODE OUTPUT, NSET=NALL
U,
*ELEMENT OUTPUT, ELSET=EALL
S,E,PEEQ,
*output, history, freq=10
*element output, elset=BB1
SP1,SP3,
*END STEP
**

*******************************************************************************************

*STEP, NLGEOM=yes, UNSYMM=YES
Step 2: EXCAVATION OF THE BOREHOLE
*STATIC
0.1, 1., 1e-05, 1.
*MODEL CHANGE, REMOVE
Borehole
*DSLOAD, AMPLITUDE=MUD1, OP=MOD
Se1,P, 1.0

.. .. .. .. ..
*END
Appendix C

MANN-WHITNEY U TEST

The Mann-Whitney U test also called the Wilcoxon rank-sum test can be used to determine whether or not two independent populations have the same distribution. It can be used to determine whether one population has values that are significantly higher or lower than the values of another population. If the populations that are sampled have the same shape but not necessarily normal and variability, then the Wilcoxon rank sum test is equivalent to comparing the centers (means) of the two populations.

The M-W test combines two populations, ranks the observations, and then looks at the sum of the ranks for each population. If the two populations are the same, then the sums should be approximately equal. If one of the populations is lower or higher than the other, then the sums of the ranks should differ accordingly. The steps needed to perform the M-W tests are presented in the following section.

Step 1: State the Null and alternative hypotheses

$H_0$: the mean maximum mud pressure from finite element analysis is not significantly higher than that of the experimental measurements.

$H_a$: the mean maximum mud pressure from finite element analysis is significantly higher than that of the experimental measurements.

Step 2: Select the level of significance (the desired significant level of $\alpha=0.05$)

Step 3: Rank the data irrespective of sample category. To find the ranks both data set need to be combined and ordered.

Step 4: Determine the test distribution to use. Critical values of $U$ can be determined from the following table which is for appropriate $n_1, n_2$ and $\alpha$ under the condition that $H_0$ is valid.
Step 5: State the decision rule. If the calculated U value is smaller than the critical value determined from the table, reject the null hypotheses.

Step 6: Compute the test statistic. The U values can be calculated from the following equations:

\[
U_1 = n_1 n_2 + \frac{n_1 (n_1 + 1)}{2} - R_1
\]

\[
U_2 = n_1 n_2 + \frac{n_2 (n_2 + 1)}{2} - R_2
\]

where \( R_1 \) and \( R_2 \) are the sum of ranks assigned to the sample with sizes \( n_1 \) and \( n_2 \), respectively.

Step 7: Make the statistical decision. If the calculated U (\( U_1 \) and \( U_2 \)) values are obviously greater than the critical values, the null hypotheses are not rejected. It can then be concluded that there is not significant evidence of maximum mud pressure difference between the results from finite element analysis and experimental measurements.
Appendix D

EXAMPLE CALCULATION: MAXIMUM MUD PRESSURE IN SAND

Two example calculations are given in this section to illustrate how to use the new approximate solution to estimate the maximum mud pressure in sand. The first example considers the same geometry and sand properties as used in additional large scale test 1. The results from the new approximate solution are compared with the data from experimental measurement and “state-of-the-art” design practice (the “Delft solution”). The second specific example which considers different geometries (deeper cover depth and larger diameter of the borehole compare to large scale tests) is calculated, and compared with the results from the “Delft solution”.

Example 1:

Construction depth \( H = 1.15 \text{m} \), the diameter of the borehole \( D = 0.055 \text{mm} \), friction angle of the sand \( \phi = 33.8^\circ \), cohesion (moisture content of 4.5%) \( c = 1.65 \text{kPa} \), unit weight of the sand \( \gamma = 17.5 \text{kN/m}^3 \), Young’s modulus of the sand \( E = 25 \text{MPa} \), Poisson’s ratio \( \nu = 0.33 \), and coefficient of lateral earth pressure at rest, \( K_0 = 0.55 \). The maximum allowable mud pressure for this case, \( P_{\text{max}} \) then needs to be calculated.

Solution 1 (New approximate equation):

Maximum mud pressure:

\[
K_0 = 0.55, \ H/D = 20.9, \ \phi = 33.8^\circ \\
(P_{\text{max}}/P_0)_\phi = \left\{17.57 - \frac{12.9K_0}{K_0 - 0.004} + \log \left( \frac{1.23K_0}{K_0 + 0.022} \right) \times \frac{1.23K_0}{K_0 + 0.022} \left( \frac{H}{D} \right) \right\} \times \left\{ 1.46 \times e^{0.001} \right\}
\]

\[
\left( \frac{\phi - 30^\circ}{10^\circ} \right) + \left( \frac{40^\circ - \phi}{10^\circ} \right)
\]
The maximum mud pressure: $P_{\text{max}} = 5.75 \times \gamma \times H = 5.75 \times 17.5 \times 1.15 = 116\, \text{kPa}$

**Solution 2 (the “Delft solution”)**

$$\sigma_0' = 20.12\, \text{kPa}, \quad c = 1.65\, \text{kPa}, \quad \phi = 33.8^0, \quad R_0 = 0.0275, \quad R_{p,\text{max}} = 0.795, \quad G = \frac{E}{2(1+\nu)} = \frac{25}{2.66} = 9.4\, \text{MPa}, \quad K_0 = 1.0$$

$$P_{\text{max}} = \left(\sigma_0'(1 + \sin\phi) + cc\cos\phi + cc\cot\phi\right) \left\{ \left(\frac{R_0}{R_{p,\text{max}}} \right)^2 + \frac{\sigma_0' \sin\phi + cc\cos\phi}{G} \right\} - cc\cot\phi$$

The maximum mud pressure: $P_{\text{max}} = 278.8\, \text{kPa}$

**Solution 3: (peak mud pressure measured from the experiment)**

$P_{\text{max}}/P_0 = 5.6, \quad P_{\text{max}} = 5.6 \times 20.12 = 112.7\, \text{kPa}.$

**Example 2:**

Construction depth $H=4\, \text{m}$, the diameter of the borehole $D=0.4\, \text{mm}$, friction angle of the sand $\phi=37^0$, unit weight of the sand $\gamma=20\, \text{kN}/\text{m}^3$, Young’s modulus of the sand $E=25\, \text{MPa}$, Poisson’s ration $\nu=0.33$, and coefficient of lateral earth pressure at rest, $K_0=0.40$. The maximum allowable mud pressure for this case, $P_{\text{max}}$ needs to be calculated.

**Solution 1 (New approximate equation):**

$K_0=0.4, \quad H/D=10, \quad \phi = 37^0, \quad P_0=80\, \text{kPa}$

Maximum mud pressure:
\[ \frac{(P_{\text{max}}/P_0)_{\phi}}{\phi} = \left\{ 17.57 - \frac{12.9K_0}{K_0 - 0.084} + \log \left( \frac{1.23K_0}{K_0 + 0.022} \right) \times \frac{1.23K_0}{K_0 + 0.022}^{(H/D)} \right\} \times \left\{ 1.46 \times e^{0.001} \times \right\} \]

\[ \phi = -300100 + 400 - \phi 100 \]

\[ \frac{(P_{\text{max}}/P_0)_{\phi}}{\phi} = (17.57 - 16.33 + 0.33) \times (1.04 + 0.3) = 2.15 \]

The maximum mud pressure: \[ P_{\text{max}} = 2.15 \times \gamma \times H = 2.15 \times 20 \times 4 = 172kPa \]

Solution 2 (the “Delft solution”):

\[ P_{\text{max}} = (\sigma'_0(1 + \sin \phi) + c\cos \phi + ccot \phi) \left\{ \left( \frac{R_0}{R_{p,\text{max}}} \right)^2 + \frac{\sigma'_0 \sin \phi + c\cos \phi}{G} \right\}^{-\frac{\sin \phi}{1 + \sin \phi}} - ccot \phi \]

\[ \sigma'_0 = 80kPa, \ c = 0 \ kPa, \ \phi = 37^0, \ R_0 = 0.2, \ R_{p,\text{max}} = 2.67, \ G = \frac{E}{2(1+\nu)} = \frac{25}{2.66} = 9.4MPa, \ K_0 = 1.0 \]

The maximum mud pressure: \[ P_{\text{max}} = 9.0 \times \gamma \times H = 9.0 \times 20 \times 4 = 720kPa \]

As can be seen from the above comparison, the new approximate solution provides an effective estimate of the maximum mud pressure, and the “state-of-the-art” design practice (the “Delft solution”) overestimates the maximum mud pressure by 320%.
Appendix E

EXAMPLE CALCULATION: MAXIMUM MUD PRESSURE WITHIN CLAY

A specific example is given in this Appendix to demonstrate how to use the frac-out pressure equation and new blowout approach to estimate the maximum mud pressure within purely cohesive soil (clay). The results are compared with the prediction from the “Delft solution” which did not consider the influence of coefficient of lateral earth pressure at rest, $K_0$ on the maximum mud pressure.

Example calculation:

A borehole is constructed using horizontal directional drilling within a purely cohesive soil at a construction depth $H$ of 4 m. The diameter of the borehole $D$ is 0.4m, the unit weight of the clay $\gamma$ is 20kN/m$^3$, undrained shear strength is $c_u$ 25kPa, and the coefficient of lateral earth pressure at rest, $K_0$, is assumed to be 0.5. The ground water table is beneath the borehole. The maximum mud pressure needs to be determined for this installation design, as does the specific mud loss mechanism to expect if the internal mud pressure exceeds the maximum allowable mud pressure.

Solution:

Cover depth, $H$=4.0m, diameter of the borehole, $D$=0.4m, maximum plastic radius, $R_{p,max}$=2.0m, undrained shear strength, $c_u$=25kPa, total unit weight, $\gamma$=20kN/m$^3$, overburden pressure, $\sigma_v = 20 \times 4 = 80 kPa$, the ratio of undrained shear strength to overburden pressure, $c_u/P_o= 25/80=0.3125$.

Solution 1: Tensile fracture pressure (Kennedy et al. 2004a):

$$P_{trac} = \sigma_v (3 \times K_0 - 1) = 80 \times 0.8 = 64 kPa$$
Solution 2: Blowout failure using the new approach (Xia and Moore 2006):

\[ P_i = C_u + \frac{1}{2} (3K_0 - 1) P_0 - C_u \ln \left( \frac{R_u}{R_p} \right)^2 + \left( \frac{C_u}{G} \right) \]

The maximum mud pressure to prevent blowout failure: \( P_{\text{max}} = 168 \text{kPa} \).

Solution 3: The blowout failure using the “Delft solution” (Arends, 2003):

\[ \sigma'_0 = 80 \text{kPa}, C_u = 25 \text{kPa}, R_0 = 0.2, R_{p,\text{max}} = 2.2 \text{m}, G = \frac{E}{2(1+n)} = \frac{20}{3} = 6.7 \text{MPa}, K_0 = 1.0 \]

\[ P_{\text{max}} = (\sigma'_0(1 + \sin \phi) + C_u \cos \phi + C_u \cot \phi) \left\{ \left( \frac{R_0}{R_{p,\text{max}}} \right)^2 + \frac{\sigma'_0 \sin \phi + C_u \cos \phi}{G} \right\}^{\frac{-\sin \phi}{1+\sin \phi}} - C_u \cot \phi \]

The maximum mud pressure from the “Delft solution” : \( P_{\text{max}} = 216 \text{kPa} \)

Failure criterion: for \( K_0 < 1 \), \( F_1(K_0, P_0, C_u) = \frac{1}{2} (3 \times K_0 - 1) P_0 - C_u = 7 \)  

Since \( F_1 = 7 > 0 \), blowout failure is expected, and the maximum mud pressure is 168kPa based on new approach. For this case, the maximum mud pressure from the “Delft solution” is 216kPa, which overestimates the maximum mud pressure by 29%.
Appendix F

EARTH PRESSURE MEASUREMENT ON BURIED HDPE PIPE

INTRODUCTION

The development of elastic soil-pipe interaction solutions in the 1960’s (e.g. Burns and Richard, 1964; Hoeg, 1968) and finite element analyses since that time (e.g. Katona, 1978; Duncan 1979) have lead to a rational framework for understanding how the relative stiffness of a pipe and the ground surrounding it influence the earth pressures that develop on the structure. Researchers and culvert designers have, naturally, sought to evaluate the efficacy of those solutions. Hoeg (1968) prepared a test structure incorporating load cells connecting different segments so that net force on each segment could be determined; the stiffness of his test pipe was controlled by placing rings within the pipe structure. He conducted buried pipe experiments for this instrumented ‘pipe’ backfilled with Ottawa sand rained into the test box. Others researchers have relied on visual observations or measurements of structural response (global changes in pipe diameter and local strains) rather than contact pressures to confirm their understanding of pipe or culvert response (e.g. Rogers et al, 1996; Dhar et al., 2004). Direct measurements of earth pressures acting on large diameter structures have been made, but the accuracy of these may be jeopardized by local stress distributions around the flat commercial earth pressure cell when it lies on or near the curved pipe surface. Furthermore, errors are known to occur because the earth pressure cell has stiffness characteristics that are different to the pipe and the soil that surrounds it (leading to positive or negative arching, i.e. load redistribution from low stiffness to high stiffness components). To date, little direct evidence has been developed regarding the magnitude and pattern of stress distribution against buried flexible pipe or culverts.

1A version of this chapter has been submitted to Geotechnique for publication:
Mark Talesnick1, Hongwei Xia2, and Ian D. Moore3 2008. Earth Pressure Measurement on Buried HDPE Pipe
Much of the culvert research reported during the 1990’s and since has focused on small diameter thermoplastic pipes (e.g. Brachman et al, 2000). However, no earth pressure data has been available for these small diameter structures because the large size and stiffness characteristics of conventional stress cells have simply made reliable measurements impossible. Exploration of issues like the effect of local variations in soil support under the pipe has therefore had to rely on inferences drawn from structural response. While Rogers et al. (1996) Dhar et al. (2004), and Brachman et al. (2008) were able to employ visual and strain measurements to explain how backfill compaction results in complex earth pressure distributions, the potential value of direct measurements of applied earth pressures is clear.

Talesnick (2005) has recently developed a new sensor to measure contact pressures. The sensor avoids the difficulties of measuring soil pressure due to interaction with a stiff or flexible membrane by eliminating relative deformations. This leads to earth pressure measurements that are independent of soil type, stiffness, density and stress history. The sensing methodology has been implemented successfully in the measurement of soil pressures on small scale laboratory models of buried structures (Talesnick et al. 2007).

A new version of the stress cell has been developed which is small enough to be inserted into the walls of plain, small diameter, HDPE pipes. This article describes the sensor, and reports on its preliminary use in the walls of plain HDPE pipe buried in granular backfill. Measurements of contact pressures are given for burial in both loose and compacted granular material, to illustrate directly the effects of soil compaction on the pattern of earth pressures that develop around buried pipes.

**INSTRUMENTATION**

The sensor consists of two parts; a membrane housing, which is mounted flush to the soil structure interface, and a housing back which provides a hermetic seal of the cylindrical
volume behind the membrane face, Figure F-1 a. The sensitive membrane face is 0.7mm in
thickness and 23 mm in diameter. The design assumes that the soil contact stress over the
dimension of the membrane is uniform. A full bridge diaphragm configuration is adhered to
the under side of the membrane face. Three sets of three holes, are equally distributed around
the flange of the membrane housing. One set of thread holes is used to align the outer face of
the membrane flush with the solid boundary. A second threaded set seals the housing back to
the membrane housing, and the final, unthreaded set is employed to fix the sensor to the solid
boundary. Two holes in the central area of the housing back allow for a sealed electrical feed-
through and a sealed pneumatic feed-through.

The concept of the sensor is based on the null method (Jennings and Burland 1960 and
Doebelin 1990). As the diaphragm begins to deflect under the earth pressures, a response that
is monitored by the diaphragm’s strain gage bridge. To maintain infinite stiffness (i.e.
negligible deformation), air pressure is applied to the cylindrical volume behind the
membrane face, and it is regulated until the outer signal of the diaphragm strain gage bridge is
returned to its initial, undeformed value. This correction is repeated 50 times per second in a
tightly controlled PID loop. The pressure required to null the signal of the diaphragm bridge
can be calibrated to the pressure applied to the outer membrane face: elastic theory dictates
that they are equal with a calibration factor of 1.0. Controlled calibrations have demonstrated
that the concept is effective, Talesnick (2005).

Figure F-1 b shows the preparation of the pipe wall that is required to fit the sensor shown in
Figure F-1 a. An external view of the sensor fastened within the wall of the test pipe is shown
in Figure F-1 c. Each of the four sensors was held in place by a ‘U’ bracket, Figure F-1 d.
Each bracket, in turn, was fastened to the pipe wall using two screws (as seen from the
outside in Figure F-1 c).
Other preparation of the pipe sample included fitting with two linear potentiometers (L.P.’s), oriented across the diameter connecting two of the contact pressure sensors, Figure F-1 d.

**BURIED PIPE TEST MATERIALS AND CONFIGURATION**

The experiments were conducted in the test cell illustrated in Figure F-2. The test cell is 320mm wide, 780mm long and 780m high, made with 8.5mm thick steel plate to limit the side wall deformation. Lexan plates of 12mm thickness were mounted inside each wall of the test cell, and another two L.P.s were used to measure the side wall deformation normal to the wall surface. Under the maximum testing pressure in this test (150 kPa), the deformation of Lexan side wall was about 0.7mm. As designed, the lateral displacement of the vertical wall was small and subsequently it is assumed that the stresses developed in the soil within the box under K₀ (zero lateral strain) conditions.

The backfill material used in each test was uniform (i.e. poorly-graded) sand with a mean grain size of 0.5 mm, uniformity coefficient \( C_u \) of 1.46, and curvature coefficient \( C_c \) of 0.94, Lapos and Moore (2004). Tests were conducted with the sand at two different densities. Tests 1 and 2 were performed with the sand in a loose condition (approximate density of 1.3 t/m³), while tests 3 and 4 were performed with the sand in a dense condition (approximate density of 1.5t/m³).

Vertical pressures were applied to the soil surface at the top of the box by an MTS servo-controlled test system acting on a metal plate placed over three hardwood planks. The experiments reported here were conducted to a maximum vertical pressure of 150 kPa.

Friction between the walls and backfill soil was minimized by placing 3 layers of polyethylene sheets lubricated with high-temperature bearing silicone grease along the walls (Figure F-3). This arrangement may reduce the boundary friction to less than 5º (Tognon et al. 2000). The ends of the pipe were not restrained in the axial direction to avoid interference.
from the walls of the test cell. A ring cut out of polystyrene foam was placed on one end of
the pipe (Figure 1d), to prevent sand from falling into the gap between the ends of the pipe
and the wall of the test cell.

A high density polyethylene (HDPE) pipe (ASTM D3350, Class PE 3408) of internal
diameter 160mm and wall thickness 27.12mm was tested. The contact pressure cells denoted
A, B, C and D were fitted at 90 degree intervals around the pipe circumference as shown in
Figure F-1 d. Tests were performed with the cells placed in two different circumferential
orientations:

I. at the crown (12 o’clock), springlines (3 o’clock and 9 o’clock) and invert (6 o’clock)
   positions

II. at the shoulders (1:30 and 10:30) and haunches (4:30 and 7:30)

The combination of results from tests at both these orientations provides the distribution of
contact pressures around the external pipe circumference. Figure F-4 shows the pipe section
in the test cell placed in orientation II.

The pipe wall was milled to accommodate the pressure sensors, Figure F-1 d, and the external
surface was planed to ensure the flat sensor was flush with the outside of the pipe wall, Figure
F-1 b. The outside of the pipe was trimmed along its length as shown in Figure F-1 c, so that
the sensor surface was level with that part of the external pipe surface. Parallel plate loading
tests were performed on the pipe sample used in the experiments, and stiffness values are
compared in Table D-1 with those for an unperforated pipe sample cut from the same pipe
length. The preparations for the pipe sample reduced the pipe stiffness by 15% for sensor
orientation I, and by just by 4% for sensor orientation II (since orientation II featured stresses
at locations of minimal bending moment).
The contact stresses that develop on the outside surface of the pipe are influenced by the relative stiffness of the pipe and the surrounding soil. Hoeg (1968) introduced an elastic soil-pipe interaction theory, which has been used by many others (e.g. Moore 2001) for estimating earth pressures acting on the pipe, as well as hoop thrusts, bending moments, and pipe deformations. That theory has been used to estimate the magnitude of the possible changes in contact pressures as a result of pipe stiffness. Since this theory represents pipe stiffness as axisymmetric (the same for the pipe in all orientations), it likely provides conservative estimates of the contact pressure changes. The alternative of conducting full 3D finite element analysis of the system, modeling the details of perforations and planed surfaces, is considered unnecessary since the objective of these calculations is simply to bound the likely effect. The calculations have been performed for the two idealized interface conditions considered in the Hoeg (1968) solution, this is smooth (zero shear stress) and bonded (zero slip) interaction between pipe and soil.

The computation indicated that stiffness reduction in the test pipe section leads to larger changes in stress at the springline for both interface conditions, and for both soil densities. Stress changes are 11% and 4% respectively, corresponding to the smooth and bonded interface idealizations. At other locations (i.e. hunch, invert, shoulder, and crown) the calculated change in stress is between 0.05% and 2%. Since the angle of friction between HDPE and soil is at least 13° (Tognon et al., 1999), the interface behaviour is expected to be close to the bonded condition and changes in pipe stiffness are estimated to have influenced stress values by less than 5%.
MEASUREMENTS

Soil Response

The response of a buried flexible pipe is a composite system whose response is dependent on the stiffness of both the pipe and the surrounding soil. Soil modulus has a significant impact on the resulting behaviour, including the earth pressures that develop around the external pipe boundary. During each test, linear potentiometers positioned on the top of the metal loading plate measured the vertical displacement of that plate as vertical force was applied. These displacements were then used to calculate constrained modulus and secant modulus of the backfill soil at each load level assuming that the soil in the test box responded with negligible lateral strain (e.g. Dhar et al. 2004). A constant Poisson’s ratio $v_{soil}$ of 0.33 was employed. Table D-2 presents the modulus values calculated for at four different levels of overburden stress, and for both loose and compacted backfill. Values are provided to four decimal places, so that all calculations of soil-pipe interaction presented in subsequent sections can be reproduced if desired. However, the one dimensional modulus calculations neglect the presence of the test pipe in the box, and the number of significant figures with which these values are provided does not reflect the accuracy of the moduli.

Contact Pressures and Pipe Deformations

The experiments were carried out by increasing overburden pressures up to 150-180kPa, which represents a burial depth of about 10 m. The vertical stress was then monotonically reduced to zero. Contact stress was measured and pipe deflections were monitored continuously throughout each test.

The raw test results are given in the series of graphs shown in Figure F-5 (loose backfill) and Figure F-7 (compacted backfill). Each figure includes the following graphs;

(a) Development of radial pressure at the crown, invert and springlines as a function of applied vertical stress.
(b) Change of vertical and horizontal pipe diameter as a function of applied vertical stress.

(c) Development of radial pressure at the shoulder and haunches as a function of applied vertical stress.

(d) Diameter change across the haunches and shoulders as a function of applied vertical stress.

The response of the pipe buried under loose backfill conditions will be considered first. The development of radial pressure, (Figure F-5 a and c) is for the most part a linear response to the applied vertical stress. This observation is especially true once applied vertical pressures surpass a value of 30kPa, which could be associated with initial seating of the pipe-soil system. The development of radial pressure on the left shoulder position (Figure F-5 c) is a slight exception to this observation. The development of radial pressure at the two springline positions fall almost one on top of the other as is expected due to the symmetry of the problem. Figure F-6 combines the measurements shown in Figure F-5 a, and c. The plots show the development of the radial pressure measured at the crown, invert, and average measurement made at the springlines, haunches and shoulders respectively. In general the radial pressure peaks at the pipe invert, is at a minimum at the springlines, is intermediate and relatively equal at the shoulder and haunch position (Figure F-6). To better define this observation the following procedure was implemented. The slope of the radial pressure versus vertical pressure plot, \( \frac{\nu \sigma_v}{\nu \sigma_v} \), was determined for each circumferential position over the applied vertical pressure range of 30kPa through 110kPa. Table D-3 presents the calculated gradients together with the coefficient of correlation and standard error of the linear correlations. As may be seen the linear approximations are of high quality. Figure F-5 b presents the vertical and horizontal pipe deflections as a function of applied vertical load. The plots are extremely
linear as shown in Table D-3 over the entire range of applied vertical pressure. This outcome, while very common is somewhat confounding considering the expected nonlinear stiffening of soil backfill. The gradient of vertical deflection (shortening) is somewhat greater (20%) than that of horizontal expansion. The deflections measured at angle of 45° to vertical are small as shown in Figure F-5 d.

Figure F-7 presents similar data as that shown in Figure F-5, however for the case of tests performed on pipe sections buried in compacted backfill. In these tests the development of radial pressures are again seen to develop linearly with applied vertical pressure (see Table 3), the springline measurements again fall one on top of the other. Figure F-8 presents the data in the same form as in Figure F-6; development of radial pressure at the crown, invert and average values at the springlines, haunches and shoulders.

Two major differences are evident in comparison to the observations made on the pipes buried in the loose backfill; i) the peak in radial pressure is found at the crown, nearly four times that measured at the invert, and ii) radial pressures measured at the pipe haunches and shoulders (Figure F-7c) do not bunch and display symmetry as noted in the data from the tests performed in loose backfill. This result may be due to several experimental factors; non-uniformity of backfill compaction, misalignment of pipe section, and possible malfunction of at least one of the pressure sensors.

Figure F-9 is an attempt to compare radial pressures for both the loose and compacted backfill conditions. The plots are based on the linear development of radial pressure per unit vertical pressure (Table D-3), where applicable, average values have been plotted. The lines on the plot are not intended to plot out a pressure distribution, rather to accentuate the difference in measurements.
The plot indicates that for the case of the loose backfill, load is transferred down into the invert of the pipe with lower support applied at the haunches, while in the case of the compacted backfill the vertical load is passed down from the crown of the pipe onto the pipe haunches, with only a small portion of the load being transferred to the invert level. These differences in pressure patterns will have a significant effect on the distributions of bending moments, associated curvature changes and bending strains that develop at inner and outer pipe surfaces. Indeed, Rogers et al. (1996) refer to the ‘heart shaped’ and ‘inverted heart shaped’ bending strain distributions, and these correspond to the compacted and dense sand backfill cases respectively.

Two overriding observations are clear from the data;

- In both the loose and compacted backfill conditions the data plotted in Figure F-9 clearly indicates that the distribution of pressure around the pipe circumference is not symmetrical about the horizontal axis.

- The distribution of radial pressure around the pipe circumference in response to loading on the soil surface is clearly dependent upon the nonuniform distribution of backfill support provided to the pipe, placement conditions, and the construction procedures.

Vertical deflection of the pipe section embedded in the compacted fill develops linearly as a function of applied vertical pressure (Figure F-7c). The vertical pipe deflection is approximately 70% of that measured in the case of the loose backfill conditions. The horizontal expansions are marginally smaller (~8%) than the measured vertical shortening.
COMPARISON WITH ELASTIC SOIL-PIPE INTERACTION THEORY

Introduction

The experimental data are now compared to calculated pressures obtained from elastic soil-pipe interaction theory. The analyses were performed for both smooth and bonded interface conditions, constant pipe stiffness (i.e. unperforated pipe properties), and soil moduli for both loose and compacted backfill, as listed in Table D-2. Figure F-10 plots the development of radial pressure at the pipe crown, shoulders, springlines, haunches and invert as computed for applied vertical pressures of 0 through 150 kPa. The plots shown in Figure F-10 a present the computations based on the fully bonded interface assumption for both backfill conditions. Figure F-10 b plots the results of the computations based on the assumption of a smooth soil pipe interface for both backfill conditions. The following initial observations can be made;

1. The plots illustrate that the radial pressure computed at any specific position around the pipe perimeter is a linear function of the applied vertical pressure and does not appear to be influenced by increases in soil modulus as stresses increase. This outcome is in agreement with the experimental data presented in Figures D-5a and c and Figures D-7a and c.

2. The plots illustrate that the computed distribution of radial pressure is symmetrical about the horizontal pipe axis. The radial stress computed at the invert match those at the crown, likewise the radial pressure computed at the shoulders match those computed at the haunches. These observations remain true irregardless of the assumptions made pertaining to backfill stiffness and interface roughness.

3. The plots illustrate that the development of radial pressure computed at any position around the pipe circumference is only mildly affected by changes in backfill stiffness. The moduli used in the computations for the compacted backfill are roughly 20% higher than their loose counterparts, yet the effect on the magnitude of the radial pressure is negligible.
The observations noted in points 2 and 3 are in contradiction to the experimental data presented in Figures D-5, D-7 and D-9 which clearly illustrate the asymmetry of radial pressure about the horizontal axis of the pipe. Furthermore, the dependence of the distribution of radial pressure on embedment/placement conditions is not detected in the computed radial pressures.

4. The two different interface assumptions have a profound effect on the computed development of radial pressure. In comparison to the bonded interface assumption, the smooth interface assumption results in more extreme radial pressures, higher pressure is computed at the crown and invert and lower pressure at the springlines. The two assumptions result in identical pressures at the haunch and shoulder positions.

Since the computed development of radial pressures plot as linear functions of vertical stress (Figure F-10) it is fair to compare the slopes of these plots to those determined from data shown in Figures D-6 and D-8 and tabulated in Table D-3.

**Comparison for Loose Backfill Condition**

The graphs shown in Figure F-11 compare the development of radial pressure at the crown, invert, springlines, shoulders and haunches as measured in the experiments performed in the loose backfill to values computed from elastic soil-pipe interaction theory. The computed results shown in Figure F-11a are based upon the assumption of a smooth interface, while those of Figure F-11b are based upon the fully bonded condition. The plots shown for the pressures measured at the springlines, shoulders and haunches are based on the average measured value.

Examination of the plots and values tabulated in Table D-3 reveal the following observations:
neither assumption results in a good fit to the entire set of measured data.

the smooth interface assumption provides an effective estimate of radial pressure at the invert, haunches and shoulders, however results in very poor estimates at the springlines (28% of the measured magnitude) and crown (1.84 times the measured value).

the bonded interface assumption results in a much better fit at the springlines, but fails to produce reasonable radial pressures at either the invert (81% of measured magnitude) or crown (1.43 time measures value) positions. The fit at the haunches and shoulders is identical to that noted for the smooth interface assumption.

Figure F-12 shows a comparison between the measured and computed pipe deflections for the loose backfill conditions. The difference between the deflections, computed based on the different interface assumptions is negligible, therefore for efficiency sake only those based on the bonded assumption have been plotted. The plots illustrate a very poor correlation between the two. The computed deflections are consistently smaller, by approximately 40% than the measured deflections. This situation is unsatisfactory since it could be implied that the discrepancies pointed out in the pressure distribution could in fact be based in the choice of soil modulus. In order to investigate this possibility the soil moduli shown in Table D-2 where adjusted (lowered) in order to induce a good correlation between the measured and computed vertical pipe deflections. A comparison between the measured pipe deflections and the computed pipe deflections (bonded solution) is shown in Figure F-12. The linear form of the computed plots fits very well to the measured ones, but more important is the very reasonable estimate of the horizontal pipe deflection. The magnitude of the soil moduli adjusted to force the fit between the computed and measured vertical pipe deflections are listed in Table D-2. The adjusted moduli tend toward the original values at higher vertical pressures. When applying the adjusted moduli in the elastic soil-pipe interaction computations the radial pressures on the pipe circumference change only marginally. This result should not
come as a surprise as the insensitivity of the distribution of radial pressure to soil modulus was clearly demonstrated in Figure F-10.

**Comparison for Compacted Backfill Condition**

Figure F-13 compares measured pressures to computed values in the same format as that shown in Figure F-11, for the condition of compacted backfill. The following observations are made:

- neither interface assumption results in a good fit to the entire set of measured data.
- the smooth interface assumption results in a good fit at the crown and haunch positions: however grossly over-estimates the radial pressures at the invert by 4.15 times the measured value, and at the shoulder by 1.51, while under-estimating the radial pressure at the springline (33% of the measured magnitude).
- the bonded interface assumption results in a much better estimate at the springlines and a reasonable fit at the crown; however the values estimated at the shoulders and invert are poor.

A comparison between the measured and calculated pipe deflection for the case of compacted backfill condition is shown in Figure F-14. As in the case of the loose backfill condition, the computations based on the original soil modulus result in an under-estimate of vertical shortening and accompanying horizontal expansion. Reduction of the soil moduli to values shown in Table 2 yields a good match of both the vertical and horizontal diameter changes, however not as consistent as for the case of the loose backfill (Figure F-14).

**DISCUSSION AND CONCLUSIONS**

New contact pressure sensors have been described which permit measurements of radial contact stresses around a buried HDPE pipe under static vertical loading. The new sensors
were developed to overcome the influence of sensor deformation on the contact pressure measurement, making measurements independent of the soil and sensor stiffnesses. The use of the instrumentation has been demonstrated through tests undertaken to measure pressures on a plain HDPE pipe buried in both loose and compacted granular material.

Figure F-15 is an attempt to illustrate the differences between radial pressures measured at different locations on the pipe circumference and computed pressure distributions. The magnitude of pressure is presented as the gradient of radial pressure per unit applied stress. The measurements, computations and comparisons of radial pressure presented above as illustrated in Figure F-15 infer the following:

1. The measured soil pressures illustrate that the distribution of radial pressure on the pipe is not symmetrical about the horizontal axis. This finding is very clear when comparing pressures measured at the crown and invert positions for both loose and compacted backfill conditions. This finding is at odds with the basic assumptions of classical elastic soil-pipe interaction theory, which implies symmetry of the distribution of radial pressure about the horizontal pipe axis.

2. The measured soil pressures indicate that radial pressure and its distribution around the pipe is sensitive to the combination of soil stiffness and embedment process. The magnitude and distribution of radial pressure computed from elastic soil-pipe interaction theory is only marginally sensitive to soil stiffness (within the range of soil stiffness tested). The effect of embedment process/installation procedures cannot be modeled by the elastic soil-pipe interaction solution of Hoeg (1968).

3. It is clear that the assumption of uniform ground support used in the derivation of Hoeg’s theory is invalid and an alternative analysis or some form of empirical adjustment is needed if the observed pattern of earth pressures is to be considered. This conclusion is in agreement with other studies which have illustrated indirectly (e.g. Rogers et al. 1996 and Brachman et al. 2008) that the distribution of radial pressure is more complex than that assumed in elastic soil-structure interaction.
analyses.

REFERENCES


Table D-1  Comparison between the stiffness of unperforated and perforated pipe sections.

<table>
<thead>
<tr>
<th>Soil Condition</th>
<th>Applied Pressure (kPa)</th>
<th>Ms (MPa)</th>
<th>vsoil</th>
<th>Es (MPa)</th>
<th>Adjusted Modulus Es (MPa)</th>
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</thead>
<tbody>
<tr>
<td>Unperforated pipe</td>
<td>10.78 N/mm²</td>
<td>8.36 N/mm²</td>
<td>10.3 N/mm²</td>
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<td></td>
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<tr>
<td>Perforated pipe at orientation I</td>
<td>9.76 N/mm²</td>
<td>8.29 N/mm²</td>
<td>9.36 N/mm²</td>
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<td></td>
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<tr>
<td>Perforated pipe at orientation II</td>
<td>7.20 N/mm²</td>
<td>6.03 N/mm²</td>
<td>6.98 N/mm²</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table D-2 Soil stiffness at different levels of applied vertical pressure.

<table>
<thead>
<tr>
<th>Soil Condition</th>
<th>Applied Pressure (kPa)</th>
<th>Ms (MPa)</th>
<th>vsoil</th>
<th>Es (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Backfill</td>
<td>0-50</td>
<td>4.156</td>
<td>0.33</td>
<td>2.764</td>
</tr>
<tr>
<td></td>
<td>50-100</td>
<td>5.628</td>
<td>0.33</td>
<td>3.743</td>
</tr>
<tr>
<td></td>
<td>100-150</td>
<td>7.156</td>
<td>0.33</td>
<td>4.758</td>
</tr>
<tr>
<td>Compacted Backfill</td>
<td>0-50</td>
<td>6.052</td>
<td>0.33</td>
<td>4.025</td>
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<tr>
<td></td>
<td>50-100</td>
<td>7.918</td>
<td>0.33</td>
<td>5.265</td>
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<tr>
<td></td>
<td>100-150</td>
<td>9.191</td>
<td>0.33</td>
<td>6.112</td>
</tr>
</tbody>
</table>
Table D-3  Linear regression of measured pressures and deformation of the pipe section

| Position      | Loose Backfill | | | | | Compacted Backfill | | | | | | |
|---------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
|               | $\sigma$, kPa  | $\Delta \sigma_v / \Delta \sigma_v$ | $r^2$ | Error kPa, mm | $\sigma$, kPa  | $\Delta \sigma_v / \Delta \sigma_v$ | $r^2$ | Error kPa, mm |
| Crown         | 30-110         | 0.884          | 0.993         | 1.7           | 10-150         | 1.478          | 0.995         | 4.19           |
| L. Springline | 30-110         | 0.595          | 0.997         | 0.75          | 10-150         | 0.591          | 0.998         | 1.18           |
| R. Springline | 30-110         | 0.616          | 0.989         | 1.5           | 10-150         | 0.504          | 0.998         | 2.72           |
| Av. Springline| 30-110         | 0.605          | 0.998         | 0.65          | 10-150         | 0.547          | 0.997         | 1.26           |
| L. Shoulder   | 30-185         | 0.803          | 0.994         | 3.0           | 25-145         | 0.466          | 0.997         | 0.87           |
| R. Shoulder   | 30-185         | 0.869          | 0.997         | 2.4           | 25-145         | 0.710          | 0.999         | 0.58           |
| Av. Shoulder  | 30-185         | 0.836          | 0.999         | 1.36          | 25-145         | 0.588          | 0.999         | 0.51           |
| L. Haunch     | 30-185         | 0.817          | 0.994         | 3.1           | 25-145         | 0.544          | 0.987         | 2.12           |
| R. Haunch     | 30-185         | 0.908          | 0.988         | 5.0           | 25-145         | 1.071          | 0.999         | 1.34           |
| Av. Haunch    | 30-185         | 0.863          | 0.998         | 1.91          | 25-145         | 0.807          | 0.999         | 0.74           |
| Invert        | 30-110         | 1.547          | 0.997         | 1.7           | 10-150         | 0.386          | 0.999         | 0.39           |
| $\Delta \delta_v / \Delta \sigma_v$ | 2.5-149 | -0.0203 | 0.021 | 2.9-150 | -0.0149 | 0.010 |
| $\Delta \delta_h / \Delta \sigma_v$ | 2.5-149 | 0.0147 | 0.021 | 2.9-150 | 0.0137 | 0.013 |
a. Back of cell and front contact surface  
b. Preparation of pipe wall  

c. Location of pressure cell in external surface  
d. Four sensors each held by ‘U’ bracket
e. Pipe end showing foam ring (part missing after test), ‘U’ brackets holding the sensors, and the linear potentiometer.

Figure F-1  Details of stress cell placement in plain HDPE pipe
Figure F-2: Test cell testing setup

Figure F-3 Side wall friction treatment

Figure F-4 Pipe in test cell after exhumation (shown with sensors in Orientation II, i.e. at shoulders and haunches)
Figure F-5 Results of tests performed in loose backfill;
(a) measured contact pressures, orientation I.  (b) measured pipe deflections, orientation I.

(c) measured contact pressures, orientation II.  (d) measured pipe deflections, orientation II.
Figure F-6 Synthesis of measured contact pressures (loose backfill).
Figure F-7 Results of tests performed in compacted backfill:
(a) measured contact pressures, orientation I.  (b) measured pipe deflections, orientation I.

(c) measured contact pressures, orientation II.  (d) measured pipe deflections, orientation II.
Figure F-8 Synthesis of measured contact pressures (compacted backfill). Ave. M - denotes average measured.

Figure F-9 Comparison of measured pressure in loose and compacted backfill as a function of position around pipe circumference.
Figure F-10  Computed radial pressures as a function of vertical pressure and position;

(a) smooth interface assumption. (b) bonded interface assumption.
Figure F-11 Comparison of computed and measured contact pressures in loose backfill;
(a) smooth interface assumption. (b) bonded interface assumption.
C. – denotes computed.
Figure F-12 Comparison of computed and measured pipe deflections in loose backfill.
Figure F-13 Comparison of computed and measured contact pressures in compacted backfill; 
(a) smooth interface assumption. (b) bonded interface assumption.
Figure F-14 Comparison of computed and measured pipe deflections in compacted backfill.

Figure F-15 Computed and measured radial pressure in loose and compacted backfill as a function of position around pipe circumference.