Post-Installation Behavior of High-Density Polyethylene Pipe

Submerged in Saturated Silty Soils

By

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ABSTRACT

The thesis examines how high density polyethylene (HDPE) pipe installed by horizontal directional drilling (HDD) and traditional open trench (OT) construction techniques behave differently in saturated soil conditions typical of river crossings. Design fundamentals for depth of cover are analogous between HDD and OT; however, how the product pipe is situated in the soil medium is vastly different. This distinction in pipe bedding can produce significant differences in the post installation phase. The research was inspired by several incidents involving plastic pipe installed beneath rivers by HDD where the pipeline penetrated the overburden soil and floated to the surface after installation. It was hypothesized that pipes installed by HDD have a larger effective volume due to the presence of low permeability bentonite based drilling fluids in the annular space on completion of the installation. This increased effective volume of the pipe increases the buoyant force of the pipe compared to the same product diameter installed by OT methods, especially in situations where the pipe is installed below the ground water table. To simulate these conditions, a real-scale experiment was constructed to model the behavior of buried pipelines submerged in saturated silty soils. A full factorial design was developed to analyze scenarios with pipe diameters of 50, 75, and 100 mm installed at varying depths in a silty soil simulating an alluvial deposition. Contrary to the experimental hypothesis, pipes installed by OT required a greater depth of cover to prevent pipe floatation than similarly sized pipe installed by HDD. The results suggested that pipes installed by HDD are better suited to survive changing depths of cover. In addition, finite element method (FEM) modeling was conducted to understand soil stress patterns in the soil overburden post-installation. Maximum soil stresses occurring in the soil overburden between post-OT and HDD installation scenarios were compared to understand the pattern of total soil stress incurred by the two construction methods. The results of the analysis showed that OT installation methods triggered a greater total soil stress than HDD installation methods. The annular space in HDD resulted in less soil stress occurring in the soil overburden. Furthermore, the diameter of the HDD annular space influenced the soil stress that occurred in the soil overburden, while the density of drilling fluids did not vastly affect soil stress variations. Thus, the diameter of the annular space could impact soil stress patterns in HDD installations post-construction. With these findings engineers and designers may plan, design, and construct more efficient river-crossing projects.

DEDICATION

To my wife and lovely daughters, Anje and Anna

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Chapter 1: INTRODUCTION

1.1 Introduction

Horizontal pipeline installations crossing rivers are increasingly important for the efficiency of utilities transportation (i.e. natural gas, drinking water, communication, and oil transmission so on) (Lixin et al 2011). For river crossings, traditional open trench (OT) and horizontal directional drilling (HDD) are common construction methods. Traditional OT is a typical excavation method and has three steps: 1) excavating surface soil to place pipeline, 2) installing pipeline, and 3) backfilling and consolidating. Alternatively, HDD is a trenchless technology and does not disrupt the soil surface. HDD also has three steps: 1) horizontally boring the pilot hole, 2) reaming to enlarge the pilot hole, and 3) doing pullback to place and secure the pipe (see the definition of these steps in Chapter 2). Traditionally pipelines have been installed using OT methods by dragline, or various isolation and excavation techniques due to the relative cost advantages provided by these methods (Velman 2008). However, in light of increasing environmental concern and regulations, today an increasing number of pipeline projects, especially river-crossing projects, are being completed by HDD methods, which is a more environmentally-friendly pipeline installation method.

While the basic design principles for depth of cover are similar (minus consideration for borehole pressure analysis) between HDD and OT construction techniques, how the product pipe is situated in the soil medium is vastly different. For OT installations, the pipe is typically bedded in either the original excavated native material or buried beneath an engineered backfill. Alternatively, for HDD the pipe is installed in a drilled borehole that provides an annular space containing a mixture of drilling fluid and native soil cuttings, referred to as "bedding" (Figure 1.2). This distinction in pipe bedding could produce significant differences related to the post installation behavior of the pipeline. Subsequently, examining the behavior of buried pipelines under conditions typical of river crossings could be valuable in order to contrast these different construction techniques.

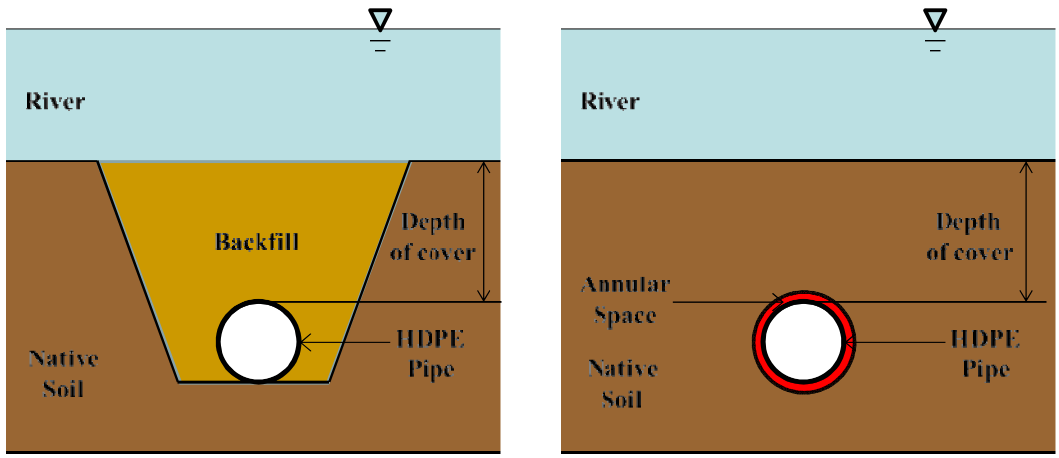

Figure 1.1 Different Bedding between OT and HDD in Post-Installation

One of the most important factors in pipeline design is to determine the appropriate depth of cover. Designing the depth of cover for a pipeline is affected by several factors including: soil properties, pipe material and geometry, and flow characteristics of the water body; particularly scour depth. Using these factors, the depth of cover must be accurately designed and installed to prevent serious damage or failure of the buried pipeline. In the rainy season, flooding or sudden heavy rains can trigger pipeline incidents and subsequent damages because of a decrease in depth of cover (Wang et al 2010). Therefore, the scour depth of a river (the depth to the bottom of the scour zone) is important to determine since the underground pipeline could be exposed due to lack of cover that may be caused by flood or inundation (Wang et al 2010). In this research, depth of cover was utilized as H/D ratio, which is the embedment ratio between the depth of cover and the diameter of the buried pipe.

This dissertation focuses on the behavior of buried High Density Polyethylene (HDPE) pipes installed by both traditional OT and HDD methods, and the critical depth to diameter ratio (H/D ratio) required for pipe floatation to occur. Results from the OT and HDD trials were compared with each other to better understand how pipes installed by these methods behave post construction. Furthermore, theoretical results were calculated and compared to the experimental results. This comparison and analysis will lead to a better understanding of the behavior of underground pipeline installations in saturated silty soils.

1.2 Research Objectives

The first objective is to understand how pipes installed by traditional open trench (OT) and horizontal directional drilling (HDD) behave in soil indicative of river crossings. A pipe buried beneath the natural water table creates unique problems and challenges regardless of construction technology. During periods of saturation, pipeline segments may become buoyant, even when they are filled with contents after pipeline operation. Pipe floatation can also happen usually shortly after burial when the backfill material is least dense (Schupp et al 2006). In actual river crossings, the aim of the design is to minimize the risk of pipeline exposure (Veldman 2008). For this situation, both OT and HDD, common construction methods for river crossings, are utilized as mentioned above. To minimize the risk, it will be valuable to learn the behavior of a buried pipe installed by the two construction methods.

The second objective is to compare the theoretical buoyancy behavior of pipes installed by OT and HDD pipe to real scale laboratory trials. The contractor and engineers determine an appropriate depth of cover, considering soil properties pipe materials, and river flow characteristics. They must then confirm whether or not a buried pipe with the determined cover depth is safe using the pipe buoyancy formula made by Archimedes' theory (Hahn 1988). In order to determine the reliability of the buoyancy theory, the outcomes simulated by a laboratory experiment should be compared to those calculated by the buoyancy formula that is being utilized in the real design of pipe installations as a safeguard against pipe floatation. This comparison is used to analyze the causes for inconsistency between the theoretical and laboratory results if there are any. The results from this research could be useful for engineers to estimate the required depth of cover in pipeline installations at water crossings.

The third objective is to determine how drilling fluid in the annular space of an HDD installation affects pipe floatation. During HDD installation, the annular space is created and filled with drilling mud (mostly bentonite). This annular space is the space between the outer diameter of a buried pipe and the borehole wall. This annular space is a distinguishing element between OT and HDD because OT installation has no annular space. Through this research, the role of the annular space will be revealed for pipe buoyancy behavior.

The fourth objective is to define a robust model for pipe buoyancy in river crossings through three analytical methods: theoretical, experimental, and numerical. A numerical calculation method (finite element method; FEM) has frequently been utilized in academic research and industrial design. The analysis through three analytical methods (i.e. theory, experiment, and FEM) would help defining the pipe buoyancy phenomenon in river crossings.

The last objective is to determine what impact these results could have on the design and construction of river crossings by analyzing various buoyancy factors found in design and construction process of the OT and HDD method. These acquired results from this analysis could be helpful for an actual design and construction of river-crossing pipeline.

1.3 Research Scope

Research relative to the buoyant behavior of underground high-polyethylene pipe (HDPE) is needed since very few studies have been conducted in this area (Polak 2005). This dissertation provides experimental data to increase current understanding of the behavior of pipelines installed by both OT and HDD at river crossings.

The tasks included in this research scope are as follows:

a) Improve upon the study of buried pipeline behavior at river crossings using data from previous studies associated with pipeline behavior in saturated silty soils in order to motivate the need for additional pipeline behavior studies.

b) Ascertain discrepancies in pipeline behavior between traditional OT and HDD in saturated silty soils post-installation.

c) Identify the factors that contribute to traditional OT and HDD pipe buoyancy post-instillation.

d) Apply pipe buoyancy theory to simulate riverbed status for each construction method.

e) Design a laboratory experiment to test the factors found in the research. Using this information, identify variables, parameters and constants and determine the most critical factors that influence pipeline behavior.

f) Identify previous literature for experimental buried pipeline behavior through observation of a pipeline in saturated silty soils and utilize this information to accurately describe the behavior of a buried pipeline at river crossings in a real-scale experiment.

g) Compare the results obtained using buoyancy theory to those found in laboratory tests to identify any differences.

h) Conduct a numerical calculation using finite element method (FEM) and define the soil stress phase affected by varying depth of cover obtained from laboratory tests.

i) Determine the total soil stress pattern by varying the critical design parameters (i.e. diameters and density of the annular space) of the HDD design in order to understand how these parameters affect the relationship between the soil and the pipeline.

j) Determine general trends from the results, define the buoyancy behavior of pipe for river crossings, and discuss limitations and recommendations in order to make a more robust model for buried pipeline buoyancy.

1.4 Thesis Organization

Chapter 1 provides an introduction to the research motivation, background, objectives, and recent trends related to our research. This chapter clarifies why this research is needed.

Chapter 2 provides a detailed description of pipeline water crossings including several installation methods and typical riverbed soil. It also describes the methodologies used for analysis, such as theoretical methods (buoyancy theory) and experimental designs, and previous research on pipe floatation. An introduction to the horizontal directional drilling industry is described including construction procedures and comparison to traditional open trench (OT). Previous research advanced three research directions for understanding the pipe buoyancy phenomenon. The first research direction was the relation between soil reaction and underground infrastructure. The second research direction was technical research related to pipe buoyancy affected by the external forces (i.e. longitudinal buckling, liquefaction). The final research direction was the effect of the annular space for horizontal direction drilling (HDD) installation. This research anticipates that the existence of the annular space in HDD method could be the critical element to dominate the results for the pipe floatation research.

Chapter 3 describes the experimental setup that was designed for observing the behavior of buried pipelines in saturated silty soils by using different construction methods. While designing the experiment, we found various influential factors that must be considered in the final analysis. The experimental procedure was modeled on an actual construction procedure. Chapter 3 also provides the method for how to simulate the annular space for the HDD method.

Chapter 4 shows the results and observations from the laboratory tests for pipe floatation. Experimental results are presented for each diameter of pipe and construction method. General trends of pipe floatation were observed through analysis of the results obtained from the experiment and are presented. The comparison between laboratory results and theoretical results for OT and HDD are presented.

Chapter 5 provides the results from the numerical analysis using finite element method (FEM). FEM has been a reliable numerical method for describing a phase of mechanical soil behavior by creating a mesh model, which simulates actual three dimensional (3D) non-linear models for pipe installation section. The software that is utilized in this research is ABAQUS 6.10. Experimental results obtained in Chapter 4 are preprocessed to allow input into the mesh modeling for FEM. The acquired results from laboratory tests were used as input for FEM. FEM results show the stress pattern of soil overburden in both OT and HDD post-installation at critical depths of cover obtained by laboratory tests. Moreover, the results can be used to create a robust model for pipe behavior at river crossings. For the HDD method, FEM was utilized in finding the influence of the critical design parameters for the annular space as well.

Chapter 6 contains the research summary and conclusions developed from theoretical and experimental results, and numerical modeling programs in this research. Also, Chapter 6 presents the contributions and implications of the research associated with its findings. For the future research, Chapter 6 describes limitations and recommendations of this research as well.

Chapter 2: BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

This chapter starts to describe the pipeline construction methodology of river crossings. The construction methodology of river crossings is very important to comprehend because it is expected that pipeline construction, materials, procedures, and equipment utilized in actual pipeline installations could influence pipeline behavior in saturated soils during or after construction. All the factors that influence pipe buoyancy in saturated soils can be determined through investigating construction methods of river crossings. Hence, understanding this part in detail will help to analyze the results of pipeline behavior afterwards. There are two watercourse construction methodologies for underground excavation; open trench (OT) and trenchless technology (TT). For OT, four types of pipeline construction method are described in this chapter. This chapter only described horizontal directional drilling (HDD) for TT because HDD was chosen for this research. The detail procedure of each construction method offers the understanding of the watercourse pipeline method, which assists in clarifying the difference between the two pipeline installation methods. Furthermore, the buoyancy theory was described as the theoretical method for this research. The buoyancy theory describes how to calculate the buoyancy impact depending on different construction methods. The theoretical method was separately described by two construction methods. Lastly, this chapter described previous research for pipe floatation test to verify that our research is creative and valuable. By grasping the trend of previous pipe floatation research, a more accurate research scope and direction will be found.

2.2 Construction Methodologies for River Crossings

As mentioned above, there are many factors (i.e. river depth, water flow during construction, discharge, scour depth, field space, environmental constraints, and streambed conditions) affecting the final decision of construction methodologies (ENGP 2011). The definite point to decide suitable construction methodologies is how well this chosen method could minimize the risk of pipeline failure or exposure (Veldman 2008). Due to several restrictions related to construction regulations from each site, the number of detail factors should be considered when determining the most suitable watercourse crossing methods (ENGP 2011). Here is a list of the main considerations:

(1) Influence of fish and fish habitat (including the species and life stages) at the crossing location during the time of construction

(2) Construction consideration: complexity, risk, safety, schedule and cost

(3) Geotechnical consideration: feasibility of trenchless method, the stability of the valley slopes and the risk of debris flow

(4) Hydrologic consideration: flow volumes, channel stability

(5) Dealing with aboriginal group, regulations, community, and stakeholders

(6) Temporary and permanent access requirements

(7) Pipeline operation and integrity

(8) Maintenance

(9) Prevention of wildlife habitat

A river classification system is a particularly crucial consideration for designers and engineers to decide the underground construction methodologies of pipeline excavation. In real river crossings' projects, there are four types of river classifications to decide the construction methodologies (MGS 2004). The first type is the size of the river or stream. This first type is regulated in a water channel that has a lasting flow and a drainage area greater than 1,000 km^2. The second is the channel with lasting flow less than 1,000 km^2 or partially frozen to the channel bed during winter. The third is a water channel that is frozen to the bed and does not flow in winter. The last one is a transient water crossing, such as a swale and depression. This classification of water course has flow only during spring runoff, and there are no apparent banks or evidence of annual sediment transport. Conveniently, the first classification could be called "Large", the second "Medium", the third "No flow", and the last "Transient."

After and during the period of construction, the elevation of downstream sediment loads are caused by open trench crossings. Thus, in order to avoid the sediment related environmental impacts, the alternate crossing techniques have been utilized in river crossing installation. In open trench method, an environmental restriction in construction sites leads an innovative method, "Isolation method." Isolation method is used to reduce the environmental and ecological damage caused by reckless excavation. This isolation method is also divided into two detailed ones depending on the construction site situation: isolation with dam and flume and isolation with dam and pump. In the actual river crossing project, determining the suitable method of river crossing construction was confirmed by the watercourse classifications mentioned above. Figure 2.1 below shows the decision making chart depending on these watercourse categories.

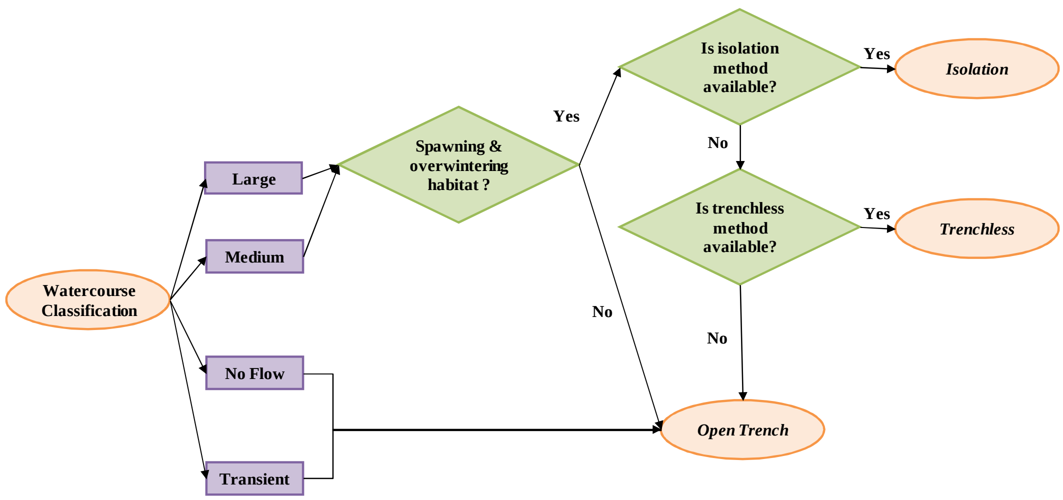

Figure 2.1 Construction Method Selection Diagram of River Crossings by Water Classification (MGS 2004)

In summary of decision making for construction methods, the size of rivers or streams with steady flow should be considered to minimize environmental damages, such as spawning and overwintering habitat. Thus, if the site is restricted by this regulation, then an engineer must consider the isolation methods first. If this method is feasible for that project, then it will be determined as the best method. However, it is not feasible at the project site, another option could be trenchless. The suitable method for the third and forth watercourse classification (No Flow and Transient) is a conventional open trench.

Another way for selecting the best method of river crossings could exist, but this way has generally been utilized in actual projects. Checking the feasibility of the construction technique proposed for the various water crossings depends on site studies and the actual construction conditions at each crossing. The next section describes the detail methods and procedures of each construction methodology.

2.2.1 Open Trench (OT)

2.2.1.1 Conventional Open Trench Technique

As shown in the Figure 2.1, open trench (OT) has four types: conventional excavation and isolation methods (dam and flume, dam and pump, and coffer dams). The traditional OT method (conventional excavation) at river crossings has the same procedure of ground pipeline projects: cutting the subsurface of the riverbed, lowering the pipeline using a crane, and backfilling. The trench is generally excavated and backfilled by either a backhoe, or a dredge. If the width of the river is large, then the *Sauerman* dragline system is mostly utilized for excavating work. This system consists of a tower or a crane connecting draw-cables hooked with buckets (see Figure 2.2) that move back and forth to excavate and dump repeatedly. It is suitable for excavating wet materials, such as dirt, mud, muskeg, blast rock, sand, gravel, large ditches, lagoons, and pipeline river crossings.

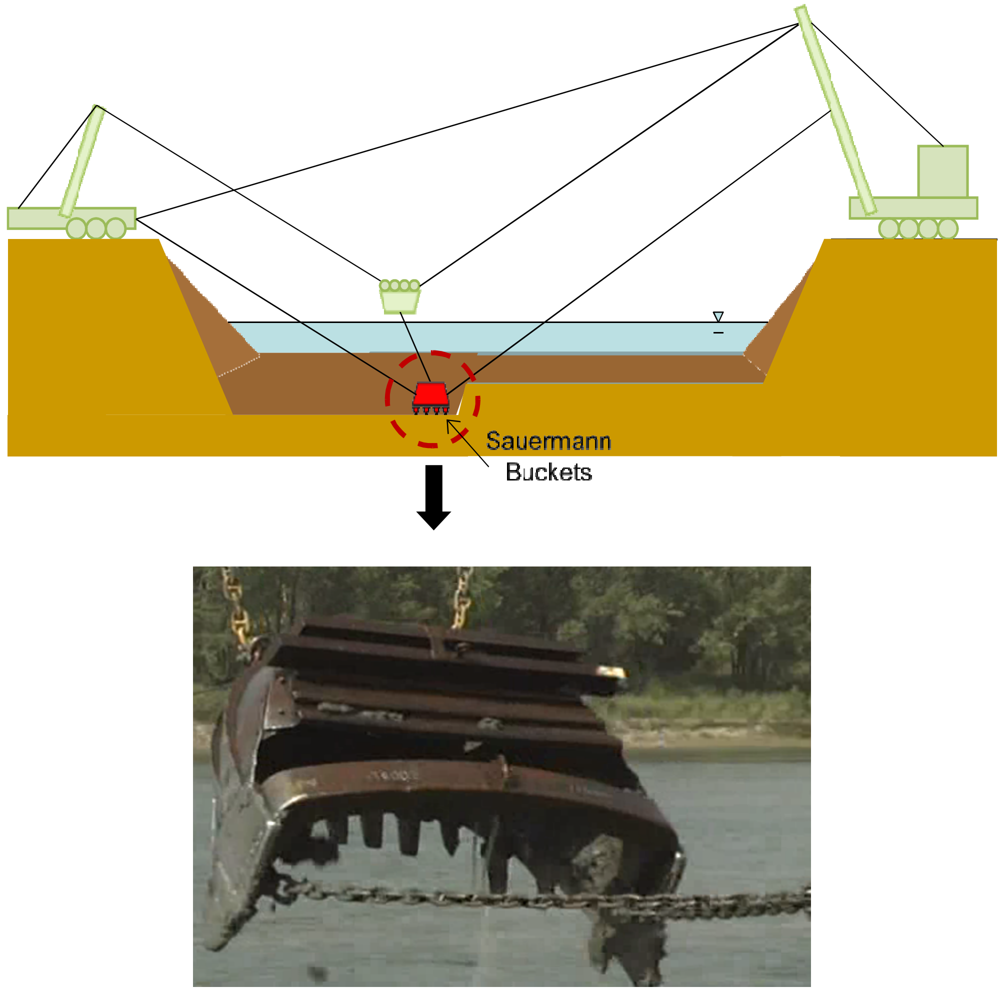

Figure 2.2 *Sauermann* Dragline Systems (Top) and Bucket (Right)

The conventional excavation method is a pipeline installation method without any isolation or diversion of flow away from the work area. In Figure 2.1, this conventional excavation method is chosen when the river or stream has no flow or there is a swale or depression site situation. Also, if there is no overwintering or spawning in watercourse, this conventional OT could be accepted. However, this conventional OT could be the terminal method unless other construction techniques of water crossing are available. This conventional

OT crossing technique is utilized to limit the duration of stream activity, because this method could minimize the period of instream activity. Benefits include rapid construction, mostly stable streamflow, and a relatively short duration of sediment release. On the contrary, the disadvantage of this method obviously is that it causes too much streambed and riparian vegetation disturbance, and high sediment release potential. So, the conventional OT method must be used in non-sensitive watercourses that do not have an instream recovery, assessment, and planning (RAP) program.

2.2.1.2 Isolation Method (Dam and Pump)

Among OT installation methods of water crossings, the isolation methods were developed to minimize the release of sediments at the river. The first isolation method is dam and pump. If it is expected that a watercourse in a project site has flow through a location or is concerned with the potential issue of fish and fish habitat, the watercourse needs the isolation methods. Among those, the isolation with dam and pump is to dam the watercourse to excavate while maintaining clean water flow around the crossing location using pumps. The construction crew must make sure there is enough workspace for spoils, pipeline work, and accessibility for both crossing sides. When the site has meandering channels and irregular streambeds with low flow, this isolation with dam and pump is a more suitable method than flumed crossing (Reid et al 2004), which will be described in the next section. This dam and pump method is good for minimal sediment release, but it is restricted to the installation and removal of dams. Also, the way of fish salvage is required in the watercourse location, and river banks and riparian vegetation must be restored. Figure 2.3 below is an isolation method with dam and pump.

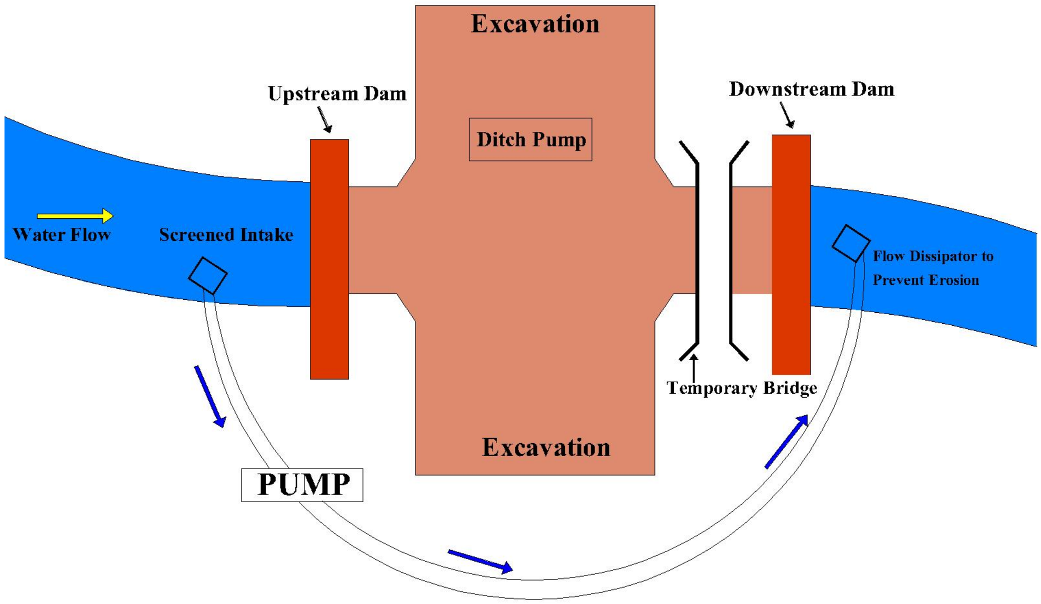

Figure 2.3 Isolation Method with Dam and Pump

2.2.1.3 Isolation Method (Dam and Flume)

The strict environmental regulations for construction methods of river crossings and waterways have been enforced since the mid 1980's. The concerns of damaging the fish habitat and spawning beds, which are ruined by construction excavation, brought the renovated open trench methods to minimize the damage of the ecosystem in river and stream environments. Methods of construction employing water dams or flumes (or a combination of both) are used to isolate the construction activity from the waterway in an effort to eliminate or minimize the discharge of silt. Fluming, on its own, is the simplest method of stream diversion and usually the most economical. Figure 2.4 below briefly depicted an isolation method with dam and flume.

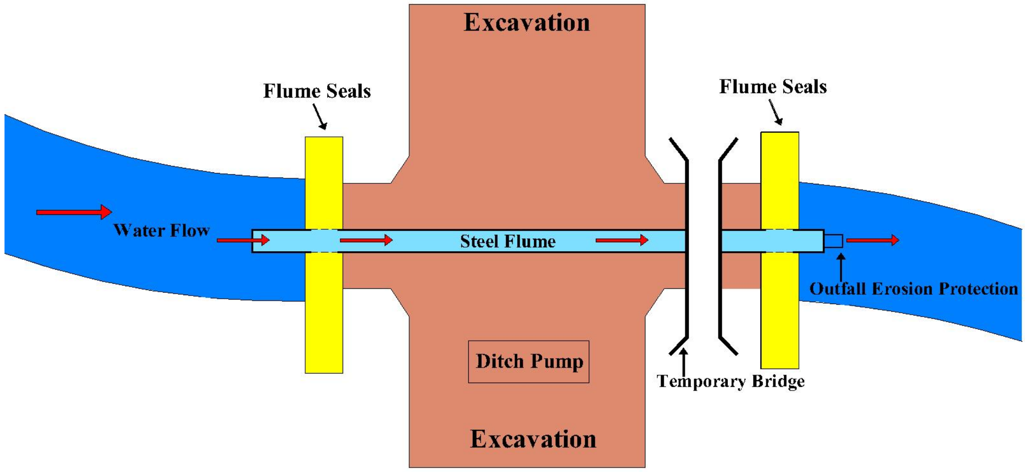

Figure 2.4 Isolation Method with Dam and Flume

The watercourse is intercepted and diverted through a suitably sized pipe or carrier. However, if the river flows are too large, diversion by means of fluming becomes impractical. As similar with dam and pump method, this method also requires the salvaging of fish and the restoration of riparian vegetation. Construction should be careful to not block or damage the flume pipe as well. These isolation methods, including dam and pump, are best suited for watercourse construction in environmentally sensitive sites. However, these methods are restricted depending on the size, flow, speed, and volume, and construction duration of watercourse. In particular, the normal flumed method cannot cover over 4 m^3/s of the volume of flow. The superflume, which is a newly developed approach to cross sensitive watercourses, has been recently developed to cover the larger capacity of flow volume estimated by 10 m^3/s (Reid and Anderson 2000). However, this method could cause a large volume of water seepage during crossings. Hence, the volume of water must be pumped from the work site quickly. The method is required to monitor for containment ponds or discharge locations. This is much fitted with large diameter pipeline crossings unless Horizontal Directional Drilling (HDD) is applicable due to economical or construction risky reasons. Thus, isolated crossing techniques could be the remarkable way for protecting aquatic environments.

2.2.1.4 Isolation Method with Cofferdams

Two isolation methods mentioned above are mostly utilized in small size of river or stream. Instead, isolation method with cofferdams is selected for big size of river or stream. This isolation method is to construct a cofferdam that builds a partial island in river for in-stream work. Figure 2.5 below is shown the isolation method using cofferdam for a big scale river pipeline work.

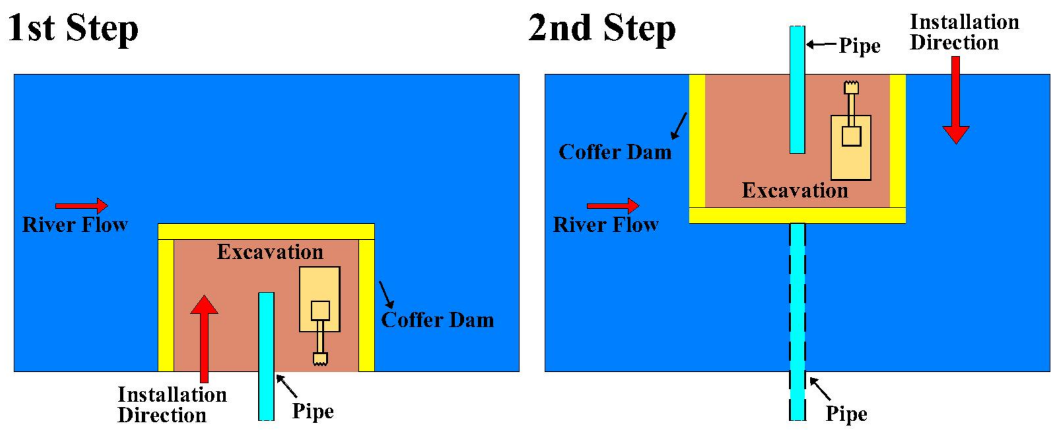

Figure 2.5 Isolation Method with Cofferdam for Big River

In Figure 2.5, the partial area in the first step is isolated and allows in-stream work for pipeline installation. River flow is allowed to continue in the remainder of the channel. After the first step is completed, the second step is to make a cofferdam for the isolation at the other side of river, lower the pipe (see Figure 2.5), and connect the opposite pipeline installed previously. A cofferdam can be constructed by using different materials (i.e. steel sheet piles, sand bags). The suited materials for a cofferdam are influenced by diverse factors, such as depth of water, available space, duration of works, bed conditions, accessibility, and potential ingress of water. Besides these factors, it is also critical to consider the environmental impact. The height of a cofferdam should be considered for potential fluctuations in water levels. Before removing the cofferdam, the work area must be re-watered to avoid sudden ingress of water. When river is deeper or flow is faster, piled cofferdams are mostly utilized for an isolation material. Also, besides the size of river, this isolation method with cofferdams is generally chosen unless trenchless technologies are effective (SEPA 2009).

2.2.2 Horizontal Directional Drilling (HDD)

In the early 1970s, for the river crossing a HDD installation method was utilized by an innovative road boring contractor who completed the crossing works of a 183 m using a modified rod pushing tool with no steering capability (DCCA 1994). Since then, a HDD installation method is one of the most reliable techniques to choose due to minimum disruption of subsurface, shorter construction duration, and small construction footprints at river crossing pipeline projects. For these particular reasons, environmental protection regulatory agencies and environmental non-government organizations lead practitioners to select the alternative method of river crossings, which could minimize the environmental damage. In this situation, HDD installations are on the rise to be a suitable technique for river crossings.

HDD installations have three steps for crossing projects. The first one is pilot bore, the second, reaming, and the last, pipe pullback. Below is the figure of pilot bore and tracking procedure, which is the first phase of crossing technique (Figure.2.6).

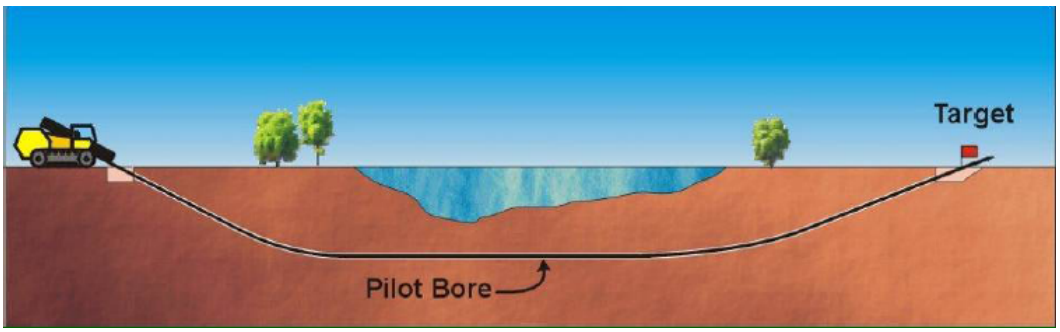

Figure.2.6 Pilot Bore in HDD (Lueke 2005)

At an entry angle between 8° and 20°, the location of a small diameter drill head is launched to the horizontal (Ariaratnam and Beljan 2005). Drilling advances until the surface of a preplanned exit location is found. In general, a transmitter (sonde), which gives the information of a pilot tube location under the river, is installed with the drilling bit. Alternatively, there are two tracking systems such as a wireline or wireless non- walkover system. Through obtaining an electromagnetic signal field, the depth, pitch, roll, and rotation position of the drilling head are shown by the tracking system. The objective of this tracking system is to find the actual location of the drill head during the bore progresses.

There are a few limitations for this tracking process, because it might be disturbed by buried utilities, steel structures, and several lines.

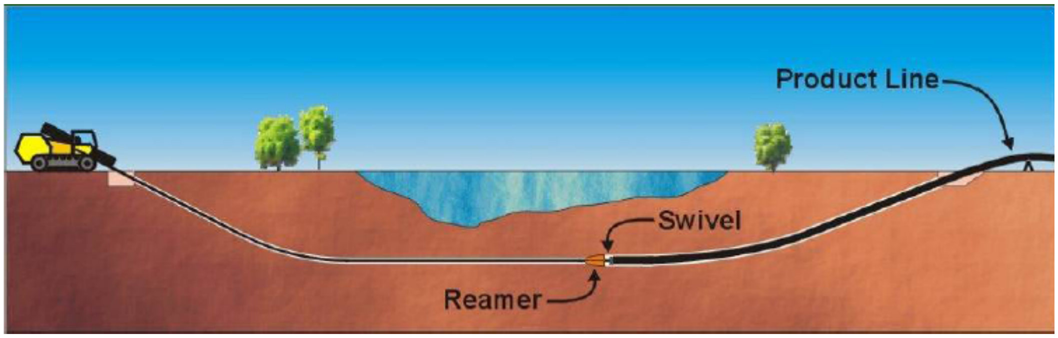

Figure.2.7 Reaming Work in HDD (Lueke 2005)

The second step is reaming (see Figure.2.7). The reamer is installed after the pilot bore has arrived in the exit hole. Reaming is a very important step, because the hole should be expanded before the pipe pullback process places the new pipeline inside the bore. The size of reaming is generally 1.5 times the diameter of the product line, even though this will be changed depending on the site conditions and the whole length of the installation (Bennett and Ariaratnam 2008). This reaming step helps reduce the frictional effects in the process of pullback step and moreover, reduces the bending moments. To earn the desirable upsizing prior to the pullback course, a prereaming step is required, especially for the large diameter sizes of pipes. The number of reaming is dependent on the soil properties, pipe diameter sizes, and drilling preferences. In particular, if the pilot bore encounters hard soils, or rocks, then the additional reaming processes are required owing to torque limitations and cleaning plant capacity. For the pipe pullback process, the product pipe is connected with the reamer. Also, a swivel is installed with the product line to allow only a reamer to drill and rotate. This swivel system helps to protect the product pipes from over- torsional stress due to the pullback process.

During all steps of HDD installation, in order to reduce the frictional coefficient between the product line and surrounding soils, drilling fluids, mixed with bentonite and water or added selective polymers or other agents, are installed. Stabilizing the borehole, removing the cuttings, reducing the torque on the drill string, lubricating the drill pipe, and cooling the drill bit could be a good description of the roles of drilling fluid installed particularly during the pullback step (Bennett and Ariaratnam 2008). Through the reamer orifices, drilling fluid is installed during the reaming and pullback phase, transporting the cuttings, and preventing the enlarged borehole from collapsing. Also, drilling fluid provides the lubrication that reduces the frictional effects between the product pipeline and the borehole wall. Furthermore, this existence of drilling fluid during the pullback step reduces the probability of the product line stuck under the water body.

2.3 Soils Typical of River Crossings

In order to decide the construction methods, one of the important factors that must be considered is the soil properties of riverbeds. It is necessary for a researcher to exactly understand soils typical of river crossings for analyzing the results of buried pipeline behavior. Thus, in order to learn soils typical of river crossings, the origin of riverbed soil must be confirmed above all. The soil properties of a riverbed could be different in terms of topography or geology. The types of river are a critical element to find the information of typical soil properties as well. Rivers can generally be divided into either alluvial, bedrock or a mixture of the two (Julien 2002). The classification of river bed soil is dependent on the topography and geology of the riverbed. Unconsolidated or weakly consolidated sediments are the major components in alluvial rivers. Moving sediments or particles composes riverbeds or banks, which are called alluvial rivers. Alluvial rivers erode their banks and deposit material on their floodplains. The channels of alluvial rivers are formed by themselves through experiencing the magnitude and frequency of the floods. This represents the ability of erosion, deposit, and the transportation of sediments. Alluvium is loose, unconsolidated soil or sediments, eroded, deposited, and reshaped by water in some form of a non-marine setting. Alluvium is typically made up of a variety of materials, including fine particles of silt and clay and larger particles of sand and gravel. When this loose alluvial material is deposited or cemented into a lithological unit, or lithified, it is called an alluvial deposit. The bed material in an alluvial river is relatively coarse at the headwaters of the streams whose slopes are steep. Contrary to the headwaters, the material size is relatively smaller at the downstream. Bed materials vary from boulders and cobbles to silts and clays.

Bedrock rivers are found in upland and mountainous regions. Their formation is made by cuts into the bedrock with the abrasion that sediments in the flow produce through collision with the channel bed. Bedrock rivers frequently include alluvium on their beds that contribute to the eroding and carving of the channel.

The research reflects the environment of actual pipeline installation sites that typically have alluvial types of riverbeds. Koloski et al (1989) defined alluvial as sediment deposited by streams. They made the report related to the geologic characteristics and origin of earth materials commonly found in the state of Washington to certain geotechnical properties. This specific data could be useful to recognize the general range of values for typical geotechnical properties, but is not representative of all riverbed features. Based on the report of Koloski et al (1989), alluvial was divided into two types: high and low energy alluvial. Alluvial with high energy generally means coarse sediment such as coarse sand, gravel, cobbles and boulders that have been deposited by fast moving water. This is often found in the headwater stream. Alluvial with low energy is fine-grained soil, such as fine sand and silt deposited by slow moving water. This can be found in middle or downstream. The soil properties of river crossings can be different regarding the location of pipelines buried. The research focused on the alluvial with low energy, which is the experimental soil utilized in this laboratory test. Based on the unified soil classification system (USCS) from the American Society for Testing and Materials (1985), alluvial river with low energy is specified in ML (silt), SM (silty sand), SP (poorly graded sand), and SW (well graded sand). These USCSs approximately describe the status of typical riverbed soils.

2.4 Pipe Buoyancy Theory

This section focuses on describing the original buoyancy theory that was created by Archimedes. Using this original buoyancy theory, the buoyancy theory for buried pipelines was created. Two buoyancy theories created for OT and HDD methods could be a good description for discovery of the buoyant factors affecting the stability of buried pipelines through the buoyancy theory.

2.4.1 Buoyancy Theory: Archimedes' Principle

In 212 B.C., Archimedes, the Greek scientist, discovered an object immersed in a fluid is buoyed up by a force equal to the weight of the fluid displaced by the object. This became known as the Archimedes' principle. The Archimedes' principle applies to objects of all densities. If the density of the object is greater than that of the fluid, the object will sink. If the density of the object is equal to that of the fluid, the object will neither sink nor float. If the density of the object is less than that of the fluid, the object will float. When a body is totally or partially submerged in a fluid, a resultant force acts on the body pushing the body upward. This force is called the buoyant force. Its magnitude is given by the weight of the fluid displaced by the body. That is

$$F_B = \rho g V \tag{1}$$

, where ρ is the density of the fluid, g is the acceleration of gravity, and V is the displaced volume.

Similar with the description of object density for buoyancy theory, buoyancy can be explained as the relation of the center of gravity. The direction of this buoyant force is upward and it passes through the center of gravity (CG) of the displaced volume (not the CG of the object itself). This point is called the center of buoyancy (CB). For a partially submerged or floating body, the weight displaced by the fluid above the liquid surface (usually air) is relatively small compared to the weight displaced by the liquid, hence it can be neglected. A completely submerged body is said to be in stable equilibrium as long as its CG (not the CG of the displaced volume) is directly below the CB. In the event when CG is coincident with CB, the body is said to be in neutral equilibrium. If the CG is above the CB then the body may be unstable, and the problem requires further analysis. Stability issues are of great concern in the design of ships (a partially submerged body). Consider a ship in an equilibrium condition such that CG is directly above CB. If the ship is inclined, the location of CB is shifted due to the change in the displaced volume; if the ship is located such that CB is at the left of CG, the buoyant force and the weight will form a couple. The tendency of this couple is to restore the ship to its original equilibrium position, hence the ship is said to be in stable equilibrium. If the ship is tipped such that CB is at the right of CG, the produced couple tends to capsize the ship and the ship is now in an unstable equilibrium position. This is the original principle of buoyancy theory when the object is partially or fully submerged.

In conclusion, buoyancy is the phenomenon that an object less dense than a fluid will float in the fluid. More generally, Archimedes' principle states that a fluid will exert an upward force on an object immersed in it equal to the weight of the fluid displaced by the object. This principle will be applied to create a theory of buried pipeline submerged in saturated soils.

2.4.2 Design Methods of Pipe Floatation in Post-Construction

The buoyancy theory mentioned above can be applied to the shallow cover floatation effects for pipe installations in submerged soils (PPI 2006). A high density polyethylene (HDPE) pipe has higher probability for buoyancy when installing the pipe material in areas having a high water table or when trench flooding is likely to occur. In the status of water crossings, a water table is always higher than the location of buried pipelines. Basically, this suggested buoyancy theory in PPI (2006) could be applied when the water table is located in a higher position than the buried pipeline regardless of topography. PPI (2006) specified that pipe floatation in submerged soils occurs when the ground water surrounding the pipes produces a buoyant force (W_w) greater than the sum of the downward forces provided by the soil weight (W_s), the weight of the pipe (W_p), and the weight of its contents (W_c) (PPI 2006).

$$W_s + W_p + (W_c) < W_w \tag{2}$$

Not all factors (W_s, W_p, W_c, *and* W_w) presented by PPI (2006) were considered in Eq. (2). In our research, the weight of a pipe's contents (W_c) were not considered, because the research for the pipe floatation only was assumed when an installed pipeline is not being operated. Thus, W_c is excluded in the theory and experiment. The buoyancy theory will be utilized for producing the results of the theoretical method that will be compared to those of the experiment.

Figure 2.8 presents how the experimental test for pipe floatation simulates the traditional open trench installation pipeline.

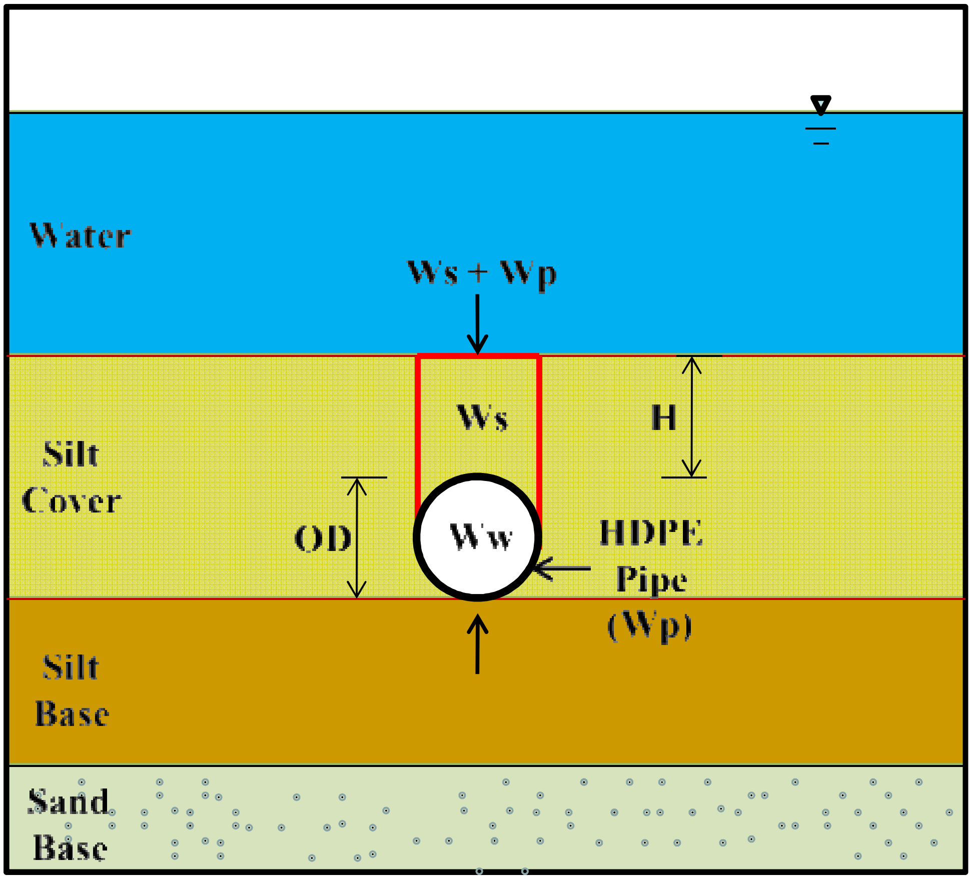

Figure 2.8 Pipe Floatation Theory in Traditional Open Trench

If the weight of water replaced by pipe (W_w) is larger than the weight of the pipe plus soil overburden, a buried pipe floats to the surface. This means the major factor of pipe floatation could be the weight of soil overburden and the weight of the pipe. The weight of a pipe is dependent upon material, diameter, and standard dimension ratio (SDR). Thus, the critical factor the engineer must consider for buoyancy is how deep the buried pipe must be installed regarding specific geotechnical situation. Denser soils increase the uplift resistance force regardless of soil types (Cheuk et al 2008). Soil density is involved in the calculation of W_s that is an important part for producing the uplift resistance force.

Thus, a key point of pipe floatation for traditional open trench will be to manage the depth of cover over a buried pipe. Table 2.1 shows the weight of the pipes (W_p) used in both the experiment and the theoretical calculations.

Table 2.1 Weight of High Density Polyethylene (HDPE) Pipe (PE3608)

Pipe Size (mm)	OD (mm)	W_p (kg/m)
50 (SDR17)	60.3	**0.64**
75 (SDR17)	88.9	**1.38**
100 (SDR21)	114.3	**1.87**

The properties such as weight and diameter were provided by The Plastic Pipe Institute (PPI). For this particular research, three diameters (50, 75, and 100 mm) were selected to be used in the experiment. The Standard Dimension Ratio (SDR) for the HDPE pipes used was as follows: SDR17 for the 50 mm pipe, SDR17 for the 75 mm pipe, and SDR 21 for the 100 mm pipe. These SDRs were chosen because they are often utilized in real pipe installations. Additionally, these pipes also have high probability to float to the surface, so this can help to present a significant difference between theoretical and experimental results.

Table 2.2 Weight of Water Replaced by a Pipe (the buoyant force) in OT

Pipe Size (mm)	OD (mm)	γ_w (ton/m^3)	W_w (kg/m)
50	60.3		**2.87**
75	88.9	1.0	**6.22**
100	114.3		**10.27**

The weight of water (W_w) replaced by a pipe in traditional open trench installations is obtained by multiplying the replaced volume by a buried pipe and unit weight of water (γ_w: 1.0 ton/m^3) together. Table.2.2 shows the weight of water displace by each pipe used in the experiment for OT. The amount of this value is the buoyant force that causes pipe floatation. To impede the buoyant force, another factor, the weight of soil overburden (W_s), was calculated. The results of the weight of soil overburden vary depending on various depth-to-diameter (H/D) ratios. First, the equation of the weight of soil overburden (W_s) is presented by

$$W_s\ (lb/ft) = (\gamma_s - \gamma_w) \times [(4-\pi)/8 \times OD + H_s] \times OD \qquad (3)$$

where γ_s is unit weight of saturated soils, γ_w is unit weight of water, OD is outer diameter of the HDPE pipe, and H_s is the depth of cover over a pipe. The weight of soil overburden is proportional to H/D ratios that could be controlled to find the critical depth of cover for each diameter. H_s is the only variable for this theoretical method. Depending on varying H_s, the weight of saturated soils over a buried pipeline is changed. As H_s increases, the total value W_s also increases. If finding the value of H_s that stops pipe floatation, the calculation will be stopped, and this obtained value could be compared to the critical depth of cover from the experiment. When considering soil overburden, as the density of the soil increases the uplift resistance force increases regardless of the soil type (Cheuk et al 2008). Soil density is integral to the calculation of W_s. The weight of soil overburden (W_s) is calculated as the volume of soil cover over a buried pipe using a prism model.

The buoyant force (W_w) is determined by multiplying the volume of outer diameter by unit weight of water. This buoyancy model assumed that a limit equilibrium solution known as the vertical slip model applies to the system as described by Schaminée et al (1990).

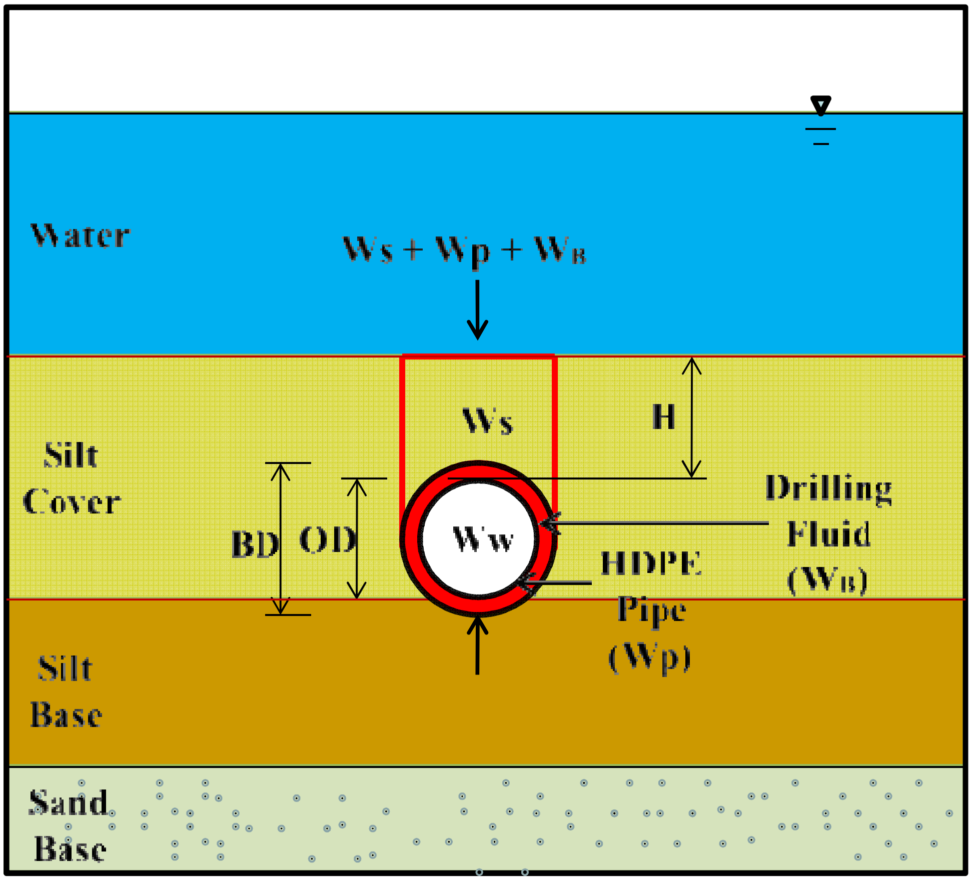

Figure 2.9 Pipe Floatation Theory in Horizontal Directional Drilling

In directional drilling installation, the failure theory of the pipe floatation was applied in Figure 2.9. Unlike Figure 2.8, there is an additive factor, the weight of drilling fluids in the annular space, which is the doughnut shape in red (see Figure 2.9). Drilling fluid in the annular space could be considered in two influential factors that are utilized in pipe floatation theory. The first factor is to increase the buoyant force due to a larger volume replaced by the borehole. On the contrary, the density of drilling fluid used or designed in experiment or reality is greater than that of water and of saturated surrounding soil. Accordingly, the theoretical calculation in directional drilling considers this borehole as the part of uplift resistance force (W_B). Also, the larger buoyant force, which is the first factor suggested, is considered in the calculation of W_w in Eq. (4).

$$W_s + W_p + W_B + (W_c) < W_w \tag{4}$$

Normally, filling the product pipe with water (increase the W_c) is considered good practice to increase the effective weight of the pipe during the pullback step when utilizing HDD on larger pipe installations. This decreases the pullback force required to install the pipe as the increase in effective weight of the pipe decreases the buoyancy of the pipe and the normal force the pipe exerts on the crown of the borehole. It is very important that the amount of water in the pipe during pullback should be controlled on larger installations because pullback forces are not inadvertently increased as a result of increased pipe drag in the borehole or on the surface. For the diameter of the pipes utilized in this experiment ballast is generally not added during pullback, and as the goal of the research was to compare the buoyancy of pipes installed by OT and HDD, the increased buoyancy assisted in conducting the experimental observations. For the situation where the HDD installation is completed sometime in the past, the annular space will form a semi-cohesive material with low permeability. This annular space then may act as a much larger pipe displacing more water than the pipe installed utilizing OT methods. Additionally, the annular space would add an additional uplift resistive force (W_B) from the weight of drilling fluid and cuttings. This approximation should be valid considering that bentonite clay is a constituent of most drilling fluids, it forms a filter cake to prevent water migration, provides some cohesive properties, and like most clays has a low permeability. Table 2.3 below is the buoyant force in HDD method.

Table 2.3 Buoyant Force in HDD

Pipe Size (mm)	OD (mm)	BD (mm)	γ_w (ton/m^3)	W_w (kg/m)
50	60.3	101.5	1.0	**8.09**
75	88.9	127.1		**12.69**
100	114.3	177.7		**24.79**

In order to compare the behavior of a buried pipeline installed by traditional open trench and HDD, the critical depths of cover, H/D ratios, are computed by the theoretical and experimental method. The meaning of H/D ratios is to minimize the value of depth to diameter ratio that can make a buried pipe safe. If a critical H/D ratio is small, the pipeline does not need large depth of cover for pipe security. It means that this pipeline installed with small soil cover has enough uplift resistance force against the pipe floatation.

2.5 Previous Research of Pipe Floatation

2.5.1 Shear Strength of Saturated Soils

Soil is a useful building material, because it has the shear strength to be able to support itself and other loadings. Soils resist the compressive stress using shear strength at contacting points between particles. In other words, this shear strength of soil is the internal resistant force to resist the failure (Das 2006). If the maximum shear resistance of the particular soil is lower than shear stress acting on soil ground, the original arrangement of soil particles vary, and this leads to shear failure of soil particles. Thus, when installing the pipeline under water, the understanding of shear resistance for native soil could be necessary, because shear failure of surrounding soils around the pipeline induces the failure of pipeline security. Geotechnical engineers must be able to predict the loading on a soil, its strength, and determine whether it will be safe or how to modify it to make it safe. A factor of safety against the shearing failure exists if the applied stress is less than the shear strength. To make a failure plane of a soil mass, the functional relationship between normal stress and shear stress is expressed. The failure envelope is basically a curve, but approximately regarded as a linear function (Das 2006). This function is shown as,

$$\tau = c + \sigma \tan\phi \qquad (5)$$

where τ is shear stress, c is cohesion, and ϕ is angle of internal friction. c and ϕ are the value of inherent soil properties to represent the soil strength that could resist the failure of soil particles. These values are obtained through the direct shear test. In order to research the behavior of buried pipelines at river crossings, the first thing understand is the interaction between saturated soils and buried infrastructure systems. The shear strength of soil could be the important factor for studying the relation of saturated soils and buried pipelines. If external force (i.e.

natural disaster; earthquake, flooding, and force induced by construction process; pull back force in HDD) exerts native soils, the soil mass becomes stressed. All soils have intrinsic shear strength to resist deformation by external force. However, if this external stress is over the limit of the soil's shear strength, then the soil will begin to lose its resistance to keep its cohesion. Soil failure causes an unstable ground situation, which can bring about various pipeline incidents. Total normal stress in saturated soils, riverbed status, is the sum of the effective stress (σ') and the pore water pressures (u).

$$\sigma = \sigma' + u \tag{6}$$

This effective stress is derived from the pure soil solids. If the external stress occurs in the ground, the pore water pressure (u) increases, because water does not flow out quickly in saturated status, which causes the excess pore water pressure in the soil. The increased pore water pressure ($u + \Delta u$) is exactly the same amount as additional stress ($\Delta\sigma$) from the external force, because effective soil stress (σ') is not changed (Das 2006). The shear failure of soil occurs if the yielding stress at contacting points between soil grains is over the shear failure envelop due to excess pore water pressure. Once consolidation starts, the pore water pressure decreases as the soil solid stress increases because the additional total normal stress ($\sigma + \Delta\sigma$) is unchanged. The structure of soil particles starts to vary in post-drainage due to an increase of effective soil stress. The stiffness of soil decreases, and soil structure cannot maintain a stabilized status. Hence, buried pipelines in this damaged soil structure are also unsafe. Thus, this shear stress theory including the interaction between pore water pressure and effective soil stress must be considered for the behavior of buried pipelines in saturated soils.

2.5.2 Unstable Buried Pipeline by External Force

Soil liquefaction is a major reason for pipe floatation in saturated soils as well. Kitaura and Miyajima (1985) studied the relation between buoyant force and excess pore water pressure caused by an increase of the external force. They described the development of the buoyant force in association with total soil stress and external force variation. Water attempted to flow out from the soil to a zone of low pressure during the external force; however, the undrained status did not allow water to move out smoothly, which causes excess pore water pressure. This excess pore water pressure increased the total normal stress in soil. After all, this situation brought about the fluidal movement around a buried pipe, which diminishes the uplift resistance force. In this research, they found that the upward force (buoyant force) does not act on the buried pipe as soon as the excess pore pressure increases. The buoyant force started to act when the excess pore water pressure built up to a certain amount.

Siddharthan and Norris (1993) also studied several factors related to pipe floatation mechanism during severe storm. Heavy storm produced wave that could make residual pore water pressure. This increasing pore water pressure can largely diminish the uplift resistance force. This research stated that an increase of residual pore water pressure by wave affects an increase of the positive buoyancy, and reduces the effective mass of the deposit owing to the upward seepage. Teh et al (2003) studied the stability of a marine pipeline in moving and liquefied soil beds through an experiment, which simulated a buried pipeline in wave flume. In liquefied condition by wave, it was found that floating or sinking depth for a buried pipeline is mostly related to the pipeline specific gravity and liquefied soil parameters (Damgaard et al 2006).

Overall, it is found that the behavior of buried pipelines in saturated soil could be linked in variation to an external force such as an earthquake vibration or a strong wave. The fluidal movement by the external force causes the decrease of uplift resistance force. In a saturated situation, providing excess pore water pressure leads to the shear failure of soil as well. Furthermore, it is evident that the shear failure of soil adjacent to structure could determine the status of buried structure.

2.5.3 Soil Arching Effect

Another factor to clarify is “soil arching effect.” Anson Marston insisted that the load on the pipe conduit does not fully exert on the pipe due to the arching effect that distributes the load to the adjoining soils around the pipe (Marston 1930). This is called “Soil Arching Effect,” which defines the action of transferring forces between native soils and stationary structure in detail. Since this has been studied for a century, Arching Effect has been recognized as a critical theory when underground infrastructure is designed and constructed. The redistribution of soil stress during or post construction of underground infrastructure occurs by the difference of the stiffness between soil and the infrastructure system (Tien 1996). Also, this theory could explain the interaction of saturated native soil and artificial structure installed underneath the river.

Hence, the critical connection between soil loadings and buried pipeline failures (i.e. settlement, deflection, break, and buoyancy) could help reveal the behavior of buried pipelines by learning the soil arching effect. When installing a man-made structure in native soil, the shear resistance of the original soil controls and distributes the amount of vertical soil pressure around the structure. This causes the stress to move on soils adjacent to the underground structure (Terzaghi 1943). There are two types of Soil Arching Effect: 1) positive arching, 2) negative arching. The advent of different types of the Soil Arching Effect is relevant to the stiffness of structures and the status of soil compactions in the ground (Tien 1996).

First, different arching effects are produced by differences in the stiffness or the compressibility of structures as mentioned previously. The former (i.e. positive arching) is when the structure installed is relatively flexible compared to the surrounding soils. This flexible pipe is more compressible than the surrounding soils. Stress inclines to move at the more compressible mass, so this pipe, a more compressible one, is quickly compressed compared to another mass technically. However, the actual stress exerted on the flexible pipe is lower than theoretical values, because the excess stress above the flexible pipe is diminished by the positive arching effect. The latter one (negative arching) is the opposite situation against the former one. This negative arching normally happens to a rigid pipe, because adjoining soils are more compressible than a rigid pipe. Hence, soil pressure is transferred to the center above the buried pipe through shearing in order to balance the stress acting on the plane.

The second factor affecting the stress redistribution is the status of compaction (i.e. backfill and bedding). Assuming the bedding is well-compacted, insufficiently compacted backfill over the pipe could have high vertical stress relatively because the backfill is more compressible than native soil. To reduce the vertical stress for unconsolidated backfill, positive arching acts on the backfill so that the excess vertical stress on the pipe could be transferred to side soil. Contrary to this situation, negative arching acts on well-compacted backfill over the pipe, meaning it will cover more stress for surrounding soils. This redistribution of stress happens because the artificial structure and soil medium respectively have different stiffness and properties for supporting the original load together. To sum up, this arching effect is contingent on the stiffness of a mass (i.e. soil and structure). In reality, engineers and designers consider this arching effect for the deformation of pipe or pipeline settlement when designing the underground structure (Tien 1996).

For traditional OT, the excavation of surface soil under the river leads native soils to lose their inherent stiffness. So, when burying the pipeline and after backfilling, it is expected that the stress on the buried pipeline will increase due to the extra stress moving from loose backfilling (Negative Arching). This occurs because the stiffness of the buried pipeline could be larger than the loose soil that lost its intrinsic stiffness. On the contrary, the positive arching effect mostly occurs in the case of trenchless construction, because the surrounding soils adjacent to the pipeline installed by trenchless technology are native, which keeps the intrinsic stiffness of soil intact (Najafi 2010). Upward shearing stresses along the sides are induced by the relative high vertical soil pressure converging on the pipe, and this action produces positive arching in order to decrease the load of vertical stress above the pipe. The adjacent soil might have larger stiffness than the buried pipeline has. Thus, the partial loading stress concentrated on the buried pipeline moves in the surrounding native soils for stress equilibrium. This soil arching effect informs the soil itself to control the loading stress adjacent to the underground structure in order to prevent soil failure. Furthermore, these understandings of saturated soil interaction within the infrastructure system could help to analyze the final results of the pipe buoyancy research.

In summary, learning the soil behavior is important, because the behavior of a buried pipeline is contingent on the interaction of soil status. Conclusively, this soil arching effect informs that soil itself tries to control the loaded stress adjacent to the underground structure in order to avoid the soil failure. As mentioned above, the relatively resistant force and shearing strength of soil against the excess load could be a considerable relationship influencing the behavior of buried pipelines at river crossings. The next is to explain the uplift resistance force, the factor preventing actual pipeline floatation incidents.

2.5.4 Uplift Resistance Force

In previous research of pipe floatation, most of the studies focused on the upheaval buckling that occurs when gas or oil with thermal heat is transported through the long pipeline installed under the ocean. This pipeline often carries gas or oil with higher heat and pressure much greater than the surrounding sea water. This thermal discrepancy induces a large compressive force as a result of the pipe and seabed friction interactions, and then this compressive force makes the pipeline expand. This expansion leads to the longitudinal buckling that makes the pipeline move upward (Ng and Springman 1994). In real pipeline installations, upheaval buckling occurred in the North Sea due to an inaccurate safety margin in the depth of cover (Bransby et al 2001; Nielson and Lyngberg 1990). Insufficient depth of cover or design error happened because the upheaval buckling has not been accurately considered.

To understand this upheaval buckling in detail, it is very critical to learn the uplift resistance force which prevents the pipe floatation. Cheuk et al (2008) found that the uplift resistance has to be mobilized to avoid the beginning of buckling phenomenon at an adequately small displacement under a buried pipe. Thus, understanding the uplift resistance and the mobilization distance (the pipe movement before the maximum uplift resistance force is reached) is critical when designing offshore pipe installations (Palmer et al 2003).

Centrifuge tests were the method used in the calculation of uplift resistance forces. Centrifuge tests have been extensively utilized in the study of soil structure interaction (Wang et al 2009). The principle behind a centrifuge test is that the soil behavior in a small scale model can be created to be identical to that of a full scale prototype if the stress condition in a small scale model is the same as those of the full scale prototype. Thus, a centrifuge test is a convenient way to determine the value of uplift resistance force when a full scale prototype cannot be established in a laboratory.

Trautmann et al (1985) analyzed the relationship between uplift resistance force, cover depth of pipe installation and soil displacements. The design of a buried pipeline in areas of vertical ground movement was partly dependent on the magnitude of the forces on the pipe and the soil displacements at which they are developed. The maximum uplift resistance force was produced within the specific range of soil displacements. The maximum uplift resistance force occurred when the displacement of soil materials ranged from 0.005H to 0.015H generally (H: depth of installation, dimensionless).

Schupp et al (2006) focused on the relationship between depth of cover, uplift rate, pipeline diameter, and pullout resistance in drained and undrained status simulating the buckling in a model pipeline under laboratory conditions and observed the relationship between soils and pipes. In the test of loose sand, the research confirmed that the displacements around a buried pipe are inversely proportional to uplift resistance force at each different diameter size.

The relationship between uplift resistance force and soil displacement has been studied by showing the real behavior of a pipe, which revealed that soil density influences the uplift resistance force regardless of soil types (Cheuk et al 2008). Cheuk et al (2008) concluded that the inclination of the shear zone that affects soil friction is contingent on the soil density, with denser soil being more dilatant. The magnitude of the peak uplift resistance is unaffected by particle size for the chosen cover depth-to-diameter ratio (H/D). However, the width of the shear zones is strongly dependent on grain size. As a result, soil friction affected by shear zone tendency could have a connection to uplift resistance regardless of grain size.

El-Gharbawy (2006) utilized the scale-model tests of a buried pipeline in loose silty sand in order to examine two traditional uplift resistance force equations (i.e. Schaminee and Pedersen) that have been generally utilized in offshore pipeline. The results of the uplift behavior in the research showed that these traditional Schaminee or Pedersen uplift models may not be a suitable for forecasting uplift resistance of pipeline, especially in low backfill densities because it found that the traditional equations for predicting uplift resistance produced uplift force higher than actual uplift force.

Cathie et al (1996) discovered very high void ratio in backfill caused pipe uplift. Subsequently, White et al (2001) proved that the relative density of soils (mostly sand) has a strong influence on the uplift resistance force. If backfill density is increased, uplift resistance will be increased. This result is the same effect as burial depth is increased. They explained actual kinematic mechanism of pipe uplift, and investigated the connected resistance. They used mini-drum centrifuge (a 0.8m diameter) for an uplift test. However, this design method needs the angle of dilation as an input parameter.

Mohri et al (2001) discussed the behavior of buried pipelines and adjacent soils about pipe uplift by using a distinct element method (DEM). DEM is the good method to describe a large deformation for both ground and underground structures with small-scale model tests. They explained that the development of a shear band and the flow of soil particles around a buried pipe determined uplift resistance forces.

Bransby et al (2002) presented the capacity of uplift force in post-installation when “Jetting” method is utilized for open trench excavation in offshore pipeline projects. Jetting is the excavation technique for an offshore pipeline: a trencher is driven over the seabed, excavate and penetrate it. The trencher with 2 jet legs pumps out water, which demolishes the structure of clay. During jetting, the seabed soil loses its intrinsic strength and is liquefied completely. The research used centrifuge model tests for simulating the status of a buried pipeline on completion of jetting and studying the uplift force and the load-displacement behavior of buried pipelines in undrained status and drained status. The research concluded that uplift force in undrained status was lower than in drained one.

In submerged condition, Endley et al (2009) demonstrated how various liquid limits (LLs) in the particular silts or mud could affect pipe floatation. They simulated pipe floatation tests in terms of various bulk densities of three different soil types, changing liquid limit (LL) by adding water through the field tests. The range in H/D ratios utilized in this test was from 1.5 to 2.0. As a result, the uplift resistance force decreased exponentially as water contents increased. Polyvinyl chloride (PVC) pipe buried in very soft soils did not float at moisture contents about 1.2 to 1.4 times the original LL regardless of soil types. However, it did float at about 2.3 times the original LL that produced the maximum floatation force in very submerged soft soils.

In shallow installations, Wang et al (2010) insisted that shear contribution can be applicable for uplift resistance force even if the H/D ratio designed is less than 1.0. The current industry tended to ignore shear strength effect when the H/D ratio was designed less than 1. They implemented a soil failure mechanism, vertical slip surface model, in loose, dense sand and gravel, and proved that the shear strength in the shallow cover depth (less than 1.0D) contributes to uplift resistance force.

In summary, the uplift resistance force has been researched at diverse directions, such as depth of cover, soil properties (density and grain sizes), and shear yielding zones. In other words, the studies of pipe floatation phenomenon have mostly been conducted by observing pipe behavior regarding the uplift resistance force and soil displacement, which means almost all investigation concerned with pipe floatation has been oriented in the geotechnical viewpoint. Also, the trend in pipe floatation research was to present the influential factors that trigger pipe floatation and the boundary condition affected by the uplift resistance force. The next section describes the influence of drilling fluid in HDD installation that may cause different results for pipe behavior between traditional open trench and HDD method.

2.5.5 Annular Space

The annular space filled with drilling fluid in directional drilling installations is a different feature from traditional open trench (OT), and this is expected to cause the varying behavior of a buried pipe in the experiment. The definition of the annular space is the space between the outer diameter of a buried pipe and the borehole wall. In the middle of directional drilling installation, drilling fluids in the annular space play various roles in the drilling operation for protecting buried pipes and the borehole. Cleaning the cutting bit, lubrication of the pipe, transporting the cuttings, and stabilizing of the borehole are the purposes of using drilling fluids (Knight et al 2001). Mostly, bentonite clay mixed with water is the major composition of drilling fluid. If needed, small amounts of polymer are added to increase the yield. Bentonite mixed with water formulates a low permeability zone around the edge of the bore, called "the filter cake." The filter cake exerts a positive net hydrostatic pressure against the bore wall, preventing native soils from entering into the borehole. If no filter cake is formed, the effective pressure on soil particles will quickly decrease to zero, even at a low drilling fluid pressure owing to the rapid drainage of the drilling fluids into the loose soils (Wang and Sterling 2007). The ideal conditions for the filter cake are fast-formation, smoothness, and reduction of the movement between the drilling fluid in the borehole and native soil. To strengthen the filter cake, mixing bentonite with polymer helps the drilling fluid flow into the borehole because the required elements for the optimal drilling fluid are less viscose, more pumpable, and flowable to maintain the original shear strength of the fluid. Constructing the optimal drilling fluids in the annular space could be the major point for constructing a sturdy borehole. Deciding the optimal mixture of drilling fluid depends on the native soil around the planned borehole.

Hypothetically, low permeability made by filter cake inside the borehole may increase buoyant force due to the larger volume of the borehole replacing native soil. In other words, the displaced weight of water, the buoyant force, is larger since an outer diameter extends the size of the borehole (called "Borehole Diameter"). On the contrary, if the replaced drilling fluid has greater density than saturated silty soil, which is native soil, this larger density might increase the uplift resistance force. As a result, these two points mentioned will influence the result of pipe floatation tests in opposition.

The pressure of the annular space during or after installation must also be considered just in case various modifications related the annular space may potentially affect the behavior of a buried pipeline installed by HDD method. Kennedy et al (2006) investigated the initiation of tensile fracture using finite element analysis for simulating the sand material and the filtercake around the borehole. The research examined the soil response depending on the variation of mud pressures. They found that hydraulic fracture may be caused by mud loss. Mud loss was identified by two mechanism theories, such as flow of mud by tensile fractures in the ground and unconfined plastic flow of the adjacent soils induced by mud pressures during HDD installation.

Larry (2004) studied the engineering design of HDD installation for polyethylene pipe. He examined design considerations for HDD installation by checking several design equations. When considering the pressure class for HDD installation, the designer must confirm not only the requirements for pullback installation but also buried pipe behavior in post-installation. He also examined the pressure status for the annular space during pullback and post-installation. During pullback, frictional resistance to pullback installation is dependent on the net upward buoyant force that is proportional to the mud-cuttings mixture weight. In order to reduce the buoyant force, empty PE pipe is filled with water.

Baumert et al (2004) investigated installation loads and borehole pressure of 19 commercial HDD installations. The soil type in these installations was silty clay. They found the existing model for predicting pull loads was created in ideal borehole condition that is a perfectly stable annular space filled with low viscosity drilling mud. In this manner, the resistance of the annular space is caused by the net buoyancy effect and bore friction at a contact surface between the pipe and the borehole wall. However, they mentioned that this ideal situation is not same as real situation. They took account of actual condition of the annular space by considering a mud drag component related to viscosity of drill mud. Also, they explained the in some cases total fluid mud volume replaced by the pipe may be less than the volume of the pipe during pullback process. This will reduce the pipe buoyancy effect. It can be indicated that the reduction of the total mud volume could be an influential element for the pipe buoyancy in HDD installation.

Baumert et al (2005) stated that the depth of cover, one of the primary factors for design in HDD installation, has not been designed accurately due to following only conventional design. When the depth of cover and drilling equipment are determined, the amount of pulling load is one of the main elements that must be considered. However, pulling load has been calculated by a typically lower viscous shear of drilling fluid and the wrong skin friction coefficient in HDD, which is the conventional design style. In this manner, inaccurate depth of cover designed by pulling load could affect the behavior of a buried pipeline.

To sum up, the annular space in HDD installation may affect the behavior of a pipe associated with the buoyancy phenomenon. The first factor would be the existence of the filter cake that is formed around the annular space. This filter cake surrounds the annular space as a mass that has cohesion, compressive strength, and a low permeability due to hydration and consolidation. This effect may lead the increase of buoyant force. Additionally, several studies found that diverse states (i.e. hydro fractures, tensile fracture, and mud pressure) around the annular space may influence the stability of a pipeline during or after the installation. Thus, it is anticipated that the existence of the annular space in HDD leads the different behavior of a buried pipeline at river crossings between OT and HDD.

In summary, this chapter presented several factors influencing buried pipeline floatation in saturated soils through previous research. These potential factors are shear strength, arching effect, uplift resistance force, and annular space. If the maximum shear resistance of the particular soil is lower than shear stress acting on soil ground, the original arrangement of soil particles vary, and this leads to shear failure of soil particles. Shear failure of surrounding soils around the pipeline induces the failure of pipeline security. Arching effect described transferring total soil stress between materials in terms of difference of their stiffness. It provides how soil stress around a buried pipeline affects the behavior of pipelines. The equations and development of the uplift resistance force in previous research help to understand mechanical soil responses against pipeline movements. Finally, through previous research of annular space it is anticipated that its features affect the buoyancy effects of pipelines submerged in saturated silty soils. With prior research, the factors found in this chapter must thoroughly be confirmed with the research results in order to elucidate buried pipeline behavior at river crossings.

Chapter 3: EXPERIMENTAL DESIGN AND PROCEDURE

3.1 Introduction

This chapter describes the detail experimental design and procedure for pipe buoyancy tests. The exact simulation results require systematic preparation, methods, and procedure, which this chapter described. The objective of the experiment is to simulate actual pipeline crossing construction under rivers and determine the distinct behavior of a buried pipeline in terms of different construction methods. This chapter mentions the experimental factors of pipe floatation that may influence buried pipeline behavior in this experimental design. These experimental factors mentioned were acquired from the design process of a river crossing pipeline. This chapter also described the experimental soil, setup, and detail experimental procedure. The experimental design is separately described by two construction methods; traditional open trench (OT) and horizontal directional drilling (HDD). In HDD simulation, the detail description for simulating the annular space status was illustrated in this chapter as well.

3.2 Experimental Factors of Pipe Floatation

Several factors that must be observed and analyzed before completing an experiment were found. These factors were discovered by scrutinizing actual river crossing design and construction process. There are four experimental setup directions for a detail understanding of influential factors related to pipe floatation experiment. The first one is to determine typical soil properties of riverbed. Finding out similar soil properties of riverbed status leads more reliable experimental results and analysis. The reliable outcomes of the research could substantially help actual river crossing design or construction process to prevent pipe buoyancy. The second one is to organize a suitable experimental setup for this simulation. Imperfect experimental setup could interrupt producing accurate results for pipe floatation behavior. Hence, this apparatus must perfectly be fabricated by considering there is no such impact from a wrong experimental design. In order to fix a pipe assembly while pouring soil-water mixture, devising supporting steel frames is a good example to organize suitable and stable simulation setup. The third one is to simulate the experimental procedure associated with different construction methods (i.e. traditional OT and HDD). Once the experimental setup was completed, a systematic procedure for the experiment must be developed by considering actual pipeline construction process. While creating a practical procedure, limitation and drawbacks from pipe floatation experiment are confirmed. Those limitation and drawbacks are reduced at a minimum. Lastly, two different construction methods have distinct bedding status in post-installation. The research tried to simulate the real bedding situation as much as possible in order to produce reliable data. For instance, while the basic design principles for depth of cover are similar (minus consideration for borehole pressure analysis) between HDD and OT construction techniques, how the product pipe is situated in the soil medium is enormously different. For OT installations, the pipe is typically bedded in either the original excavated native material or buried beneath an engineered backfill. Alternatively, for HDD the pipe is installed in a drilled borehole incased in an annular space composed of a drilling fluid and native soil cuttings. With these considerations, the experimental procedures were made following actual construction process of watercourse crossings. These considerations for a pipe floatation experiment were broadly described based on how to organize the experiment. In these descriptions, more detail design factors can be classified for investigating the influence of pipe floatation results. These detail design factors are following:

1) Pipe: Material, Service, Size, Project Length
2) Soil Bed Properties
3) Depth of Cover
4) Borehole Size (for HDD method)
5) Flow Characteristics (scour, flood, and tide)

These factors are closely concerned when watercourse pipe crossings are designed and constructed in reality. Except for flow characteristics, every factor was managed in fabricating a proper experiment system. For an efficient and reliable experiment, many factors mentioned above must be considered. An organized scheme for the experiment will help acquiring accurate data set.

3.3 Experimental Design

To prepare a real scale experiment for pipe floatation, this section is to depict in detail how the experimental design was organized. The exact and organized experiment process is compulsory due to accurate results of laboratory tests. This experiment was designed as a factorial design considering testing soil, pipes, depth of cover, and annular space that were mentioned in previous section. Pipe diameters, installation methods, depths of cover, and annular space diameters are a dependent variable, and soil conditions are constant.

3.3.1 Factorial Designs

3.3.1.1 Soil

First of all, the most challenging work was finding an appropriate soil for this pipe floatation experiment. The incidents in which pipe floatation had occurred were located in the eastern United States for a crossing of the Mississippi River or one of its tributaries In consultation with the engineer involved in these crossings it was determined that the soil was silt with minimal if any clay, sand, or gravel. The soil had a low unit weight and small cohesive properties. To assist in finding a similar soil local to Arizona, the researchers employed a specialty soil consultant who eventually found a suitable soil in Tucson, AZ. This soil was pure silt, and the product of a wash plant that processed sand and gravel for aggregate. Once the soil was determined for the experiment, the laboratory tests of the soil were performed to obtain the critical soil properties that could affect pipe floatation. Also, these test results helped to determine whether or not the chosen soil sample could be suitable for simulating a river crossing. The specific gravity of the extracted soil sample was calculated. A direct shear test was also carried out to gain the internal angle of friction and the cohesion of the soil, which are shear strength parameters. Saturated unit weight was obtained by checking the value of graduated cylinder with known weight of test soils in same condition of an experiment. Table 3.1 shows each property for soil samples obtained by the laboratory test.

Table 3.1 Various Parameters of Test Soil Samples

Saturated Unit Weight (ton/m^3)	Specific Gravity (G.S)	Soil Moisture Contents (%)	Friction Angle (°)	Cohesion (kpa)
1.197	2.763	77	23.1	10.9

The saturated unit weight in the test soil sample was obtained after soil-water mixture settled down for 24 hours. Table 3.1 showed the saturated unit weight and specific gravity for the soils used in the experiment. On analysis it was found that this soil had a specific gravity of 2.763, a saturated unit weight of 1.197 ton/m^3, a 23.1 degree internal angle of friction, and had cohesion of 10.9 kpa. The specific gravity (G.S) of the soil sample was 2.763, placing it within the range of clayey or silty soils, which vary from 2.6 to 2.9 (Das 2006). In terms of PPI (2006), the saturated unit weight of silts and clay is from 1.394 to 2.098 (ton/m^3). This saturated unit weight in the soil sample is smaller than the given range of soil classification in PPI (2006). Subsequently, the saturated unit weight of 1.197 ton/m^3 used in the experiment can be regarded as very loose silty soil.

In Das (2006), the internal angle of friction for silts has the range of 26 to 35 degrees. Through the direct shear test, the internal angle of friction of the soil was determined to be 23.1 degrees. This value for soil sample seems to be low compared to one of silts in Das (2006). Shear strength parameters of test soil sample are shown in Table.3.1 above. Cohesion could be also compared to typical value of general soils that have been utilized as the real application in geotechnical research. Test soil sample had 10.9 kpa for cohesion, which is approximate value of typical cohesive strength for SC (clayey sand) or CH (clay of high plasticity) based on Lindeburg (2003). With these results, the soil samples could be deemed as loose clayey silts mixed with some sands. These values are indicative of high plasticity clays and silts, and determined to be suitable to simulate a typical river deposit.

To summarize, the laboratory test revealed that the properties of the tested soil samples could be classified as clayey silts or loose silts with sand. As mentioned in Chapter 2, almost of the soil parts in typical riverbed generally are composed of sandy silts or silty sands with different composition ratios between silt and sand depending on the topological features. The soil had a low unit weight and minimal cohesive properties. These values are indicating this soil could be determined as high plasticity clays and silts that are suitable to simulate a typical river deposit. In addition, a flocculent was added to speed the settling of the silt sized particles.

3.3.1.2 Pipes

A high density polyethylene (HDPE) pipe for this research was chosen due to highly usage of polyethylene pipe for utility infrastructure systems. In the mid 1950's, polyethylene was used as a pipe material at first (PPI 2006). The use of this polyethylene mostly was in oil field production. As the oil and gas industry was grown up quickly, a flexible, strong, and lightweight pipe was vastly needed. The performance benefits of polyethylene pipe in the original oil and gas related applications have led to its use in equally demanding piping installations such as potable water distribution, industrial and mining pipe, force mains and other critical applications where a tough, ductile material is needed to assure long-term performance. A polyethylene pipe could be applied and utilized in many application including gas, municipal, industrial, marine, mining, landfill, and electrical and communication duct applications. Additionally, it can be effectively utilized for above ground, buried, trenchless, floating and marine installations regarding construction methods. Recently, a polyethylene pipe is being highly employed in especially natural gas distribution (PPI 2006).

For this experiment, the properties (i.e. weight and diameter) for a test pipe assembly were provided by The Plastic Pipe Institute (PPI). Three diameters (50, 75, and 100 mm) particularly were selected to be used in the experiment. The Standard Dimension Ratio (SDR) for the HDPE pipes used was as follows: SDR17 for the 50 mm pipe, SDR17 for the 75 mm pipe, and SDR 21 for the 100 mm pipe. These SDRs were determined because they are often utilized in real pipe installations. Also, these diameters can easily be floated due to large SDRs, so this can help save testing time and present a significant difference between theoretical and experimental results quickly.

3.3.1.3 Depth of Cover

The depth of cover is one of the critical factors that must be considered in the design step of watercourse crossing. Once the crossing profile has been taken and the geotechnical investigation complete, a determination of the depth of cover under the crossing is made. For the depth of cover, numerous conditions around the project site must be confirmed: flow characteristics of the river, the depth of scour from periodic flooding, future channel widening/deepening, and the existence of existing pipeline or cable crossings at the location. However, the research did not consider these factors, which were not necessary for the laboratory test. Instead, this research determined the depth of cover is the variable index to compare the behavior of a buried pipeline at river crossings regarding two construction methods. It is expected that the critical depth to diameter ratios obtained by an experimental and a theoretical method could clearly present the difference between two construction methods. In reality, the minimum depth of cover for each construction method is different depending on pipe diameter, material, type, service, and local regulations. The importance of the depth of cover is also emphasized in post-installation or after pipeline operation. In post submerged pipeline installation, the regular inspection of depth of cover is required based on the regulation of Pipeline and Hazardous Materials Safety Administration (PHMSA). The National Transportation Safety Board (NTSB) concluded that the probable cause of the accident was the failure of the pipeline operator to maintain the pipeline at the cover depth to which it was initially installed (PCCI 2006). Thus, it could be proven that the cover depth in both actual design step and post-installation is very important for a river crossing to prevent potential incidents of pipeline exposure or failure.

According to the Pipeline Safety Regulations from the U.S. Department of Transportation (US DOT), for OT installation methods the minimum 1.2m of depth of burial is required for gas pipeline security unless considering scour depth

(ASCE 1996). Additionally, in general pipes installed by HDD have a depth of cover of at least 6 m below the expected future river bottom, after considering scour (PPI 2006). In the section of "Experimental Procedure", how the critical H/D ratios were acquired was described.

With this information an appropriate experiment could be designed and examined three different pipe diameters and two installation methods (HDD and OT) installed at varying depths with the goal of finding the critical ratio of depth to product diameter where the overburden pressure was sufficient to prevent buoyancy from occurring.

3.3.1.4 Creation of Annular Space

This light concrete cast for drilling fluid in the annular space was designed as a unit for the weight of drilling fluid utilized in the actual installation. Figure 3.1 below was shown light concrete mold of a HDD pipeline assembly just after the metal mold was stripped off.

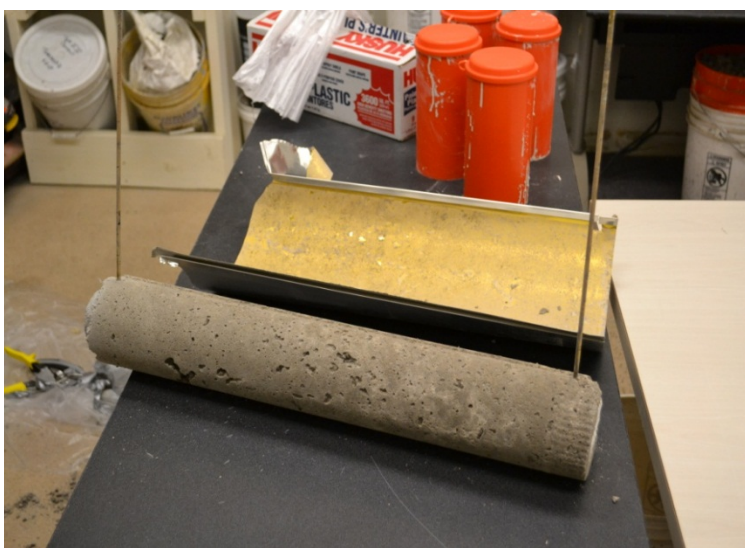

Figure 3.1 Light Concrete Mold for a HDD Pipe Assembly

Based on the pipe buoyancy theory, the critical point is the density of materials (i.e. pipe, surrounding soil, drilling fluid) related pipe installation. The density of drilling fluid in the annular space is associated with calculation of W_B (weight of drilling fluid). Simulating a suitable specific gravity for the drilling fluid that would be comparable to a real world installation was a critical issue in order to obtain reasonable results in the experiment, as the buoyancy forces developed are directly linked in the density (unit weight) of soil around a buried pipe. ASTM (1999) provides an estimate of drilling fluid or bore hole slurry of approximately 1.500 ton/ m^3, however based on the unit weight of the soil utilized in this experiment this would have been too heavy. From field observation, the return mud weight generally falls between 1.200 and 1.350 (ton/m^3) (Duyvestyn 2009). Therefore for the purposes of this research, the approximate mid-point of this range was utilized in the laboratory trials. The simulated annular space cast around the pipes was constructed of a low weight concrete designed to have a unit weight of approximately 1.280 ton/m^3. A sheet metal mold was utilized as a form during the pouring of the concrete for the annular space, and the HDPE pipe was cast in the approximate center of the mold. With this light concrete mold, HDD experiments were conducted following the same procedure of OT experiments

3.3.2 Experimental Setup

A concern for the experimental design is to find a repeated method to conduct this pipe buoyancy test. This is necessary for building an appropriate test apparatus. For this test apparatus, a pre-fabricated metal tank, which is 900 mm in width, height, and length (see Figure 3.2), was created. Two soil layers as a riverbed base were constructed inside the tank (See Figure 3.2 and 3.3). The bottom layer is 120 mm depth of sand and next layer is 230 mm of silt. These two layers are never varied in repetitive tests.

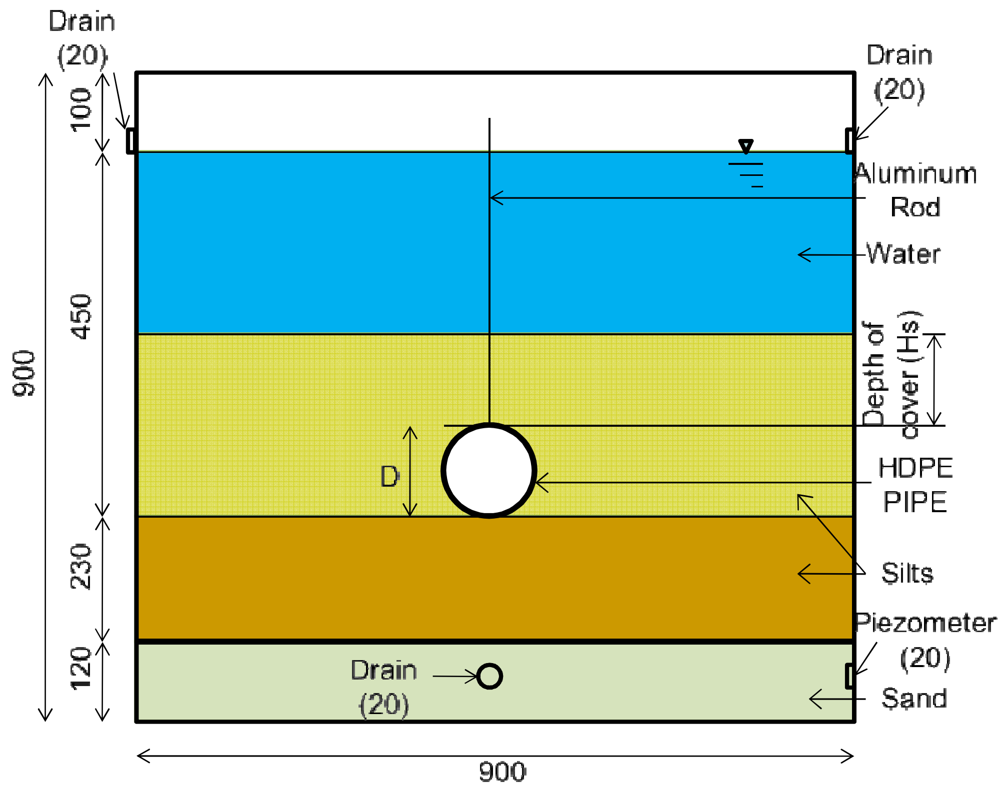

Figure 3.2 Front View of a Manufactured Tank (unit: mm)

Additionally, two 20 mm diameter holes were drilled near the top of the tank, one on the right and one on the left side, and fitted with the proper plumbing to prevent overflows. The other hole was used to mount a piezometer along the right side of the tank. Water table or static water pressure can be checked by a piezometer. A trap door, operating on hinges, measuring 600 mm by 600 mm was attached to the front of the tank to facilitate the deposit and removal of soil (see Figure 3.3 and Figure 3.4).

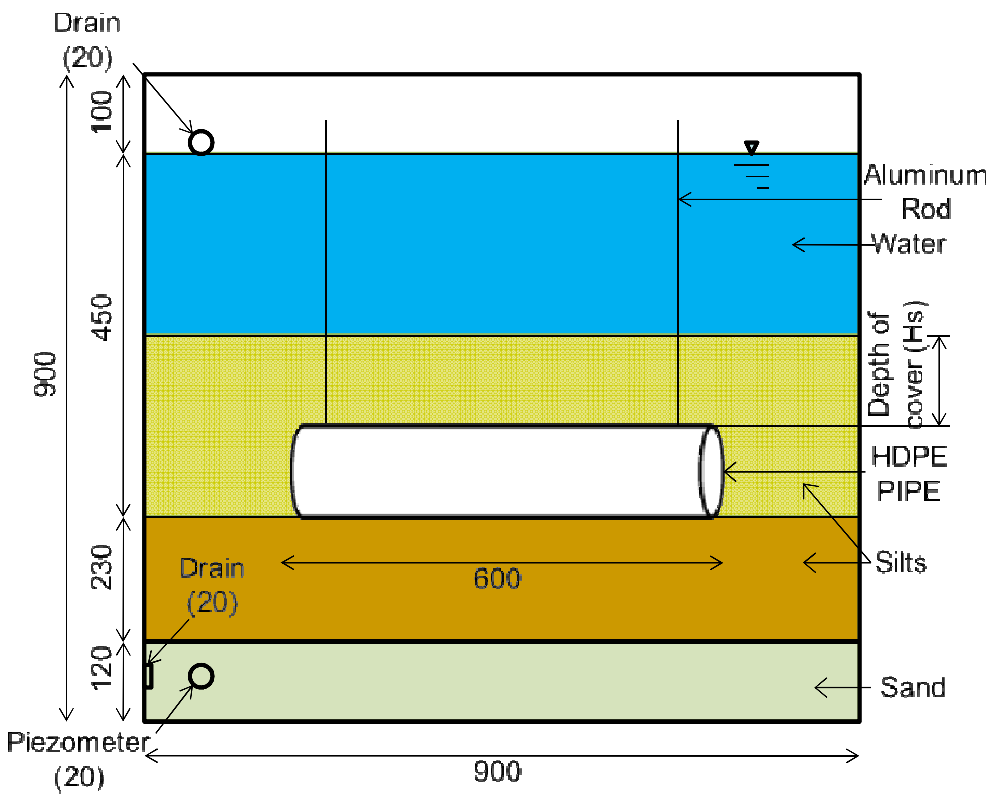

Figure 3.3 Side View of a Manufactured Tank (unit: mm)

Figure 3.4 Fabricated Metal Tank

For simulation of an alluvial soil deposit, soil and water were mixed in the approximate ratios of 1:3 (1 part soil to 3 parts water), and this soil-water mixture was poured into the tank through the dispersion trough that evenly had seven holes to distribute the surface of soil (see Figure 3.11). There are overflow drain ports near the top of the tank for draining the excess water volume as the required number of buckets for the expected depth of cover was added. To install the pipe within the soil deposit, a support frame was constructed above the tank utilizing 12.5 mm diameter threaded rod and clamp holders (see Figure 3.5). From this support frame the pipe was supported for installation at the prescribed depth required for the diameter of pipe and depth of cover being analyzed.

Figure 3.5 Supporting Frame System with Plastic Trough

The pipes were fabricated to be 600 mm in length to fit within the tank and minimize any boundary effects from the sides of the tank (see Figure 3.6). To further minimize these edge effects, the ends of the pipe were capped and covered with Teflon tape to prevent any friction developing on the ends of the pipe. The pipes were held in place utilizing hollow aluminum rods inserted though a small diameter hole drilled into the crown of the pipe, and seated into a shallow divot in the invert of the pipe. The aluminum rod was epoxied into place, and the pipe was sealed to prevent water penetration during the experiment. The configuration of the rod seated in the divot in the invert of the pipe prevented any rotation of the pipe as the soil-water mix was poured into the tank. The two aluminum rods were then secured to the frame above the tank utilizing clamp holders (see Figure 3.6). These hollow aluminum rods provided the rigidity needed to maintain the pipe in the center of the tank and at the required depth, without adding a significant amount of weight or friction from the soil around them to the pipe assembly. Multiple pipe assemblies (with the aluminum rods) were constructed for each diameter tested.

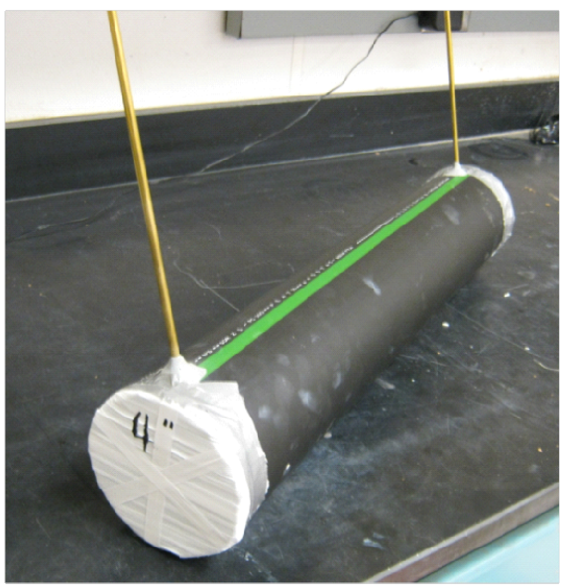

Figure 3.6 Test Pipe Assembly for OT

To conduct the test for HDD technique, a concrete cast around each pipe assembly created an annular space. After testing OT, these used pipe assemblies were simulated using a light weight concrete designed. This light weight concrete is designed based on a unit weight of drilling fluid installed in the annular size. Based on what Ariaratnam and Beljan (2005) mentioned previously, the method of simulating the annular space would be considered an installation completed over a month in the past. The annular space filled by drilling fluid could become a mass that has cohesion, compressive strength, and a low permeability due to hydration and consolidation. As a result, this research devised concrete cast represented the mass of drilling fluid in the annular space. Therefore, the pipe and annular space would act as one composite pipe. The concrete cast for each assembly was painted to minimize the penetration of water into the porous concrete (Figure 3.7).

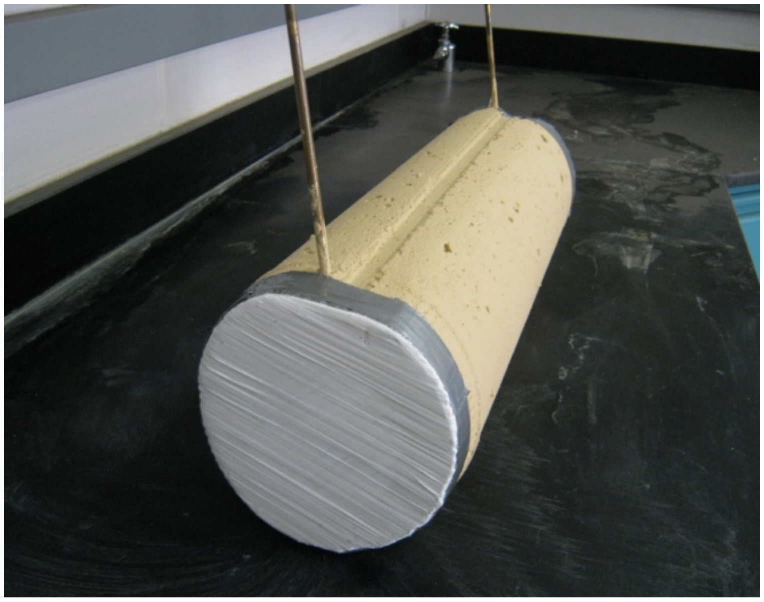

Figure 3.7 Completion of a HDD Pipeline Assembly

3.3.3 Experimental Procedure

3.3.3.1 Open Trench

We developed a standard testing procedure for testing the pipes that included methodologies to install the pipes, and to determine the critical depth (height of soil cover) to diameter (H/D) ratio. A total of four days were required to prepare the tank, install the pipe, determine the depth of cover, and observe if pipe buoyancy occurred. The procedure utilized was as follows:

(1) Preparation of soil bed: Remove silt soil to form 450 mm base above the bottom of the tank. At all times the level of water in the tank was maintained to be at least 50 mm above the top of the soil. Prior to each test the base bedding and water was thoroughly mixed with an industrial paint mixer and then let settle for 24 hours (see Figure 3.8).

Figure 3.8 Preparation of Soil Bed

(2) Pipe Placement: After 24 hours, the pipe assembly was installed. The pipe invert was gently pressed into the soil surface, and the aluminum rods secured to the testing frame above the tank (see Figure 3.9).

Figure 3.9 Pipe Placement

(3) Backfill Placement: Once secured the soil-water mixture was poured into the tank utilizing a dispersion trough to ensure an evenly distributed placement of the soil. As discussed previously, the soil-water mixture was added at the 1:3 volume proportions. (see Figure 3.10).

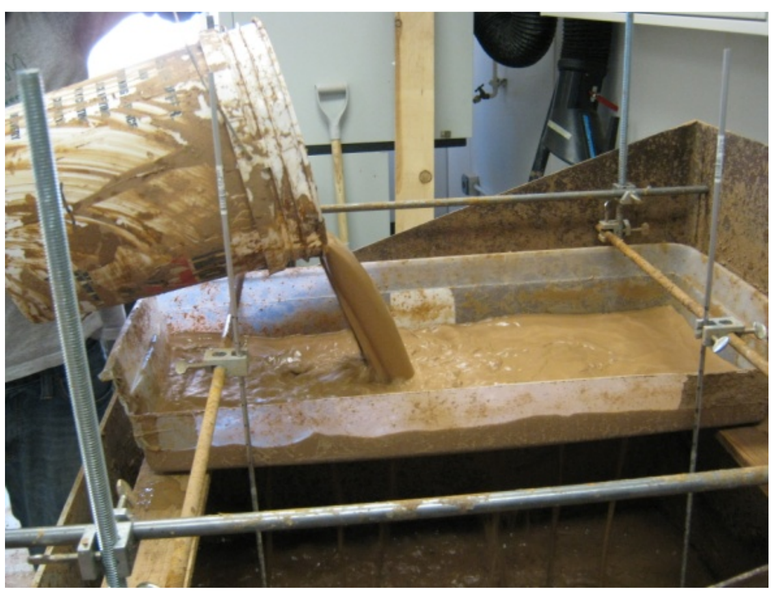

Figure 3.10 Backfilling Placement

Through 7 holes at regular intervals on the trough, soil-water mixture was dropped equally and consistently (see Figure 3.11). Prior to each pipe installation a calculation was made to estimate the amount of soil needed to achieve the targeted depth of cover being examined. After the required amount soil was added to the tank it was left to sit for 24 hours allowing a brief period of consolidation of the soil particles around the pipe. The flocculent added to the soil assisted in this process. The water level in the tank was kept at or near the overflow level to ensure that the soil was always fully submerged during the test.

Figure 3.11 Distribution of Soil-Water Mixture

(4) Determination of the critical depth of cover: After the second 24-hour period the actual depth of cover over the pipe was measured. Check the depth of cover at the determined three points (front, middle, and

end) of a buried pipe, and average them (see Figure 3.12). Then the clamps holding the aluminum rods on the pipe assemblies were gently released allowing free movement of the pipe. Allow 24 hours for observation of pipe floatation (see Figure 3.13).

Figure 3.12 Recording the Depth of Cover

The assembly was watched for another 24 hours to determine if the pipe buoyed out of the soil. If after this time period no floatation occurred, this procedure was repeated for an incrementally smaller depth of cover. Alternatively if buoyancy occurred the procedure was repeated with an incrementally larger depth of cover until floatation no longer occurred. This procedure was repeated numerous times for each pipe diameter with the objective of finding the critical depth of cover where buoyancy occurred.

Figure 3.13 Releasing the Clamps

3.3.3.2 Horizontal Directional Drilling

For HDD experiment, a light weight concrete mold pipe (see Figure 3.7) was used. Only the difference of the experimental procedure between OT and HDD is the use of different pipe assemblies. The procedure of the pipe floatation test for HDD is different from the actual HDD construction procedure, and moreover this situation is not fully representing in simulating actual HDD boreholes in the riverbed. However, the buoyancy factor is closely connected with the density of pipe surroundings according to Archimedes' buoyancy theory. Hence, concrete mold pipe assemblies for the annular space could satisfy we in simulating buoyancy factor of actual HDD pipeline installation and finding the difference of buried pipeline behavior. Figure 3.14 is shown in the experimental setup for HDD test.

Figure 3.14 Experimental Setup for HDD

In summary, the chapter described the detail laboratory setup and procedure to simulate the buoyant behavior of a buried pipeline installed by OT and HDD. The full-factorial laboratory tests were created. The buoyancy effects were described as critical H/D ratios that are minimum value of depth of cover ratios that prevent pipe buoyancy at varying factors. Diverse relationships between factors were also discussed in this research.

Chapter 4: LABORATORY TESTING AND RESULTS

4.1 Introduction

This chapter describes the results of pipe floatation experiments in terms of different construction methods (open trench: OT and horizontal directional drilling: HDD). Three diameters (i.e. 50, 75, and 100 mm) of high-density polyethylene (HDPE) pipe were utilized in this experiment. To analyze the results of buoyancy effect, the critical H/D ratios are calculated by theoretical method and acquired by laboratory tests. For this analysis of experimental results the research utilized two types of comparisons. Firstly, the experimental results were compared with theoretical ones that were computed in Chapter 2. This comparison will show the difference between pipeline installation design and actual pipeline construction in river crossings. For instance, the first comparisons were between a theoretical and experimental method in both traditional OT and HDD. This result shows whether or not the design standard utilized in real life correctly anticipates the actual behavior of a buried pipeline through the experimental simulation. Secondly, the chapter presented comparison of buried pipeline behavior between OT and HDD. From this comparison, the differences of pipeline behavior between OT and HDD at river crossings are revealed in Chapter 4. If there are, the cause of this difference will be investigated and analyzed. With all outcomes at the end of this chapter, the researcher mentions general trends, findings, and detail analysis about the pipe floatation research.

4.2 Experimental Results

A total of 57 tests were conducted for both open trench (OT) and horizontal directional drilling (HDD) installation. Fifty, seventy five, and one hundred mm HDPE pipe respectively was utilized and tested for obtaining the critical depth of cover with and without a simulated annular space. The critical H/D ratios obtained by OT and HDD respectively were compared to understand the pipeline behavior. To obtain the targeted depth of cover in the test, this experiment went through many trials and errors by adding the number of buckets where soil water mixture is. As more experimental runs were executed, the behavior of the soil and water mixture could be anticipated as well, and this led to the effective examination that determined the critical depth of cover. Depending on pipe diameters, the experimental results of each construction method were presented. As mentioned in the section, "Experimental Procedure," the critical H/D ratio was recorded whether or not a pipe floats to the surface. If a buried pipe with a given depth of cover surfaces, this depth is recorded as the floating depth, and the next step is to increase the depth again and check whether or not the pipe floats. This step is repeated until the pipe stops floating. Once a buried pipe does not float, the depth of cover at this time is recorded as the critical depth. This critical depth from the experimental setup is the value that is compared with the one calculated by the theoretical method (the buoyancy theory). The cover depth on the pipe should be checked before a buried pipe is freely released in fully saturated submerged situations. In order to record the depth of cover, three checking points (the front, middle, and end) on the pipe were used and the average value of three checking points was computed as the checked depth of cover for each test. Also, these three depths of cover on the pipe were used to confirm that mixture of soil and water equally distributes in the experiment, because the unbalanced grade over a buried pipe causes undesirable pipe floatation, which disturbs the accurate experiment. In the experiment, the allowable discrepancy among three checking points was determined within 1 cm after the soil mixture settles down. If the discrepancy is over 1 cm, the cover depth recorded is judged as an unreliable data for checking pipe floatation. The average number of tests was approximately 11 for OT and 8 for HDD. In order to determine when the experiment should stop, the paper regulated the allowable discrepancy for the critical H/D ratio recorded between tests as 0.05. If the discrepancy between the critical H/D ratio and the nearest value of the critical one is over 0.05, the test should be repeated to acquire a narrower and more accurate H/D ratio. The test of each diameter could end easily if the critical H/D ratio is quickly determined within this criterion. All records for recording critical H/D ratios in the experiment were shown in Appendix A and B.

4.2.1 Open Trench

Following the experimental steps in Chapter 3, the experiment in traditional open trench (OT) was started for each diameter (50, 75, and 100 mm HDPE pipe). Table 4.1 showed the result summary of the pipe floatation experiment in traditional OT method.

Table.4.1 Results of Pipe Floatation in OT Experiment

Pipe Size (mm): [OD]	Depth of Cover		Depth from the bottom of Pipe (cm)	Discrepancy of grade (cm); within 1(cm)
	Cover Depth (cm)	H/D ratio		
50 [60.3]	9.10	1.51	15.126	0.317
75 [88.9]	11.53	1.30	20.422	0.317
100 [114.3]	19.76	1.73	31.191	0.190

The next sub-sections described the test results of each diameter simulated by OT method.

4.2.1.1 50 mm

In a 50 mm HDPE pipe, the critical H/D ratio was 1.51 and the depth of cover 9.1 cm (1.51 × 6.03). This means when a pipe is installed at a minimum H/D ratio of 1.51, an installed pipe is safe against pipe buoyancy in submerged soils. The discrepancy of cover depth obtained from three checking points shows that the mixture of soil and water easily distributes equally. If the three checked points are not evenly distributed, a buried pipeline in the experiment might unexpectedly surface. The lack of cover depth at certain positions leads to a deficiency in the uplift resistance force. In a 50 mm HDPE pipe, the discrepancy of grade at 1.51 H/D ratio was 0.317 cm. Thus, 1.51 H/D ratio could be regarded as a reasonable result for 50 mm HDPE pipe, because the discrepancy of grade at the H/D ratio of 1.51 is within the standard (1cm) mentioned. The number of pipe floatation tests for a 50 mm HDPE pipe is 11. A 50 mm HDPE pipe (0.64 kg/m) is very light, so the experiment for a 50 mm HDPE pipe took more days than the other diameters so as to acquire the critical depth of cover. The average of bucket numbers is approximately 18. To narrow the range between depth-to-diameter (H/D) ratios and to confirm whether or not the ratio found is the critical depth of cover, the researcher conducted several tests for an accurate critical H/D ratio. The testing data was shown in Appendix A.

4.2.1.2 75 mm

In a 75 mm HDPE pipe, the final result says that a 75 mm HDPE pipe does not float if the H/D ratio is guaranteed more than 1.30, meaning the depth of burial must be over 11.53 cm in Table 4.1. The final number of tests for 75 mm was 12. In this experiment, the researcher found the importance of equivalent grade at three checking points (front, middle, and end). When one test had the H/D ratio of 1.23 (see Appendix A), which was lower than the critical H/D ratio (1.30) determined, a buried pipe did not float because the check point at the middle was higher than at other end points, and moreover, 1.58 cm of the grade discrepancy was over the standard that the research decided (1cm). If the checked point at the middle is higher than at other end points, a buried pipe might have the probability to be secured in soils. Hence, this recorded cover depth was regarded as unreliable due to the unequal grade for the depth of cover. Again, as mentioned in the result section of a 50 mm HDPE pipe test, achieving the uniform level at the grade under the water would become an issue in traditional OT installations. If backfilling does not achieve the equalization of grade along the pipeline installed, a buried pipe could float even if the buried pipe has the suitable depth of cover designed by accurate calculation. Therefore, the critical H/D ratio in a 75 mm HDPE pipe was determined at 1.30, which had well-distributed ground level (see Table 4.1 or Appendix A).

4.2.1.3 100 mm

In a 100 mm HDPE pipe, the critical H/D ratio for cover depth was 1.73. This ratio is a little higher than that of the 50 or 75 mm HDPE pipe. As described above, the 100 mm HDPE pipe had a different standard dimension ratio (SDR21) that was relatively lighter and thinner contrary to SDR17 for the 50 and 75 mm HDPE pipe. This explains why the 100 mm HDPE pipe could have a higher critical H/D ratio. This pattern, the highest H/D ratio in a 100 mm HDPE pipe, was shown in the results of the theoretical method as well. Soil volume replaced by a 100 mm HDPE pipe is largest, meaning the buoyant force is also largest. Subsequently, a 100 mm HDPE pipe requires more cover depth for secure pipe installation compared to other diameter sizes. The grade discrepancy for subsurface in a 100 mm HDPE pipe was 0.19 cm at the critical H/D ratio (1.73).

Consequently, the results of cover depth by each diameter in OT experiment show that a bigger HDPE pipe obviously needs more soil cover for safe installation because the buoyant force becomes higher due to large soil volume replaced by a buried pipe. The range of the critical H/D ratio for the experimental results was 1.30 to 1.73, and the average value was 1.51.

4.2.2 Horizontal Directional Drilling

The summary of the experimental results in HDD method is shown in Table 4.2 below.

Table.4.2 Results of Pipe Floatation in HDD Experiment

Pipe Size (mm): [OD]	Borehole Size (mm)	Depth of Cover		Depth from the bottom of Pipe (cm)	Discrepancy of grade (cm); within 1(cm)
		Cover Depth (cm)	H/D ratio		
50 [60.3]	101.6	2.36	0.39	8.395	0.063
75 [88.9]	127	8.08	0.91	16.967	0.508
100 [114.3]	177.8	9.35	0.82	20.777	0.190

Since this section, the pipe floatation results for each diameter in HDD method are presented. In Table 4.2, the critical H/D ratio of each diameter was inconsistent. They had the discrepancy of 0.52 among three results. The result happened due to the different sizes of the annular space that was fabricated by concrete. This analysis was also described in the end of this chapter.

4.2.2.1 50 mm

The critical depth of cover in a 50 mm HDPE pipe in HDD test was 2.36 cm (H/D ratio: 0.39) in Table 4.2. The discrepancy between checking points was within 1 cm (0.063), meaning soil particles were equally distributed. The number of HDD test for the 50 mm pipe was 8. The average number of buckets utilized in 50 mm pipe experiments was about 8 buckets, which is lower than OT. The borehole diameter (BD) of 100 mm HDPE pipe was utilized for a 50 mm HDPE pipe. This BD is 1.68 times the outer diameter (OD: 60.3 mm), which was larger than 1.5 times OD that the research planed.

4.2.2.2 75 mm

The 75 mm HDPE pipe assembly for HDD test had the BD of 5. The BD was 1.43 times greater than 75 mm outer diameter (88.9 mm). The experiment informed the critical depth of cover for 75 mm pipes was 8.08 cm (H/D ratio: 0.91). The critical H/D ratio of 0.91 was relatively high against the result of a 50 mm HDPE pipe. The test number was 7, and the average of buckets used was 15 so as to obtain the aiming depth of cover.

4.2.2.3 100 mm

A 100 mm HDPE pipe assembly for HDD was manufactured in 114.3 mm diameter BD, which is 1.56 times greater than the OD of an original product pipe. This size was nearly close to the planned BD (1.5 times OD) compared to other diameters. As a result of the experiment, a 100 mm HDD pipe assembly was secured when it was installed at the minimum depth of 9.35 cm (H/D ratio: 0.82). The grade after pouring soil-water mixture was well-distributed, not being over the research limit, 1 cm. The test number was 6.

To sum up for the results of HDD pipe floatation tests, it was clearly revealed that the critical depth of cover or H/D ratio depended on the ratio between borehole diameters (BD) and outer diameters (OD). As the ratio increases, the depth of cover required decreases. It means that the annular space has an influence on determining the depth of cover. The detail analysis of the experimental results is shown in the next section of the comparison.

4.3 Result Comparisons between Theory and Experiment

The ways to compare result data were divided into 4 types. The first and second one was to compare the theoretical and experimental results in traditional open trench (OT) and horizontal directional drilling (HDD) method respectively. The buoyancy theory (see Chapter 2) from Archimedes' Theory represented the pipeline design theory for watercourse projects. The experimental results could represent the simulation of actual buried pipeline behavior. Hence, understanding the difference between pipeline design theory and actual buried pipeline behavior is good to analyze regardless of pipeline construction methods. The third and forth section were to compare buried pipeline behavior between different construction methods (OT and HDD). The focus of this analysis is to reveal the differences between buried pipeline behaviors in terms of construction methods.

4.3.1 Comparison: Theory vs Experiment in OT

Comparing the results of each of the pipe floatation tests from theory and experiment, allows the researcher to understand the gap between the actual behavior of buried pipelines from the experiment and theoretical calculation based on the Archimedes theory. Table 4.3 is shown the results of pipe floatation in OT.

Table 4.3 Critical H/D Ratios in OT

Nominal Pipe Diameter (mm)	Pipe Outside Diameter (mm)	SDR	Critical H/D Ratio in OT	
			Theory	Experiment
50	60.3	17	3.0	1.51
75	88.9	17	2.99	1.30
100	114.3	21	3.15	1.73

In the theoretical results of OT, the critical H/D ratios from all three different sizes seem to be very analogous. The average of these results is 3.05. On the contrary, the mean of the experimental results is 1.51, which is approximately half of the average experimental results.

4.3.2 Comparison: Theory vs Experiment in HDD

For a HDD method, the pattern of the final pipe floatation data in Table 4.4 was not consistent due to the different ratios between OD and BD. Table 4.4 presented the final depth to diameter (H/D) ratios for each diameter.

Table 4.4 Critical H/D Ratios in HDD

Nominal Pipe Diameter (mm)	Pipe Outside Diameter (mm)	Borehole Diameter (mm)	GS Annular Space (actual)	Critical H/D Ratio in HDD	
				Theory	Experiment
50	60.3	101.6	1.35	0.52	0.39
75	88.9	127	1.34	1.27	0.91
100	114.3	177.8	1.35	0.96	0.82

The clear thing found in Table 4.4 was unusually the larger cover depth ratio required in a 75 mm HDPE pipe. Based on the ratio between BD and OD for the 75 mm pipe assembly, the borehole had a 1.43 times larger ratio than the 75 mm outer diameter. This number is relatively lower than other ratios calculated. This could require more depth of cover or depth to diameter ratio comparatively. This trend in Table 4.4 was observed with both a theoretical and experimental result. Conversely, a 50 mm pipe assembly for HDD had a larger ratio (1.68) than a 75 or 100 mm pipe assembly. So, the critical depth of cover was small. Theoretical data still produced larger ratios than experimental data. This was the same pattern as the OT test shown above. The average ratio obtained in the theoretical method was 0.92, but in the experimental method was 0.71, which is roughly 80 % of the theoretical average. This is not a big difference compared to the results from OT in Table 4.3, but this still proves that the theory anticipates more depth of cover for secured pipeline installation.

4.3.3 Comparison: OT vs HDD in Theory

Three kinds of diameters (50, 75, and 100 mm) in HDPE pipes were considered to understand the trend of pipe floatation through theoretical calculation. The average critical H/D ratios in OT using the design buoyancy theory was 3.01, which is approximately 3.3 times greater than that in HDD (0.92). Based on this comparison, OT needs more burial than HDD, meaning the pipeline installed by HDD could be located at higher position compared to the one by OT. The detail results are shown in Table 4.5 below.

Table 4.5 Critical H/D Ratios in the Theoretical Method

Nominal Pipe Diameter (mm)	Pipe Outside Diameter (mm)	Borehole Diameter (mm)	GS Annular Space (actual)	Critical H/D Ratio in Theory	
				OT	HDD
50	60.3	101.6	1.35	3.0	0.52
75	88.9	127	1.34	2.99	1.27
100	114.3	177.8	1.35	3.15	0.96

This result informed that the unit weight (density) of drilling fluid replacing natural soil gives more stability to a buried pipeline. By checking specific gravity (GS for the annular space) in the forth column in Table 4.5, the weight of drilling fluid (1.35) is heavier than the replaced soil (1.27, see Chapter 2). The buoyancy theory highly depends on the density of materials. The buoyancy theory highly depends on the density of each material (see Chapter 2). As a result, it was expected that the larger density of drilling fluid installed might play a major role for increasing the uplift resistance force that leads lower critical H/D ratios. Mostly, the deeper the pipeline is buried, the higher the construction cost becomes because there is a greater chance of facing solidified soils or bedrock, thus causing drilling delays.

4.3.4 Comparison: OT vs HDD in Experiment

The comparison showed the different pipeline behavior by construction methods using the experimental method. Table 4.6 shows the experimental results of the pipe floatation in terms of open trench (OT) and horizontal directional drilling (HDD) respectively.

Table 4.6 Critical H/D Ratios in the Experimental Method

Nominal Pipe Diameter (mm)	Pipe Outside Diameter (mm)	Borehole Diameter (mm)	GS Annular Space (actual)	Critical H/D Ratio in Experiment	
				OT	HDD
50	60.3	101.6	1.35	1.51	0.39
75	88.9	127	1.34	1.30	0.91
100	114.3	177.8	1.35	1.73	0.82

All critical H/D ratios were relatively small compared to the theoretical results above. OT (1.51) required approximately twice as much depth of cover than HDD (0.71). This pattern was discovered in the theoretical method as well.

4.3.5 Summary of Result Comparisons

In order to compare the behavior of a buried pipeline installed by traditional OT and HDD, the critical depths of cover (H/D ratios) were respectively acquired from both theoretical and experimental method. The meaning of the critical H/D ratio is the minimum value of the H/D ratio that can guarantee the safety of a buried pipeline. If a critical H/D ratio is small, the pipeline installed with small soil cover has enough uplift resistance force preventing pipe floatation. At second hand, the small critical H/D ratio could denote that ground or subsurface status around a buried pipe would be stable. To obtain these final H/D ratios, four comparisons were introduced and described for the analysis of buried pipeline behavior in saturated silty soils. Overall, each comparison fully reached the following two final conclusions.

1) The critical H/D ratio in HDD method required less depth of cover to be secured.

2) Compared to the depth of cover obtained by the theoretical method, the laboratory test produced a smaller depth of cover for the pipe security.

These conclusions are supposed to be proven by reasonable and logical analysis, which was described in the next section.

4.4 General Trends, Findings, and Detail Analysis

4.4.1 Effect of the Annular Space

Based on what the research found above, the discrepancy between the buried pipe behavior of traditional open trench (OT) and horizontal directional drilling (HDD) was revealed in saturated submerged soils. Subsequently, it could be confirmed that traditional OT method requires more depth of cover for pipe security. The reason for these results was found in the effect of installed drilling fluid in HDD. Ariaratnam and Beljan (2005) mentioned that the annular space visually disappeared in the status of the borehole in post-construction. The integrity of borehole did not change and the unconfined shear strength around annular space increased in the one year cross-sectional excavation of HDD. Also, the uplift resistance force in the borehole increased in the one year excavation compared to one day, one week, and one month. These theories support that the stability of a pipeline will be well-maintained as time goes by. In the case of borehole location above the groundwater table, the density of the installed slurry mud became similar to the surrounding soil formation (Knight et al 2001). This research simulated and considered when buried pipeline installation is completed over a month. As described before, one of the major roles of drilling fluid in HDD is to protect the borehole wall from collapsing at the beginning of HDD construction. To stabilize the formation of the borehole, drilling fluid forms the filter cake around the border of borehole. This filter cake composes the low permeability zone within the borehole, preventing infiltration or exfiltration for some time. By low permeability of the filter cake, the stability of buried pipeline is determined early in the HDD installation. This filter cake may help maintain the original density of drilling fluid, replacing saturated surrounding soil. In my research, it was assumed that the borehole wall was surrounded by filter cake that protects infiltration from outside. Thus, the original density in drilling fluids would be maintained in composing the annular space in a month.

Another consideration of the annular space related pipe buoyancy is the existence of filter cake. The research assumed that the filter cake was well-built in producing low permeability in order to simulate the annular space created by light concrete cast. If filter cake was not existed or in poor status, it would lead to high permeability around the annular space. The high permeability causes fluidal movement for adjacent soils of a buried pipeline due to higher pore pressure. The roles of filter cake make stable borehole to prevent infiltration or exfitlration between borehole and native soils. This action guides an increase of effective stress and low permeability. Finally, a filter cake is very critical for reducing the excess pore pressure in the annular space that causes fluidal movement of soils due to the static liquefaction (Wang and Sterling 2007).

The irregular trend of the critical H/D ratio obtained in HDD test could support the importance of borehole design or construction associated with buried pipeline behavior. The irregularity of the critical H/D ratio from each diameter must be caused by the different ratios which are supposed to be 1.5 between BD and OD. BD is generally obtained by each outer diameter (OD) times 1.5 which is being used when product diameter (D) is more than 203.2 mm and less than 609.6 mm (Bennett and Ariaratnam 2008). The other way for BD size is 100 mm plus outer diameter when product diameter is less 20 cm (Bennett and Ariaratnam 2008). In this research, the theoretical calculation only considered the former one, which has general application for the size of the borehole in actual pipeline projects. BD utilized in the research was created by concrete molds that were not perfectly the same size of 1.5 times OD. It was too difficult to find the anticipated sizes of concrete molds that could simulate the annular space for HDD test pipe assemblies. This also happens in actual construction when constructing the annular space design and the ideal borehole size is not easy to acquire (1.5 times OD) accurately in real installations. Due to difficulty of finding a suitable size for a mold, each borehole (diameter of annular space), for the size of 50, 75, and 100 mm HDPE, had somewhat different sizes compared to the average size rule. In theory, each BD (actual) in the experiment had a small difference from an ideal BD. The 50 mm concrete mold pipe had a borehole size of 101.6 mm, which is 1.68 times nominal the outer diameter of a 50 mm HDPE pipe. The 75 mm concrete mold had a borehole the size of 127 mm, which was 1.43 times nominal outer diameter. Lastly, a 100 mm concrete mold (177.8 mm of the annular size) had 1.55 times the nominal outer diameter of a 100 mm HDPE pipe. All this data for the experiment was used in the calculation of the pipe buoyancy theory mentioned in Chapter 3 for comparative purposes. Depending on different ratios, the critical H/D ratios for pipe floatation were varied irregularly. The HDD pipe assembly having large BD size had a small H/D ratio (see Table 4.4, 4.5 or 4.6). Conversely, a small ratio between BD and OD (1.43: 75 mm pipe mold assembly) led to a large critical H/D ratio. Hence, it could be concluded that if the BD to OD ratio is smaller than 1.5, the critical H/D ratio would be increased. On the contrary, if the BD to OD ratio is larger than 1.5, the critical H/D ratio would be decreased. Overall, the result informed that BD is closely linked with pipeline behavior under the watercourse.

Lastly, the major reason for the dissimilarity of pipeline behavior between OT and HDD was revealed by different densities between native saturated soils and drilling fluid. Cheuk et al (2008) mentioned increasing the uplift resistance force for a buried pipe is dependent on the density of soil cover over a buried pipe no matter what kind of soil is utilized. The unit weight of installed drilling fluid planned in the research was 1.281 ton/m^3 (specific gravity: 1.28) that is the average value of general drilling fluid returns found in actual HDD installations (Duyvestyn 2009). In the research, larger specific gravity (average 1.35) of the annular space simulated in this experiment resulted in lower critical H/D ratios in theory and experiment. The unit weight of experimental soil used in this calculation is 1.197 ton/m^3, which is smaller than the density of drilling fluid simulated. The increase of the density of drilling fluid led the increase of the uplift resistance force that results in a small critical H/D ratio. As a result, the density of drilling fluid must be a decisive factor to make those differences between OT and HDD. Hence, the specific gravity of drilling fluids designed and constructed in the annular space could be a key point for buried pipeline behavior. Every status between native soil and drilling fluid in real world is not always same as this pipe floatation test.

To sum up, the aforementioned analysis is sufficient for obviously proving the existence of the effect of the borehole about buried pipeline behavior in both OT and HDD. The researcher can summarize three conclusions about the effect of the borehole.

1) The ratio between BD and OD affects the critical depth of cover (H/D ratio).
2) The density of drilling fluid constructed in actual river crossings is very critical for installed pipeline security.
3) Composing well-built filter cake around the borehole wall helps to prevent pipe buoyancy.

4.4.2 Theory vs Experiment

One of the most critical points that the research found was to discover the difference of buried pipeline behavior in terms of analytical methods, such as a theoretical method (the buoyancy theory) and experimental method. The theoretical method (the buoyancy theory) has been used for checking the buoyant influence of a buried pipeline. Accordingly, designing the depth of cover is based on the calculation of the buoyancy theory that was utilized as the theoretical method. Hence, the theoretical results could be regarded as the outcomes from the actual pipeline design theory. Alternatively, the real-scaled experiment represents the actual behavior of pipeline installation. As comparing the results between the theoretical and experimental method, the theoretical method resulted in a larger critical H/D ratio. It means that the actual behavior of a buried pipeline was not the same one as the buoyancy theory forecasts.

4.4.3 Consideration of Soil Friction Effect in the Buoyancy Theory

Previous literature demonstrates that shear strength parameters are important in estimating uplift resistance force. White et al (2001) presented the equation of peak uplift resistance per unit length (P) that is defined as the sum of overburdened soil weight and the soil friction on the slip planes. Basically, this equation was referred from Schaminee et al (1990). Eq. (7) is about the calculation of uplift resistance force from White et al (2001) and Cheuk et al (2008).

$$P = \gamma' H_f D + K \gamma' \tan\Phi H_f^2 \qquad (7)$$

K is the earth pressure coefficient at rest, and the cover depth (H_f) is the distance from the ground surface to the waist of the pipe, and γ' is the unit weight of submerged soil. This equation is for calculation of peak uplift resistance force when upheaval buckling happens. The first portion ($\gamma' H_f D$) in Eq. (7) was the weight of soil overburden (W_s) that is also described in the buoyancy theory. The second part ($K \gamma_{sub} \tan\Phi H_f^2$) in Eq. (7) describes the soil friction. In brief, the soil friction calculated by shear strength parameter (friction angle) helps a buried pipe stay safe in submerged soil due to increasing uplift resistance force. Thus, the soil friction factor on the slip planes can be added in the original buoyancy theory presented in Eq. (1). The varied buoyancy theory was completed by adding the formula of soil friction to describe the relation between pipelines and soil particles. In the calculation of the transformed buoyancy theory, it assumes that a natural friction angle in soil never varies in a saturated condition.

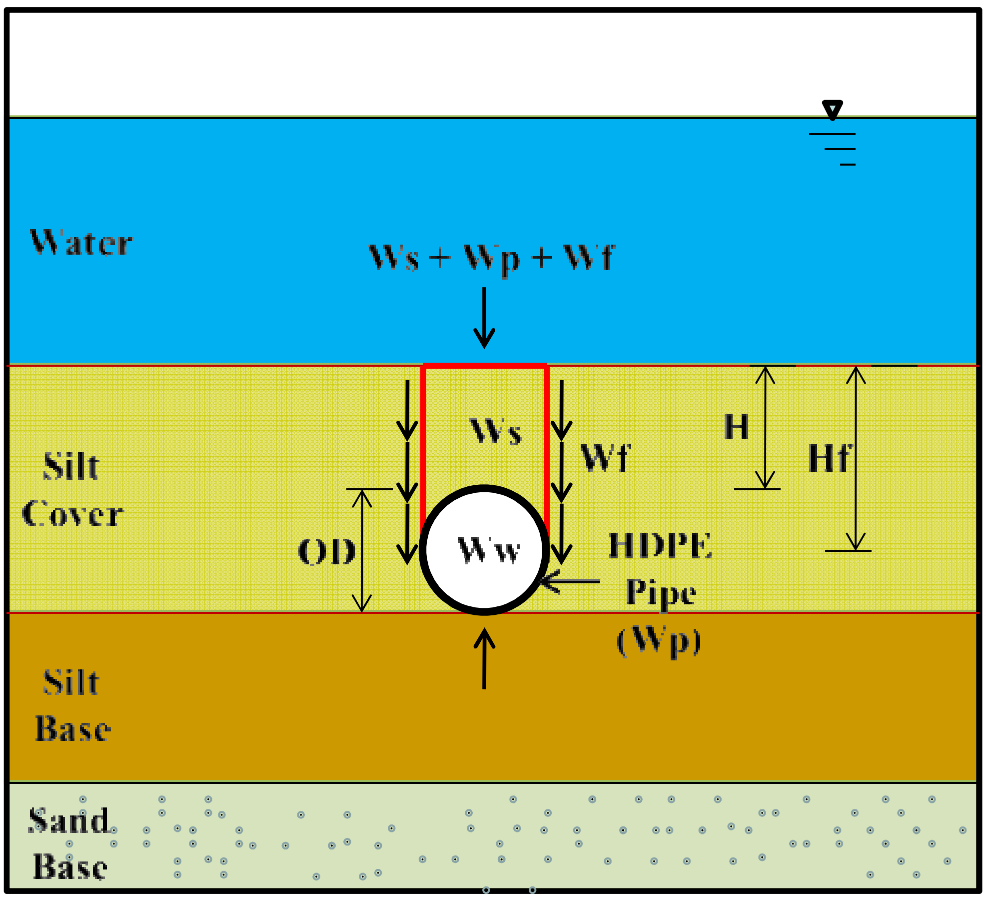

Figure 4.1 Shearing Resistance of Soil with Vertical Slip Surface

The shear resistance formula is diverse in terms of several soil failure mechanisms or assumptions. In our research, the sliding block with vertical slip surface (see Figure 4.1) is assumed as a deformation pattern for soil failure. Eq. (7)

was made assuming sliding block with vertical slip surface. Also, it was the general and simple formula for uplift resistance force. Since Eq. (7), other equations have been developed in several deformation patterns depending on different assumptions and conditions. Different kinematic mechanisms lead a series of theoretical models for uplift resistance (Trautmann et al 1985; Schaminee et al 1990). Nevertheless, the accepted solutions through those models have not been determined and investigated as exact solutions for uplift resistance force (Kvalstad 1999). Hence, it was truly hard to find the exact application method for soil failure mechanisms regarding massive assumptions and different conditions. Finally, the given assumptions and equations from uplift resistance force completed the transformed equation for pipe floatation that could be applied in real pipeline installations. The next equation is a new equation about pipe floatation by adding soil friction effect caused by shear strength parameter (W_f).

$$W_s + W_p + W_f < W_w \tag{8}$$

Eq. (8) is regarded as the transformed buoyancy theory by the adding soil friction factor. Eq. (8) says that inherent shear strength in certain soil could affect stopping soil failure by pipe floatation in saturated conditions. The soil friction factor obtained by an internal angle of friction could be a missing one in the original buoyancy theory. Based on Eq. (8), for pipe security the weight of soil overburden plus pipe and soil friction must be greater than the weight of water replaced by the pipe and buoyant force. The critical H/D ratios obtained by the transformed buoyancy theory were compared with one of the experimental method in Table 4.7.

Table 4.7 Comparison between the Transformed Buoyancy Theory and Experiment in OT

Nominal Pipe Diameter [OD] (mm)	Critical H/D Ratio in OT			
	Transformed Theory		Experiment	
	Depth of Cover (cm)	H/D ratio	Depth of Cover (cm)	H/D ratio
50 [60.3]	4.11	1.73	9.10	1.51
75 [88.9]	6.03	1.72	11.53	1.30
100 [114.3]	8.06	1.79	19.76	1.73

As a result, each of the H/D ratios acquired by the transformed buoyancy theory came fairly close to H/D ratios in the experiment. The results of the transformed buoyancy theory still have safe H/D ratios compared to the experimental results. The results shown in Table 4.7 were produced by applying to one of soil plastic failure assumptions. These results would be changeable depending on what kinds of soil failure assumptions were utilized. Also, this transformed theory was only calculated for OT. Table 4.7, however, presented that considering the soil friction factor for pipe floatation test seem to be reasonable, even if further research in association with soil friction factors for pipe buoyancy will be required in the future.

Simulating the experiment for pipe floatation was a good way to show how a buried pipeline at water crossings behaves in real pipeline construction.

However, we found that the conservative buoyancy theory produced half of a critical H/D ratio acquired from the laboratory tests. Hence, the varied results calculated by using the transformed buoyancy theory are good enough to satisfy real application of pipeline installation at river crossings compared to the previous results calculated by the original buoyancy theory. The transformed theory produced a more accurate critical H/D ratio than the original theoretical method, while this result analysis was only confirmed for traditional open trench (OT). Accordingly, the soil friction factor could be a related factor that caused difference between theory and experiment. The soil friction factor requires engineers and contractors to obtain an internal angle of friction or dilation angle that are barely investigates real installation projects due to economical or external constraints. For this reason, most pipeline projects overlook the soil friction factor. Nevertheless, this reformative result shown in Table 4.7 enlightens engineers and contractors about the importance of the soil friction factor calculated by the shear strength parameter for the economical design of pipeline installations at river crossings. At this point, this topic will need more detail research work in the future.

In summary, this chapter found the annular space in HDD influences the behavior of a buried pipeline submerged in saturated silty soils. The density of the annular space caused different results from OT laboratory tests. This chapter also found the results of the conventional buoyancy theory are different from those of the laboratory tests.

Chapter 5: NUMERICAL ANALYSIS FOR PIPE BUOYANCY

5.1 Introduction

This chapter presents the results of numerical simulation for buoyancy behavior. The objectives for using finite element method (FEM) were summarized as two things: 1) comparing maximum soil stresses occurring in soil overburden between open trench (OT) and horizontal directional drilling (HDD) and 2) studying the pattern of maximum soil stress occurring in soil overburden when critical design parameters, a density and diameter in the annular space, are varied. Firstly, Chapter 5 describes the behavior of a buried pipeline through comparing the soil stress occurring in soil overburden between OT and HDD methods. Soil overburden is an important zone related to pipe buoyancy. When engineers examine plastic soil failure in riverbeds, maximum soil stress occurring in soil overburden is the critical factor that must be within yield soil stress. If maximum soil stress occurring in riverbed is over the limit of yield soil stress, soil could be deformed, which leads unstable ground conditions. Thus, understanding and comparing the pattern of soil stress post-HDD and OT installation is required to confirm the stability of pipeline installation. Secondly, the research also discovers how the annular space in HDD method influences pipeline behavior under rivers. For this, we examined the pattern of soil stress at varying design parameters (diameters and densities) in the annular space. The creation of FEM was based on the results from laboratory tests completed in Chapter 4. A total of 42 FE models were built to analyze soil stress patterns relative to pipeline behavior underneath rivers.

5.2 Finite Element Method (FEM)

The finite element method (FEM) concept began as an aircraft structural analysis. The FEM is a simple technique to produce an approximate solution related to diverse engineering problems by calculating differential equations (Pepper and Heinrich 2006). Basically, a complicated boundary that is composed of a continuum is simply divided into geometric shapes. These are called finite elements. These elements are expressed as material properties, governed by constraints and loadings given by unknown values. Calculation of these elements is completed by differential equations to show the approximate behavior of the continuum. The FEM has been utilized for all kinds of analyses of structural mechanics, which analyze deformation and stress about the dynamics of structures. The FEM is broadening the range of application from deformation and stress analysis to field analysis, such as heat flux, fluid flow, magnetic flux, and seepage (Chandrupatla and Belegundu 2002). As the FEM technique is developed more and more, the application of FEM is being extended to include adaptive structures, automotive crash simulations, computational biomechanics, computational probabilistic mechanics, simulation of advanced engineering materials, material forming processes, computational fluid dynamics, and simulation of pollutant transport in geomaterials (Kaliankin 2002). The FEM is widely used in engineering when the deformation of complex isolated objects is modeled. In geological areas, FEM produces better results compared to other numerical methods (i.e. finite difference, finite volume method, etc) because it particularly concentrates on material interfaces for accurate outcomes to discover the relation of soil particles, while in geoscience the accuracy of FEM generally depends on personal preferences, experience and background.

FEM is also good for simulating the behavior of structures built in soil, rock or other materials that may experience plastic flow when their yield limits are reached. Every material property in FEM is described by an element. These elements create a mesh or grid modified by the user who tries to make the shape of the object to be modeled. The user decides the applied forces or boundary restraints for each element or zone. This element or zone behavior is based on a prescribed linear or nonlinear stress-strain principle. The shape of the grid can be changed in terms of the material yield. It will easily describe the plastic collapse and flow very accurately. FEM cuts the shape of a structure into elements and reconnects the elements at nodes, which play the role of pins holding the elements together. FEM has the advantage of handling very complex geometries easily.

In this research, ABAQUS 6.10 was the software used for numerical analysis and three-dimensional (3D) FEM was created to analyze pipeline behavior. This chapter describes in detail how to create mesh modeling (i.e. boundary conditions, applying loads, modeling, and analysis procedures) using ABAQUS 6.10 and the mechanical behavior theories that were applied for this model. In order to analyze the results efficiently, this chapter uses various analytical methods for analyzing FEM results. The major role of the annular space associated with pipe behavior in saturated silty soils is discussed.

5.3 Previous Research

FEM has been conducted in structural analysis and expanded in diverse areas for academic research. Previous FEM research that examined the relationship between soil and pipeline is summarized in this section.

In the 1990s, numerous FEM research was conducted by examining whether or not the experimental results were reasonable. Moore (1995) used three-dimensional FEM for studying the stress analysis in buried polyethylene (PE) pipes. A 3D FEM was proven as an efficient method to estimate several kinds of stresses including radial, circumferential, and axial normal stress He found maximum tensile axial stresses along with liners occurred at the spring line and that the depth of cover becomes deeper or backfill becomes looser when local axial tension is increased.

FEM was also popular for studying the deformation of a buried pipeline affected by diverse loads. Moore and Hu (1996) utilized a linear visco-elastic finite element analysis for understanding the deflection of high density polyethylene (HDPE). They created two different rheological models to study HDPE deformation using FEM and discovered the specific ranges in vertical pipe deflection rates using two deflection relaxation (5% and 10%) models they created.

Zhang and Moore (1997) also proved the superiority and reliability of time dependent FEM outcomes in analyzing HDPE components under several loading situations (i.e. as comparing the laboratory results). Also this research proved that the visco-plastic model was good to predict the behavior of HDPE material behavior.

Brachman et al (2000) revealed the best condition for a laboratory test evaluating the structural analysis of small-diameter product pipes in soils. Due to surface or boundary friction, accurate results were not acquired by laboratory tests for a buried pipeline. Unlike large-scale testing for pipeline experiments, this boundary friction in small-diameter pipes led to different results for stress or strain, which is calculated by FEM. FEM was utilized to understand how boundary roughness affects the response between the soil and a pipe in the laboratory test. They found this side friction effectively reduced 17% of the maximum vertical stresses in smooth boundaries.

Dhar and Moore (2000a) conducted linear and non-linear analyses for buried HDPE pipes using FEM and compared the results to laboratory ones. Three types of mechanical behavior (i.e. linear elastic, linear visco-elastic, and non-linear visco-plastic) were chosen and analyzed in order to determine which type would be the best for factorial studies. They simulated and analyzed conclusions based on HDPE deflection responses through laboratory and FEM results. The viscoelastic model was determined as the best mechanical behavior for HDPE pipe based on strain limit (0.5%). If an HDPE model is over this value, this research then asserted that non-linear plastic behavior is dominated in this HDPE.

Finite element analysis has also been utilized for understanding borehole stability relative to hydraulic fracture in the annular space of HDD installation. Kennedy et al (2004) performed numerical calculation to analyze the hydraulic fracturing of the soil above the crown of the cavity during HDD installation. They investigated the outcomes of elastic plate theory while calculating the mud pressures that cause tensile fracture. At 2 and 5 m of the depth of cover, they conducted a factorial study in saturated clayey soil. The results (tangential crown stress) simulated by FEM were compared to those calculated by the elastic plate theory developed by Obert and Duval (1967). Both FEM and Elastic Plate Theory (EPT) resulted in the same trend. While they did not matched in the plastic state, both results helped develop a better method for calculating drilling fluid pressure.

Wang and Sterling (2004) examined the stability of the annular space in loose sand using numerical simulation, FEM software, and ADINA. The main focus of their analysis was the stability of borehole wall depending on the existence of a filter cake. They also studied shear failure around the annular space by calculating plastic yield. They discovered shear failure occurs around the annular space prior to mud loss. A filter cake was found to reduce the excess pore pressure generated in the bore wall, and that without a filter cake, static liquefaction around a buried pipe may occur due to high permeability.

Kennedy et al (2006) continually studied the tensile hoop stress in sand during HDD installation using FE models. This research considered the annulus of a filter cake in sand material when building modeling mesh. They examined the soil response against the variation of mud pressure and considered shear failure in the sand cohesive filter cake area. They used a typical set of soil parameters for the sand and filter cake, then applied them to FEM and examined how the variation of mud pressures and filter cake thickness affect tensile fracture in the soil. Finally, they decided that the influential factors for mud loss during installation are the initial cohesion (*c*) and coefficient of lateral earth pressure (*K*).

Xia and Moore (2006) investigated the values of drilling mud pressure that causes failure of the adjacent soils to the river bed. Two ground failure theories (tensile fracture and blowout) suggested by Kennedy et al (2006) were examined by using numerical method (FEM). Considering the influence of both *c* and *K* at maximum pressure, they designed the suitable mud pressure for both hydro fracture and blowout that lead to losing soil intrinsic confinement in the extending plastic zone up to the ground surface. At last, they determined tensile fracture theory could be the most applicable mode for preventing mud loss when surrounding soils were normally consolidated. Alternatively, blowout theory was the suitable mode when soils were heavily over-consolidated.

Most FEM studies have been primarily conducted the stability of buried pipelines or the relationship between pipe and soil. For HDD the stability of the annular space (i.e. Hydro-Fracture) was the major topic for FEM. Determining suitable mud pressure to build steady annular space is absolutely critical for preventing hydro-fracture status, which causes the collapse of the borehole wall. The numerical analysis in previous research mostly considered borehole stability during HDD installation, while this pipe floatation research was assumed one month or more after installation. This research examined the soil stress pattern in soil overburden above an annular space and a product pipe; these results were associated with stable soil overburden relative to buoyancy effect. This research focused on the comparison of the soil stress pattern between OT and HDD methods when pipelines were installed at the same critical depths of cover found in the previous pipe floatation experiment. Furthermore the major role of the annular space associated with pipe behavior in saturated silty soils was discussed. ABAQUS 6.10 was the software used for numerical analysis and three-dimensional (3D) FEM was created to analyze pipeline behavior.

5.4 FE Modeling for Buried Pipeline in Saturated Silty Soils

The FE models were built based on what laboratory tests previously found. The most important findings in the experiment were the critical depth of cover, which is a minimum depth to diameter (H/D) ratios that prevent pipe buoyancy. The research found 6 critical H/D ratios, and FE models were created using these ratios for each construction method. Each construction technique had six 3 Dimensional-FE models, and each result from the two construction methods was compared to understand the patterns of soil stress around a buried pipeline. When modeling meshes, both soils and annular space were made of a *solid* shape that is one of the mesh shapes in ABAQUS, but a pipeline that has small thicknesses was made of a *shell* shape. Figure 5.1 below was shown in one of the 3D-FE model meshes created in both OT and HDD.

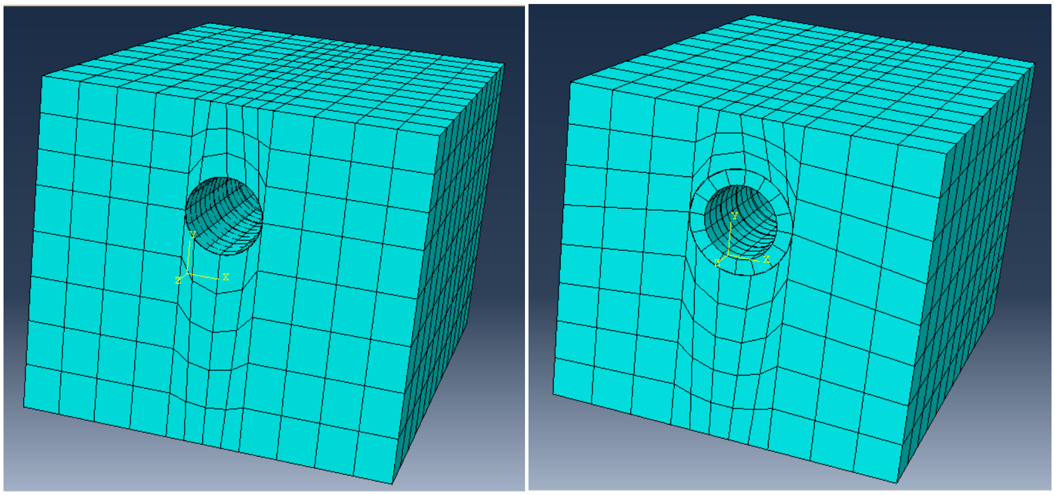

Figure 5.1 FE Modeling for OT (left) and HDD (right)

The number of elements in FE models was different depending on product diameter, annular space size, and depth of cover. The total element number approximately ranged from 1500 to 3000. In order to create suitable and organized meshes, the interval used between elements was 0.05m, which was assigned as the global size. The mesh shape used in the whole models was hexagon. Soil columns over and under the buried pipe, shown in Figure 5.1, were designed separately to make denser mesh. A little finer mesh in this section could lead better outcomes. Figure 5.2 below was FE models created for pipeline and annular space.

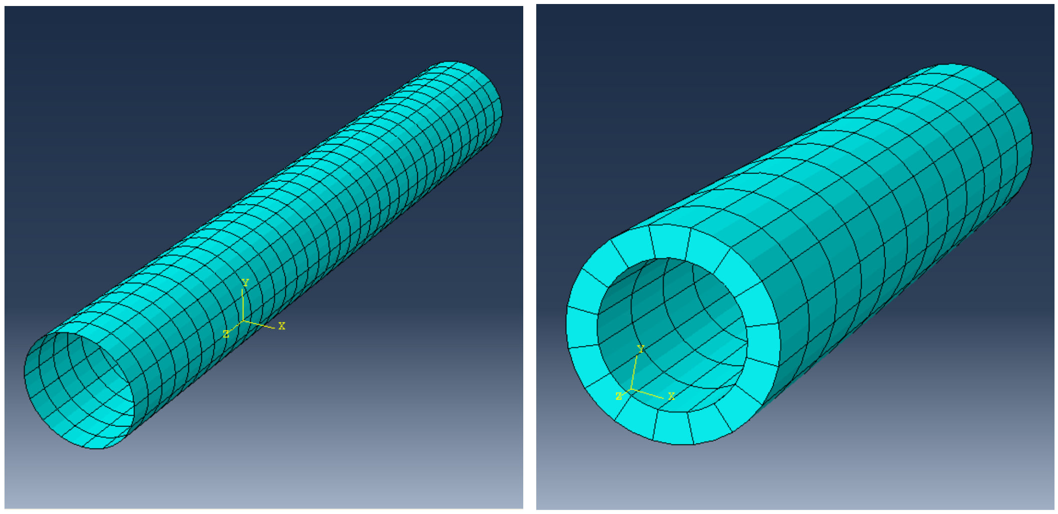

Figure 5.2 FE Modeling for Pipeline (left) and Annular Space (right)

All contacting surfaces between materials were tied so that each FE model could influence or be influenced by each other. The stress analysis was conducted and concluded based on these FE models.

5.4.1 FE model Dimensions and Applying Loads

The scales or dimensions of soil, pipeline, and annular space in FE models were referred to by previous laboratory data described in Chapter 3. The dimensions for each material were specified in the next section, “Model Parameters.” The total dimensions in FE models are shown in Figure 5.3 below.

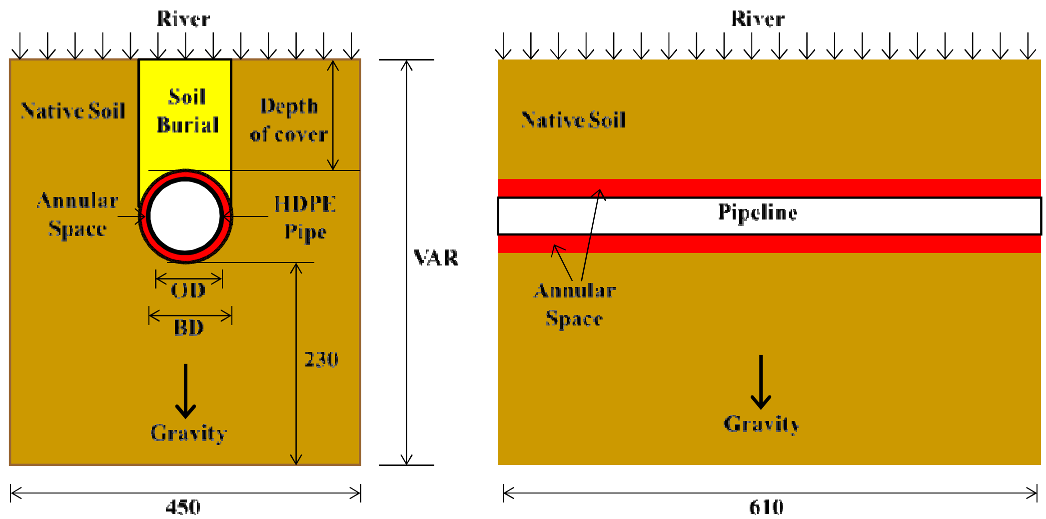

Figure 5.3 Total Dimensions in FE model and Applying Loads

The width and length of the total mesh were 450mm and 610mm respectively. The total height varied depending on the depths of cover, product diameter sizes, and the diameters of the annular space. The distance from the bottom of the pipe to soils was 230mm, which was fixed. Both gravity and river load were applied to FE models. Those loads are considered as the static load. In this model there were no external loads acting on soils and pipelines. The gravity was applied to this model so the weight of soil, pipe, and annular space, which were called “the dead load,” were computed by inputting the negative gravity (-9.81N) that represents force direction. The second load applied to the model was river load. This FE model was simulated in pipeline river crossings. The weight of a river must be considered for describing a river crossing in the FE model. Also, pipeline length is dependent on the width of a river. Only the width in this FE model was a constant value, which was 450mm. As a uniform load (4.412 kpa), the weight of a river was applied to the surface of soil mesh shown in Figure 5.3.

5.4.2 Model Parameters

This section describes the assigned parameters associated with previous laboratory tests in order to build FE models. In the experiment, sand was used as the base of the manufactured tank, but the FE models did not consider sand base that may not affect the final results of FE models. In other words, the existence of sand material was negligible for the results of FE models. Also, fast numerical analysis and modeling simplicity were why the sand base was excluded from the FEM modeling. Only upper soil cover was considered in a modeling mesh so as to focus the pipe buoyancy situation. Understanding the effects of soil overburden could be beneficial to examining the behavior of a buried pipeline. Each property (i.e. soil, pipe, and depth of cover) used in the laboratory test was utilized for building FE models. Besides the properties stated previously, other properties (i.e. mechanical behavior) were required to create FE models. The modeling meshes made for soils and buried pipelines were analyzed assuming they behave elasto-plastically. From the standpoint of soil mechanics, the stress and strain of soil do not exactly behave linearly as elastic material (Lee 2010). So, the most general and suitable theory in this situation would be Mohr-Coulomb theory, which has produced reasonable outcomes in actual soil behavior. Mohr-Coulomb theory could guide the best solution in order to understand the interaction between soil and buried pipeline. The material properties used for an HDPE pipe were reflected in using elastic and plastic properties referred to by PPI (2006). The soil properties used in this analysis were also found in previous literature.

5.4.2.1 Riverbed Soil

The soil properties employed in the laboratory were used as an input number to reflect riverbed soil status. The soil density found in several laboratory tests was 1.197 ton/m^3; the friction angle was 23.1degrees; the cohesion was 10kpa. In Chapter 3, it was determined that this soil composition is very appropriately represents alluvial particles typical of riverbed soils. These properties were somewhat small compared to generally riverbed properties (silt conditions). However, a modeling must be consistent with the soil composition acquired from previous research in order to examine and conclude that buried pipeline behavior is consistent with the previous work. Unlike Figure 2.8 or 2.9, sand and silt base is not considered adequate for building FE models because we diminished mesh errors from occurring by mesh complexity and expected that these factors would not affect the final results that were considered only by soil overburden. This soil data is a very critical element to determine the final results of buried pipeline behavior in FE modeling. For OT installation, the FE model was not considered backfill property for soil cover, meaning the soil cover in OT was designed the same as the one in HDD. The soil burial used in laboratory tests in both OT and HDD was the same, composed of particle settlement after dumping soil-water mixture.

5.4.2.2 High Density Polyethylene Pipe

In this modeling, 50, 75, and 100 mm diameter of high density polyethylene (HDPE) pipe were utilized to build FE models, which was the same as the previous laboratory test. The length of the HDPE pipe utilized was 61mm, which is the size used in both the experiment and buoyancy theory. The plastic and elastic properties for an HDPE pipe were obtained in PPI (2006). The density of HDPE pipe was 0.959 ton/m^3. In elastic properties, Young's modulus was 192,900 kpa, which was the mean value used during short-term period of HDPE usage. Both Poisson ratio and yield stress were 0.45 and 31,000 kpa respectively.

5.4.2.3 Drilling Mud

The simulation method of the annular space was developed to create a light concrete mold to cover the product pipeline. Since the annular space was regarded as a cohesive, consolidated, and compressive area, we assumed that the annular space could move with a product pipe as a composite material. The borehole diameters (BD: diameter of the annular space) were determined to be 1.5 times the outer diameter (OD) of the product pipeline, which follows the rule of thumb size, which is referred to as the best practice of HDD installation (Bennett and Ariaratnam 2008). It was difficult to find accurate mold sizes for the expected annular space, so the light concrete molds utilized did not exactly fit the borehole. In FE models, we utilized the same data found in the laboratory tests, such as borehole diameter (BD) and the density of drilling fluids. Both elastic parameters (i.e. density: 1.35t/m^3, Young's modulus: 1.35ton/m^3 and Poisson ratio: 0.5) and plastic parameters (cohesive strength: 15 kpa, friction angle: 0, and dilation angle: 5°) were utilized to mirror the original clay properties (Bowles 1996; Das 2006).

In summary, all data used to create these FE models was taken from previous research, to create a connection to the previous experiments. The dilation angle of 2°, which was referred from the previous literature, was utilized (Lee 2010). SIMULIA (2009) described the range from 1° to 5° in dilation angle did not affect the result of FEM model. Table 5.1 below was shown in the summary of input data used in the FE model.

Table 5.1 Material Properties in FE model (Das, 2006; Bowles, 1996)

Property Types		Soil (Silt)	HDPE pipe	Drilling Mud
Density (ton/m3)		1.197	0.959	1.35
Elastic Property	Young's Modulus (kpa)	20,000	192,900	30,000
	Poisson Ratio(μ)	0.35	0.45	0.5
Plastic Property	Yield Stress (kpa)	-	31,000	-
	Cohesive Strength (kpa)	10	-	15
	Friction Angle (°)	23.1	-	0
	Dilation Angle (°)	2	-	2

5.4.3 Boundary Conditions

The size in FE models was created by following the original dimensions designed in the laboratory tests. In order to simulate a similar situation in the pipe floatation experiment, a sufficient distance from the borehole was critical to preserve the stress diagrams in post installation (Kennedy et al 2006). It is very important to reduce the effects of model boundaries on the analysis results because in the previous literature of FE modeling several cases underwent huge boundary effects that may have disturbed producing accurate results in the shear failure zone (Wang and Sterling 2004). In this FE modeling, both mesh boundaries for the annular space and pipe were almost located at the center of the whole mesh modeling. Along the vertical borehole axis, the FE model was symmetrically designed to reduce errors from occurring on the analysis results. The mesh modeling around the upper soil burial over a buried pipe was made finer in order to obtain more reliable results. To minimize the boundary effect, a sufficient distance from the center mesh boundary must be maintained because distance helps keep geostatic stress conditions around buried pipeline consistently (Kennedy et al 2006). The FEM study was only interested in the variation of upper soil burial over the pipeline, and when analyzing the stress analysis in this area, stress pattern in the middle elements cutting meshes at each end of FE models was examined to ensure that the final result would be somewhat free from boundary friction. Figure 5.4 below shows the front view of boundary condition in 3D FE model.

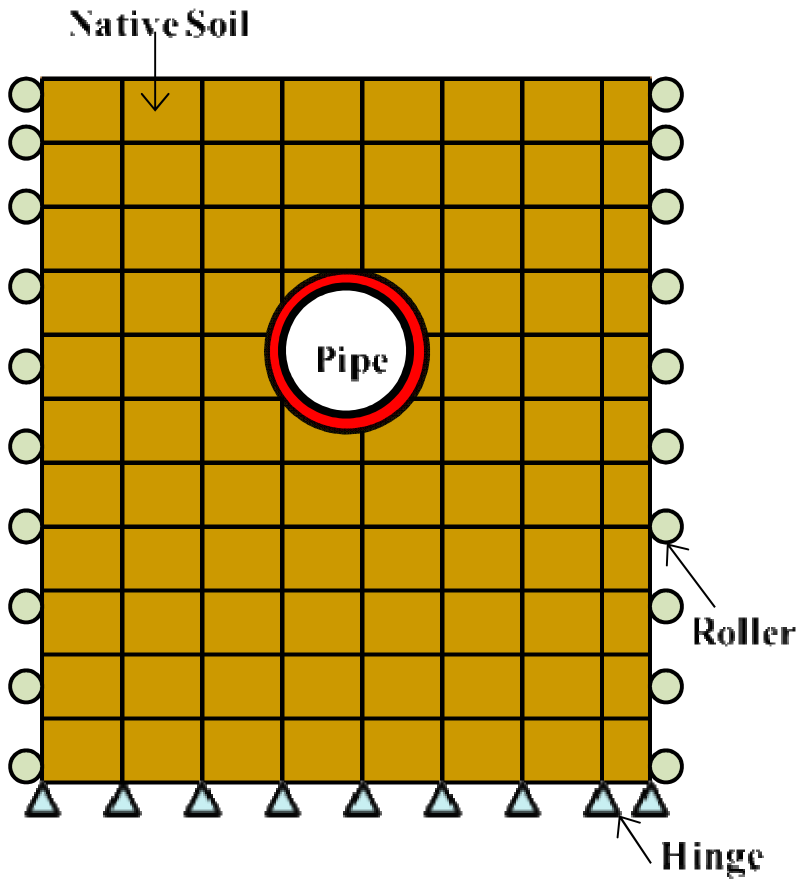

Figure 5.4 Boundary Conditions in the Front View

Figure 5.4 shows a restrained bottom side in this FE model. Only four surface sides in the FE model were allowed to move vertically. Horizontal movement was restrained in the whole model. In this research, the vertical movement of soil overburden is the most important part that must be considered. Figure 5.5 showed the side view of the boundary condition assumed in this 3D FE model.

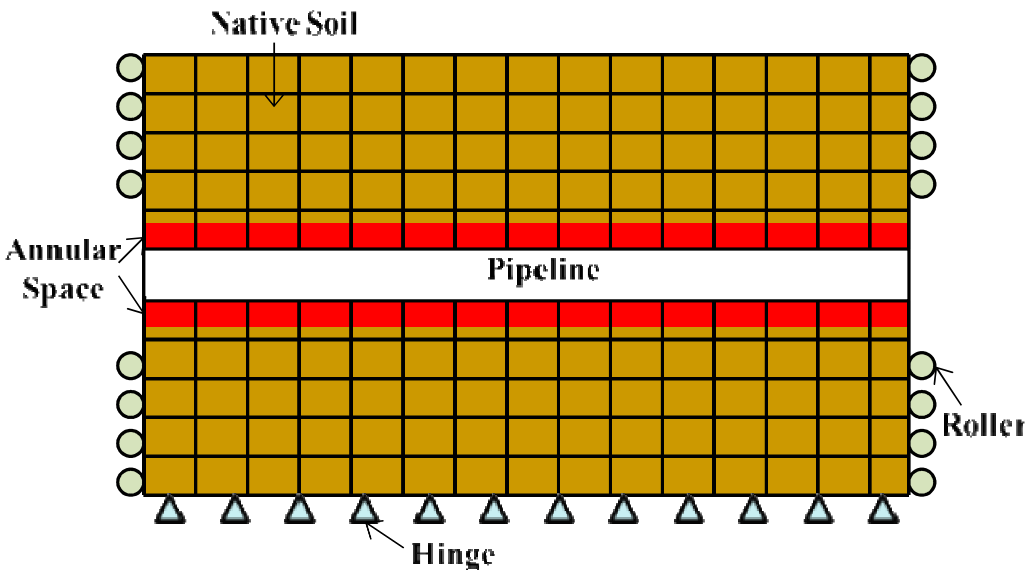

Figure 5.5 Boundary Conditions in the Side View

As shown above, the front and bottom surface were restrained by hinges. This FE model followed the dimensions for what the experiment simulated. The front sides were allowed to move vertically. The boundary condition at the front sides was activated by rollers. Figure 5.6 presents boundary conditions for the end of pipeline and annular space in FE models.

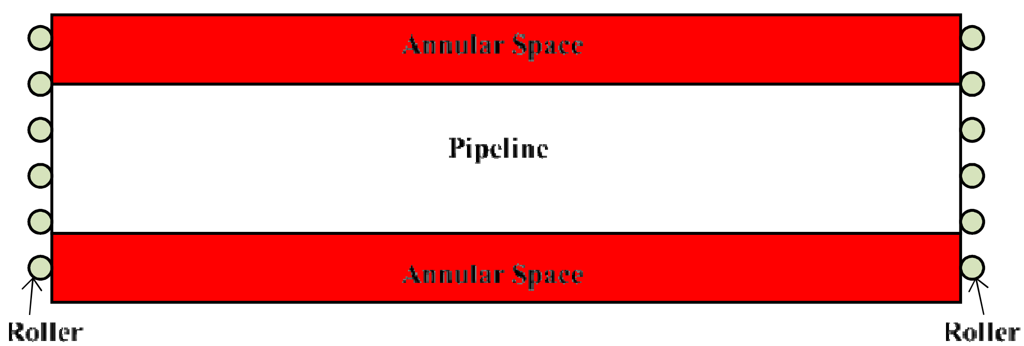

Figure 5.6 Boundary Conditions in Pipe & Annular Space

The pipeline was able to move vertically by rollers. The annular space was also assumed to be the same as boundary condition of pipeline. All boundary conditions were chosen to study the vertical movement while the horizontal direction was restrained.

5.5 Analytical Methodologies for FEM

To determine the behavior of a buried pipeline, this section mentioned two analytical methods that we performed. The two methods were as follows:

1) Check maximum soil stress and its pattern around a buried pipeline in both OT and HDD: stress analysis is very critical for engineers to decide design parameters (i.e. depth of cover, annular space properties; density, diameter, and pressure). We only focused on maximum soil stress in the top soil over the pipeline. Investigate the stress pattern in soil cover boundary in both OT and HDD methods that were simulated at the same critical H/D ratios obtained from the laboratory tests. Note the maximum soil stresses occurring in both OT and HDD methods, and compare those. If there is a difference between those methods, analyze why it happens.
2) Change values of design parameters (i.e. diameters, soil or mud properties, and depth of cover) and substantiate the relationship between them and pipe behavior through finite element analysis. Different soil properties (i.e. density, Poisson ratio, Young's modulus, and plastic properties) can vary as well. This change can inform the relationship of pipeline behavior regarding diverse soil status. Another

way is to vary the depth of cover designed. As the depth of cover increases, observing changes in soil stress is a good approach to understand the relationship between soil stress and depth of cover. In HDD installation, the properties (i.e. diameter and density of drilling fluid) of the annular space can vary. As varying these properties, we can reveal how the annular space influences soil stress.

5.6 Results

The FEM results were comprised of three stress analyses: 1) comparing soil stress patterns in soil overburden above the pipeline installed by OT and HDD at the same H/D ratios, 2) analyzing soil stress patterns at varying diameters of annular space, and 3) analyzing soil stress patterns at varying densities of annular space. The first results showed maximum tensile or compressive stresses occurring in soil overburden when each pipeline installed by HDD and OT methods was buried at the same critical H/D ratio. We also found the position where maximum stress occurred in saturated soil burial. The second and third results presented how the annular space influences soil stress above a buried pipe. In order to understand the role of the annular space, we varied the critical design parameters (i.e. density and diameter) in the annular space. The density of drilling fluid and borehole diameter are important parameters for determining accurate pipeline installation design of HDD method. Thus, these will give a critical idea about the role of the annular space related to pipe behavior in saturated silty soils.

The FE models were analyzed by the static load status, which was only applied to the gravity typical of static loads. ABAQUS 6.10 was suitable software to study the general static status. Stress considered in this analysis was Von Misses Equivalent Stress that has been utilized for plastic behavior between pipeline and soil (Lee 2010). This theory follows the yield criterion related to the total strain energy theory. When modeling, these FE models were assumed to follow the Mohr-Coulomb theory, which is generally used in plastic yield theory. The material properties used in this model were categorized as mechanical behavior in materials (i.e. soil and pipe) was assumed elasto-plastic. Additionally, the annular space in this model was assumed to be well built in a month, meaning the annular space has a good filter cake, which effectively protects the borehole wall. In one month, the boundary in the annular space could act as a single composite, which the previous experiment assumed. The FE model only deliberated three-dimensional (3D) stress around soil overburden. Soil displacements occurring at nodes were very small during the initial and static load step. Robert and Britto (2008) found the displacement during the geostatic step (static load) was acquired by considering the gap between initial and calculated stresses by ABAQUS. This caused displacements that were too small due to the little gap between the initial and calculated stresses. Thus, the displacement in this FE model was negligible. In addition, element types in the 3D-FE model were second-order element (or Quadratic), which has 20 nodes in one element. Rao (1999) proved the results obtained by higher order meshed model are more precise. Hence, this FE model implemented second-order interpolation and quadratic geometry for each element.

5.6.1 FEM Stress Analysis: OT vs HDD

This section describes the stress pattern in the soil overburden after OT and HDD installations at six critical depths of cover discovered in the laboratory tests. The subsequent sub-sections summarized and compared soil stresses affected by both construction methods at each diameter. The final stresses were presented in Table 5.2 below.

Table 5.2 Max-Stresses at the Critical H/D Ratios in OT and HDD

Product Diameter (mm)		50		75		100	
Critical H/D Ratios		1.51	0.39	1.30	0.91	1.73	0.82
Max-Stress (kpa)	OT	5.044	5.074	6.183	6.023	7.310	6.205
	HDD	3.902	3.722	4.504	4.254	4.795	3.952

5.6.1.1 50 mm

In 50mm HDPE pipe, both 1.51 and 0.39 in critical H/D ratios were acquired in the laboratory tests. 1.51 was the minimum value for soil cover height to prevent pipe buoyancy for OT installation, and 0.39 was the value for HDD installation. Two FE models for each H/D ratio were created. The maximum stress in soil burial over the pipe was 5.044 kpa when the pipe installed by OT was buried at 1.51 of the H/D ratio. HDD was 3.902 kpa. Appendix C was shown in this stress pattern at about 50 mm installations. The maximum stress in both OT and HDD installation occurred at the direction of 11 and 1 o’clock in the contacting surface between soil and pipeline. While the maximum stress trend in both OT and HDD was almost analogous, the trend in stress contour was different. The minimum stress occurring in the OT FE model was spread from the contacting surface at the crown of the pipeline to the top surface of the FE model (see Appendix C). Contrary to this, the minimum stress occurring in the HDD FE model was exited around the surface sides of the FE model. Finally, the stress was decreased as the depth of cover was increased even when variation was minimal. However, the relationship between depth of cover and soil stress was not consistent compared to the results in both 75 and 100 mm diameters in the next section.

5.6.1.2 75 mm

In OT installation, the maximum stresses in the 75 mm product diameter were 6.183 and 6.023 kpa at 1.30 and 0.91 of the critical H/D ratio respectively. In HDD installation the maximum stresses were 4.504 and 4.254 kpa. Generally OT installation caused higher stresses in soil burial than HDD. Additionally, the location of the maximum stress occurring in the 75mm FE model was same as the one with 50mm. The whole stress pattern in the 75 FE model was also similar to that of the 50 FE model (see Appendix C). However, as the depth of cover increased, soil stress also increased, unlike the 50 mm diameter HDPE pipe.

5.6.1.3 100 mm

The same phase in 100 mm diameter of FE model was also found as shown in Table 5.2. OT installation led to higher stresses in soil burial than HDD installation, which was same as the other product diameters (see Appendix C). In addition, the aspect of stress contour was very similar to other diameters. Furthermore, another finding in both 75 and 100 mm in OT installation was that the stress increased as the burial increased.

5.6.1.4 Summary and Analysis

Twelve FE models were created in order to study the phase at varying stresses and to contrast the total vertical stresses between OT and HDD FE models when buried at the same depth of soil burial. This comparison provided three findings for HDD and OT installations.

Firstly, the soil overburden area in OT installation had higher stresses than in HDD installation. Every 12 FE model that proved OT installation brought about higher soil stresses compared to HDD FE models in same situation. This means that soil stress in HDD is partially transferred into the annular space. In other words, the annular space may observe partial stresses that were supposed to be in soil overburden. This effect could be described by the arching effect. Compared to material properties, silty soils used in the research were relatively softer and weaker than the stiffness of annular space (general clay soil properties) used in FE models. Hence, the annular space was capable of partial soil stresses in terms of the arching effect that transfers compressive stresses, which depends on the differences between their material stiffness.

Secondly, as the depth of cover in 50mm HDPE pipe was increased, stress was decreased in the OT installation based on Table 5.2. Unlike the 50mm diameter pipe, the 75 and 100 mm pipe had a different trend for the relationship between depth of cover and stress. Datta (1999) found this phase could be different depending on soil materials and pipeline properties (i.e. diameter and material). Thus, the two different trends between depth of cover and stress could vary depending on the diverse materials and parameters; these need to be examined in detail through finite element analysis.

Lastly, the locations where the maximum and minimum stresses occurred in soil overburden were at the direction of 11 and 1 o'clock on the pipe in the soil burial. This trend was same in both OT and HDD FE models. The maximum stresses axially occurred at these directions along the pipeline. The minimum stress in OT installation occurred at the crown of the contacting surface between the pipeline and the soil while the minimum stress in HDD installation occurred at the surface soil.

Summarizing the pattern of soil vertical stress, the amounts of maximum soil stresses between HDD and OT installations were fairly different while the pattern of soil stress contour was very analogous. Soil stress occurring in OT installation was approximately 1.4 times greater than that in HDD installation due to the existence of annular space.

5.6.2 FEM Stress Analysis: Changing BD in HDD

Two different trials were conducted in order to examine the role of annular space. The first trial was to change the diameters of the annular space, which was called "borehole diameter (BD)" shortly. The ratios between BD and OD (outer diameter of a product pipe) in the previous laboratory tests ranged from 1.43 to 1.63. For this FEM trial, the data obtained in the previous 75mm HDPE pipe test was utilized as a standard for FE modeling. 1.30 of critical H/D ratio was utilized as a standard depth of cover. BD in the 75mm HDPE pipe was 1.43 times OD in the previous pipe floatation research. As 1.5 times OD was the research standard for BD, BD was scaled from 1.33 to 1.73. A total of 15 FE models were built for 50, 75, and 100mm HDPE pipe. Figure 5.7 was shown in the phase of soil stress at varying BD scales.

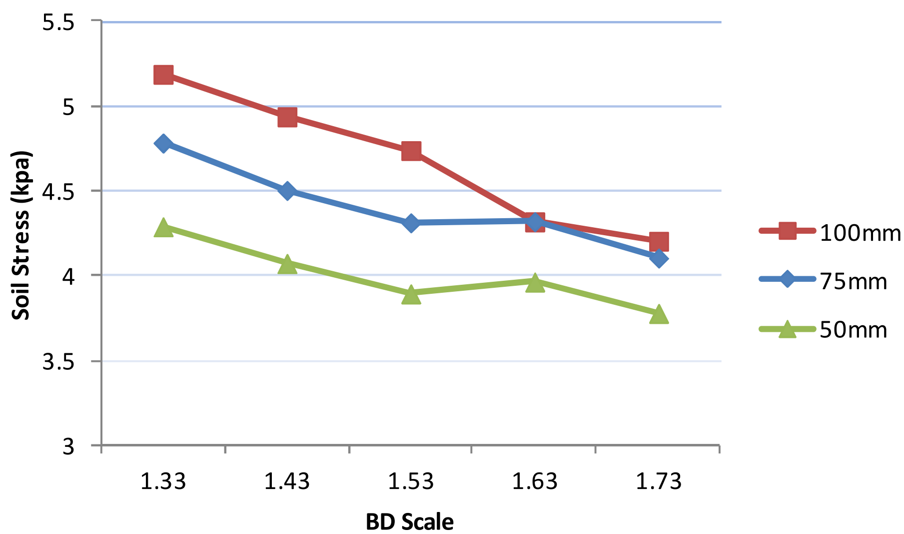

Figure 5.7 Soil Stress Patterns at Varying BD Scales

As a BD scale was enlarged the total stress in soil burial over the pipe was mostly decreased in all three diameters. If the portion of the annular space was increased, partial soil stress could be moved into the annular space due to the arching effect. The contour pattern in soil stress was nearly same as the results in the previous section. The maximum stress flew axially at the direction of 11 and 1 o’ clock. The strong relation between soil and annular space was concentrated at the contacting surface. Based on these results, it could be concluded that the size of the annular space also helps control the total soil stress in soil burial, which is located over the buried pipeline. The next section was another test in order to find out the role of annular space at varying its density. The detailed FE models that were created are shown in Appendix D, E, and F.

5.6.3 FEM Stress Analysis: Changing Densities of Annular Space

In the previous section, the study revealed that the size of the annular space is a critical element that is able to affect soil stress variation in saturated silty soils. In this trial, changing densities in the annular space were also a good indication in examination of the relationship between soil stress analysis and annular space parameters. Figure 5.8 below was the final result of the changing densities of annular space.

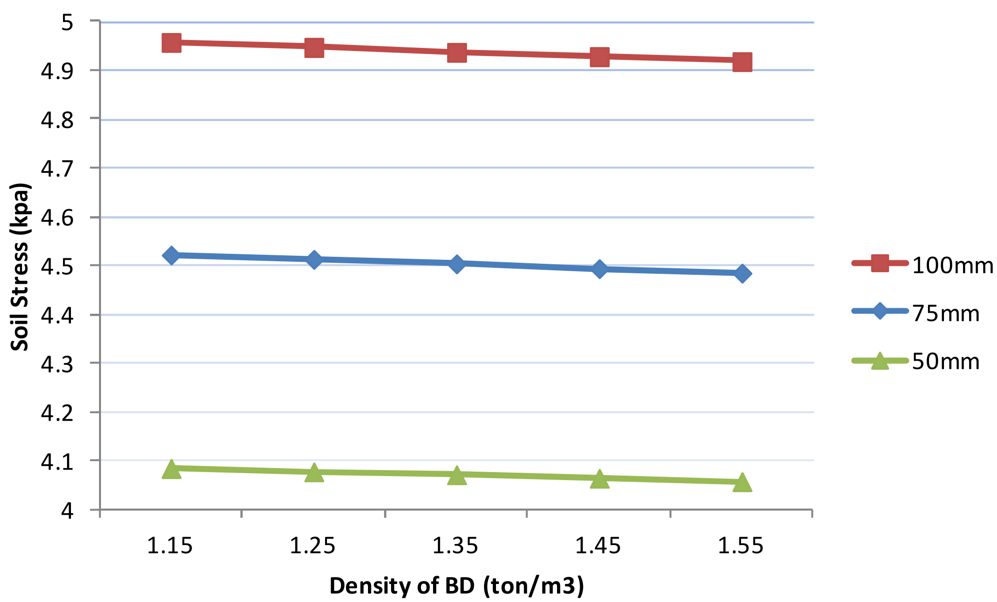

Figure 5.8 Soil Stress Patterns at Varying Densities of BD

This graph presents the trend of varying stresses associated with BD densities, which almost has a linear behavior. While the increasing density in the annular space reduced soil stress, this variation was too small to impact the soil stress pattern. In Chapter 4, the research found that the density of the annular space affected the behavior of buried pipeline regarding pipe buoyancy. An increased density caused an increased uplift resistance force that prevents pipe buoyancy as previous literature mentioned in Chapter 2. However, in this FEM results, the density of drilling fluid did not vastly affect the amount of soil stress compared to the first trial (changing diameters of the annular space). The detail descriptions for this stress analysis were shown in Appendix G, H, and I.

5.7 Summary in Numerical Analysis

The research revealed the stress pattern in soil cover above the pipeline installed by open trench (OT) and horizontal directional drilling (HDD) at six critical depths of cover. Also, by changing parameters (i.e. diameter and density) in the annular space, the relationship between annular space and soil stress was approached. Multiple FE models were created to perform two objectives mentioned in this chapter.

The pattern of soil stress diffusion in both HDD and OT FE models was very analogous. The greatest soil stress occurred not at the crown of the pipe but at the direction of 11 and 1 o'clock on the pipe. Maximum soil stress was axially flown along the pipeline. Maximum soil stress patterns in both HDD and OT were nearly identical, while they had a different pattern of minimum soil stresses. A more interesting part was the difference of maximum soil stresses between OT and HDD methods. HDD installation brought about less soil stress over the buried pipes compared to OT installation. Soil stress in post-OT installation was approximately 1.4 times greater than HDD installation. This was because the annular space partially supported the soil stress that occurred in the soil burial due to the arching effect. OT method caused greater soil stress in soil overburden above a buried pipeline. These results could not directly conclude that HDD installation is a better method at the same depth of cover in the standpoint of pipeline safety. However, these results showed that at the same depths of cover HDD could be a better construction method when riverbed status is questionable for plastic soil failure that happens when total soil stress occurring in riverbeds is over the limit of the original yield soil stress. In this manner, resulting in less soil stress obviously means that HDD would be better suited to soil plastic failure.

In the previous laboratory results, the annular space was a very critical factor to control the pipeline behavior. To prove this fact using FEM, we varied the density and diameter of the annular space to understand how these parameters affect soil stress occurring in riverbeds. In the long run, these parameters significantly affected soil stress pattern in soil burial. An increase in borehole diameter (BD) and density led a decrease in the soil stress over the pipeline. Thus, the research concluded that the annular space could help manipulating the soil stress occurring in riverbed. These two kinds of FE models helped substantiate the critical roles of the annular space.

Chapter 6: CONCLUSIONS AND RECOMMENDATION

6.1 Conclusions

6.1.1 Research Summary

The main objective of this research is to reveal the buoyancy effect for buried pipelines installed by traditional open trench (OT) and horizontal directional drilling (HDD) methods in saturated silty soils typical of a riverbed. After obtaining the results of buoyant behavior, we compared OT and HDD installation methods to define the behavior of buried pipelines. Critical buoyancy factors were determined. This and other final factors that affect buried pipeline behavior were analyzed. To obtain the results of pipeline behavior, laboratory tests and theoretical methods were performed producing critical H/D ratios for both the OT and HDD methods. A real-scaled metal tank was manufactured and utilized for laboratory tests simulating the pipe buoyancy effect. The diameters of HDPE pipe used in the laboratory tests were 50, 75, and 100 mm. For the theoretical method, buoyancy theory was applied to calculate the critical depths of cover, which were comparable with experimental results. Finally the numerical analysis (FEM) was performed to understand the stress pattern of a soil burial boundary, which was created by utilizing the critical depths of cover (H/D ratios) obtained from the experiment.

6.1.2 Critical H/D Ratios: OT vs HDD

Among those buoyancy factors (i.e. pipe diameter and material, soil properties, depth of cover, saturation, etc), the depth of cover is obviously a critical factor for pipeline design and installation because the lack of depth of cover leads to pipe buoyancy post-installations. In this research, the depth of cover was used as the variable in factorial designs in order to contrast the results between OT and HDD methods. The summary of the results is as follow:

1) The critical depths of cover calculated in the theoretical method were a very consistent value (approximately 3.0 for OT and 1.5 for HDD) for each diameter.
2) There was discrepancy between the theoretical results and the experimental depth of cover ratios.
3) The depth of cover was affected by the standard dimension ratio (SDR) of each diameter.
4) The annular space led different results in the laboratory tests between OT and HDD methods; HDD required less critical H/D ratios than OT to prevent pipe buoyancy.
5) The density of annular space in HDD caused different buoyancy behavior from OT.
6) The ratio between borehole diameter (BD) and outer diameter (OD) was determined as a critical parameter for pipeline behavior.

The theoretical results of each diameter for both OT and HDD were similar. Based on these results, we found that the depth of cover was affected by the standard dimension ratio (SDR) of each diameter. The SDR is the ratio of thickness to pipe diameter. Thus, a high SDR means that the thickness of a pipe is relatively thin. In both the theoretical and experimental method, a 100 mm HDPE pipe with a SDR of 21 required a larger H/D ratio to prevent pipe floatation than a 50 and 75 mm HDPE pipe with a SDR of 17. The SDR result of a 100 mm HDPE pipe proved that a higher H/D ratio must be required for safe pipeline installation if a HDPE pipe with a higher SDR is designed. Thus, engineers must deliberately consider the SDR when considering the effect of pipe floatation in river crossings.

In HDD installation, the annular space significantly influenced pipeline behavior. Depending on the parameters of the annular space or drilling fluid, pipelines installed by HDD behaved differently in saturated silty soils compared to pipelines installed by OT. The only structural difference between OT and HDD is the existence of the annular space. This difference is a good indicator of the different outcomes between OT and HDD. Previous research discovered that the buoyancy effect in water body crossings relies on the density of soil overburden. Hence, this experiment focused on the density of surrounding soils or pipe materials. The annular space was simulated using a light concrete cast that was manufactured following the expected density (1.282 ton/m^3; see Chapter 3). The final light concrete density (1.345 ton/m^3) was discovered to be higher than what was expected and all three test pipe assemblies of concrete mold had nearly the same density. Therefore, the concrete mold density was considered as a fixed value when comparing final results.

The next influential factor was the ratio between outer diameter (OD: product diameter) and borehole diameter (BD). Basically, the borehole size simulated in the laboratory was 1.5 times the outer diameter of the pipe that is referred from Bennett and Ariaratnam’s book (HDD good practices guideline).

This borehole size has generally been utilized in actual construction practices for HDD installation design. Due to the difficulty of finding the same size of concrete mold for each BD, we utilized sizes that were similar but not exact, which is representative of actual construction practices. Conclusively, the ratio of BD to OD became the major variable to determine the critical H/D ratios for each diameter regardless of construction methods. Based on the laboratory results in Chapter 4, the final critical H/D ratios from the HDD test were not consistent. A 50 mm HDPE in an HDD test had the smallest H/D ratios among the three diameters due to a higher ratio (1.68) between BD and OD. On the contrary, a 75 mm HDPE, which had the smallest ratio (1.43) between BD to OD, resulted in the largest H/D ratio. A greater ratio between BD and OD requires a smaller critical H/D ratio. The ratio in the annular space installed in practical construction may be very influential for the security of a buried pipeline. Engineers must carefully determine a suitable ratio for the annular space if the pipe buoyancy situation of a project site is questionable. Therefore, it can be concluded that the existence of the annular space in the HDD method incurs a different pipe buoyancy phenomena than the traditional OT installation.

The results followed our hypothesis that pipes installed in saturated silty soils by HDD and OT methods of construction behave differently. The critical finding was that pipes installed by HDD require less depth of cover than similarly sized pipes installed by OT method. The main reason was due to differences in density between the soil covering and the drilling fluid. The drilling fluid had a greater density than the native saturated soils, so that in HDD installations the density of the drilling fluid results in a lower critical depth of cover. Subsequently, the unit weight of the drilling fluid utilized in a river crossing must be specifically designed to minimize the conditions that could lead to pipe buoyancy.

This research also found that there was discrepancy between the theoretical results and the experimental depth of cover ratios. The theoretical calculations were more conservative than in the cases examined in this research. This research did not consider soil friction created by shear strength factors. In previous research (White et al 2001), the uplift resistance force calculation in buoyancy theory considered soil friction factors; however, the traditional pipe buoyancy theory utilized in this research did not consider soil friction (PPI 2006). When considering soil friction in practical design and installation, it was very difficult to determine the appropriate shear failure theories because of different soil plastic failure assumptions. There are diverse shear failure theories applied to pipe buoyancy theory depending on geotechnical properties and failure assumptions. In detail, these soil failure theories required several soil properties (i.e. dilation angle) that were rarely obtained by geotechnical investigations in real life. Due to economic reasons, pipeline construction often overlooks the properties of riverbed soil, thereby increasing the chances of pipe floatation incidents. Thus, this conventional pipeline buoyancy equation must be supplemented by precise and reliable soil friction factors through continuing research.

Overall, two clear findings in the laboratory test can be summarized. Firstly, pipelines installed by HDD have more flexibility in determining the depth of cover for preparing for scour effect, which may trigger the loss of soil cover. This is because pipelines installed by OT need more depth of cover than similarly sized pipelines installed by HDD in order to prevent pipe buoyancy. Secondly, the existence of the annular space is very important for safe design and prevention of pipe buoyancy. Particularly, determining the density and ratio of BD to OD in the annular space was very important for pipe security in river-crossing projects. To fully understand the importance of the annular space, a detailed analysis of annular space influence on the behavior of a buried pipe was performed. Soil overburden above the pipe is a crucial boundary in considering pipe buoyancy. For this, numerical analysis was utilized to examine the pattern of soil stress occurring in soil overburden.

6.1.3 Soil Stress Analysis by FEM: HDD vs OT

In Chapter 5, we suggested two objectives for using finite element modeling (FEM) for the numerical analysis of pipeline behavior. The first objective was to compare maximum total soil stress occurring in the soil cover boundary for the two installation methods. Each installation method had six FE models to test the critical depths of cover found in previously laboratory tests. The six critical H/D ratios utilized in FE models were the sum of three H/D ratios each from horizontal directional drilling (HDD) and traditional open trench (OT) methods. Soil stress was considered only the section of soil burial directly over the pipeline because that section of soil is critical for considering pipe floatation behavior. The maximum soil stress occurred not at the crown of the pipeline but at the direction of 11 or 1 o' clock in the contacting surface between the soil and the pipeline.

A more interesting part was the difference in maximum soil stresses between OT and HDD methods. When the OT method is used for a river pipeline crossing, stress occurring in the soil cover is higher than when the HDD method is used. This could be because the annular space partially absorbed soil stress preventing it from occurring in the soil cover. Additionally, when examining the variation of soil stress in regards to varying depth of cover, the soil stress itself increased the most as the burial depth was increased except in the 50 mm HDPE pipe for the OT installation method. Overall, these results could not directly conclude that HDD installation is a better method using the same depth of cover from the standpoint of pipeline safety, because maximum soil stresses obtained from this FEM research were too small to regard these results as a critical situation. Nevertheless, less soil stress occurring in post-HDD installation could be a very attractive point for engineers and contractors to determine the suitable construction method for river crossing projects. When total soil stress occurring in the soil overburden is over the limit of soil yield stress, the soil is deformed. This soil deformation brings about unstable pipeline behavior. Hence, considering total soil stress occurring in the soil boundary is very important when engineers and contractors design and install new underground infrastructure. Conclusively, all of these results support the importance of understanding annular space in relation to pipeline behavior, which leads to the next step of FEM.

6.1.4 Soil Stress Analysis by FEM: Annular Space

The first objective of using FEM was to find out the soil stress distribution and maximum soil stress that occurs in the soil cover area when both horizontal directional drilling (HDD) and open trench (OT) methods are utilized for a pipeline crossing through a riverbed. To determine the role of the annular space in HDD, two parameters, density of the drilling fluid and diameter of the annular space, were varied to show how the annular space affects soil stress in post-HDD installation.

An increase in the density of the annular space, which is filled with drilling fluid, led to a very small decrease in soil stress in the soil cover zone. Although there were small variations in soil stress, the research determined that the density of the annular space did not vastly impact the variations of soil stress occurring in the soil overburden. On the contrary, varying the diameters of the annular space led to significant variations in maximum soil stress. Increasing BD shows decreased soil stress in the soil burial zone. Thus, the diameter of the annular space could impact on total soil stress occurring in the soil overburden.

6.2 Recommendation for Future Research

6.2.1 Pipe Floatation Research

Pipe buoyancy research was designed to simulate conditions present in the borehole a month or more after installation. This specifically referred to the cast in place annular space around the pipe that was utilized to simulate the density and volume of the space typical of HDD installations. This analysis did not consider the annular pressures experienced in the borehole during construction, or the depth of installation required to manage the borehole pressures developed during the installation process. With these limitations, this research may be better suited to examine pipe installations where the depth of cover changes due to river scour. It is possible to say that HDD would potentially be very beneficial for pipeline safety in different riverbed configurations, including scour depth.

The research could be expanded to include a more practical study of river-crossing pipelines. Firstly, the actual location of the pipeline inside the annular space should be considered in post-installation. In reality, the installed pipeline is generally not going to be positioned concentrically within the borehole. If the installed pipeline is upwardly located inside the annulus, it may lose upper drilling fluid, which leads to an unbalanced shape in the annular space. Losing some amount of drilling fluid over the pipeline could cause a pipe floatation accident due to a decrease in the uplift resistance force. Secondly, practical models must consider river flow characteristics (scour, flood, and tide). These characteristics significantly influence the behavior of a buried pipeline in saturated soils. Huge floods trigger pipeline erosion, exposal, and floatation at river crossings (Wang et al 2010). Moreover, scour caused by the action of a flow may be one of the causes of pipe exposure (Moncada-M and Aguirre-Pe 1999). Hence, if we deliberate these factors, then this research model will closely be an applicable and practical model for river crossings in practice. Thirdly, the procedure utilized provided a repeatable methodology that produced consistent results from which the behavior could be observed, while the soils utilized in this examination have a lower unit weight than those that might be found in natural deposits. Future research should include scaling the soils to create more project applicable conditions. Fourthly, this research also found that there was discrepancy between the theoretical results and experimental depth of cover ratios. The theoretical calculations were more conservative than the experimental results in the cases examined in this research. This research did not consider soil friction created by shear strength factors. In previous research (White et al 2001), the uplift resistance force calculation in buoyancy theory considered soil friction factors; however, the traditional pipe buoyancy theory utilized in this research did not consider soil friction (PPI 2006). Thus, the next process would be to design an accurate and desirable buoyancy theory model, which considers soil friction factors through examinations of previous research.

6.2.2 FEM Research

Future FEM research could go in several directions: 1) changing parameters and modeling shape, 2) determining the relationship between borehole pressure and pipe buoyancy, and 3) considering the annular space without the use of a filter cake.

Firstly, several parameters (soil types, pipe material, depth of cover, consolidation, saturation, etc) could be applied to new FE models. As stated before, the FE models created in this research were following previous laboratory tests shown in Chapter 3 and 4. All scales and dimensions were too small to regard this model as a practical pipeline for installation because the objective of this simulation was only to compare the behavior of buried pipelines installed by two representative installation methods. Hence, several parameters could be applied to create a more practical FE model. OT installation in the FE models did not consider soil backfill properties, which may be looser than the soil property applied to this research. In addition, future work might change the mesh shape or number of elements in order to obtain optimal FEM results.

Secondly, the relationship of soil and pipeline in new FE models could be revealed through the stress analysis of different borehole pressures. Before performing this, however, we would have to determine the relationship between borehole pressures and pipe buoyancy. During HDD installation, designers and contractors should consider borehole pressures that causes hydro-fracture or unconfined plastic failure. There are various stresses occurring around the annular space during the HDD pullback process. Tension, bending, external hoop, and pipe overbend at the entry are good examples of stress occurring around the pipeline during HDD installation (Harper 1999). Various forces (i.e. buoyant or frictional forces between the borehole wall and the product pipe) inside the annular space act on segments of the pipe (Huey et al 1996). Understanding all of the stresses and forces found in HDD installation are important to create healthy borehole conditions. This research assumed that the annular space was well-created. However, future work could be reversely assumed (borehole in poor status), simulating FE models with an unstable borehole having been installed.

Lastly, future research should clarify the relationship between pipe buoyancy and filter cake. Wang and Sterling (2007) found that without the filter cake, static liquefaction in adjacent soils around a pipe could occur due to high permeability. This liquefaction leads to soil erosion and fluidal movement, which could make a pipe float. This research was done without considering a filter cake; for future work, the filter cake should be included.

Overall, using FE models for future research will help build a robust model for pipe buoyancy and successful pipe installation for contractors, engineers and even researchers.

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APPENDIX A

[FEM] CRITICAL H/D RATIOS IN OT EXPERIMENT

Pipe Size (mm):[OD]	Traditional Open trench				
	Test NO	Depth of Cover (cm)		Depth from the bottom (cm)	Float (F) or not (N)
50 [60.3]	1	5.72	0.95D	11.75	F
	2	6.78	1.12D	12.81	F
	3	7.32	1.21D	13.35	F
	4	7.37	1.22D	13.40	F
	5	7.72	1.28D	13.75	F
	6	8.15	1.35D	14.19	F
	7	8.2	1.36D	14.24	F
	8	8.94	1.48D	14.97	F
	9	8.99	1.49D	15.02	F
	10	9.09	1.51D	15.13	N
	11	9.09	1.51D	15.13	N
75 [88.9]	1	6.86	0.77D	15.75	F
	2	7.24	0.81D	16.13	F
	3	7.87	0.89D	16.76	F
	4	8.56	0.96D	17.45	F
	5	10.06	1.13D	18.95	F
	6	10.67	1.20D	19.56	F
	7	10.8	1.21D	19.69	F
	8	10.8	1.21D	19.69	F
	9	10.9	1.23D	19.79	N
	10	11.1	1.25D	19.99	F
	11	11.53	1.30D	20.42	N
	12	11.63	1.31D	20.52	N
100 [114.3]	1	9.47	0.83D	20.90	F
	2	11.63	1.02D	23.06	F
	3	12.14	1.06D	23.57	F
	4	12.50	1.09D	23.93	F
	5	15.57	1.36D	27.00	F
	6	15.77	1.38D	27.20	F
	7	16.99	1.49D	28.42	F
	8	17.53	1.53D	28.96	F
	9	17.60	1.54D	29.03	F
	10	18.31	1.60D	29.74	F
	11	19.35	1.69D	30.78	F
	12	19.76	1.73D	31.19	N
	13	19.86	1.74D	31.29	N

APPENDIX B

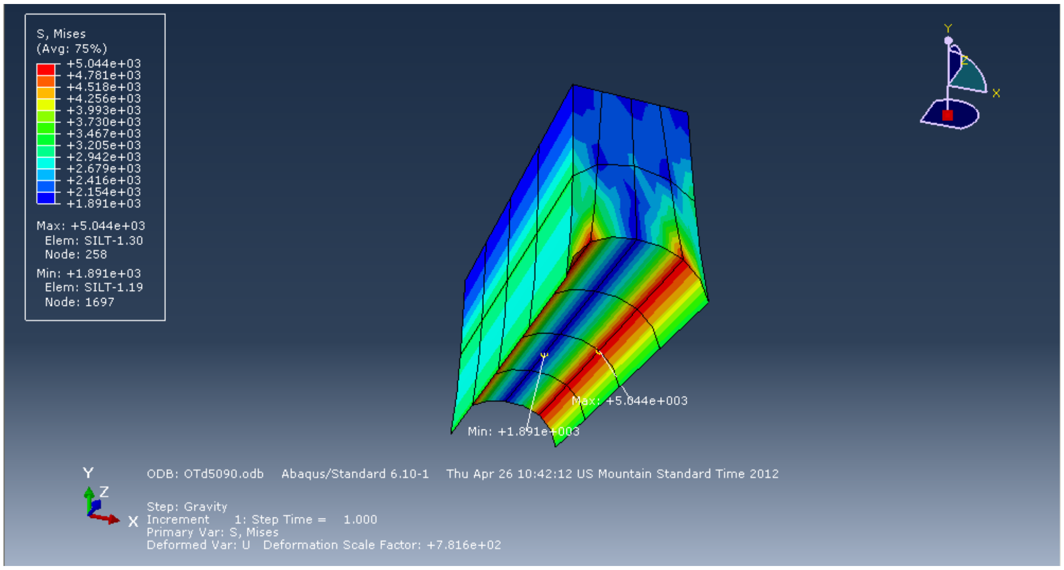
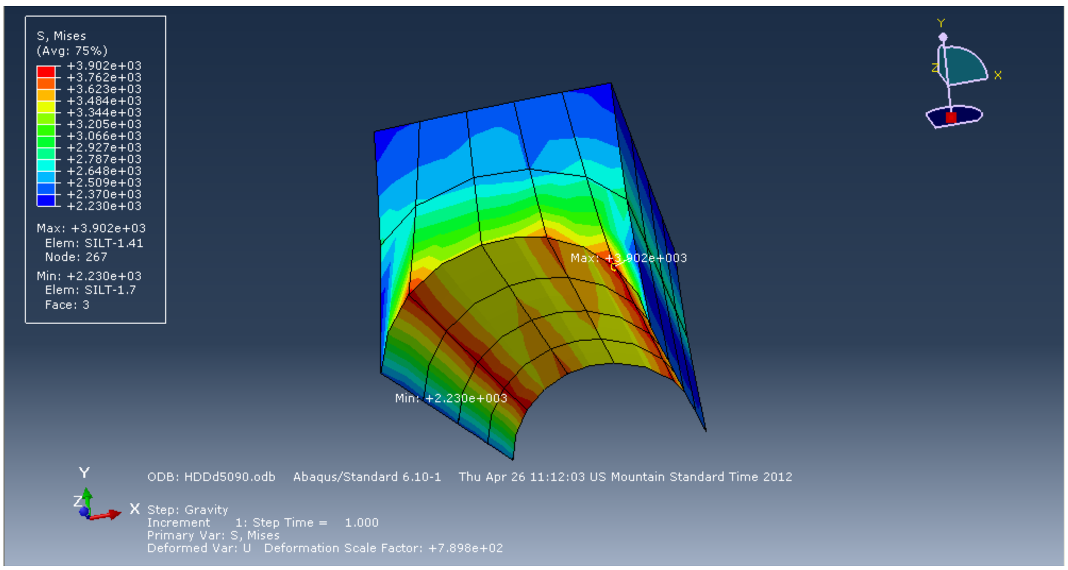
CRITICAL H/D RATIOS IN HDD EXPERIMENT

Pipe Size (mm):[OD]	Horizontal Directional Drilling				
	Test NO	Depth of Cover (cm)		Depth from the bottom of Pipe (cm)	Float (F) or not (N)
50 [60.3]	1	1.45	0.24D	7.48	F
	2	2.18	0.36D	8.22	F
	3	2.36	0.39D	8.39	N
	4	2.44	0.40D	8.47	N
	5	2.92	0.48D	8.95	N
	6	3.56	0.59D	9.59	N
	7	5.03	0.83D	11.06	N
	8	5.61	0.93D	11.65	N
75 [88.9]	1	6.99	0.79D	15.88	F
	2	7.49	0.84D	16.38	F
	3	7.67	0.86D	16.56	F
	4	7.80	0.88D	16.69	F
	5	8.08	0.91D	16.97	N
	6	8.18	0.92D	17.07	N
	7	8.46	0.95D	17.35	N
100 [114.3]	1	8.46	0.74D	19.89	F
	2	8.79	0.77D	20.22	F
	3	9.07	0.79D	20.50	F
	4	9.35	0.82D	20.78	N
	5	9.63	0.84D	21.06	N
	6	10.80	0.94D	22.23	N

APPENDIX C

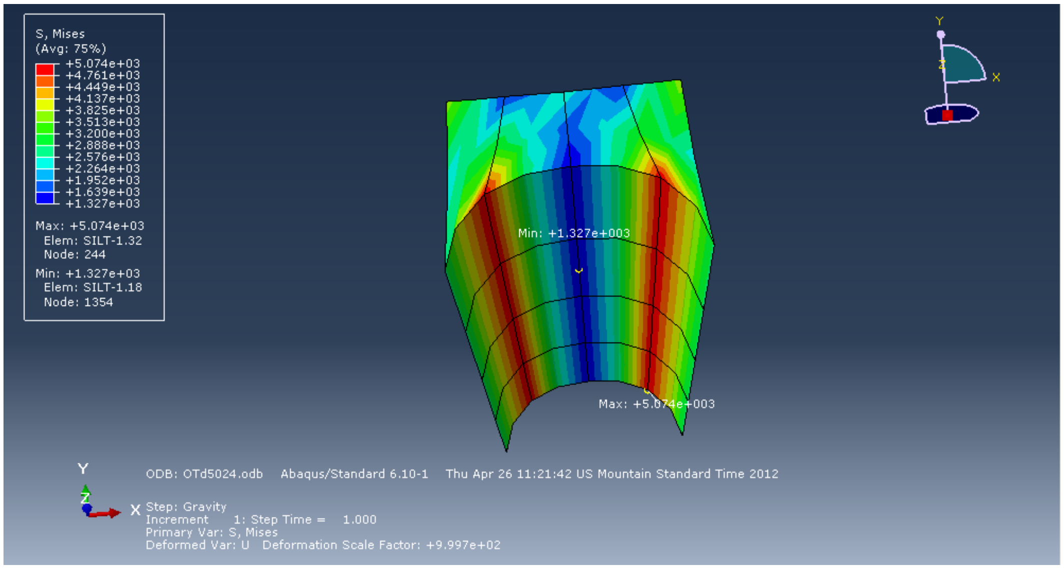
[FEM] STRESS ANALYSIS AT CRITICAL H/D RATIO

CRITICAL H/D RATIO: 1.51, **D50MM**

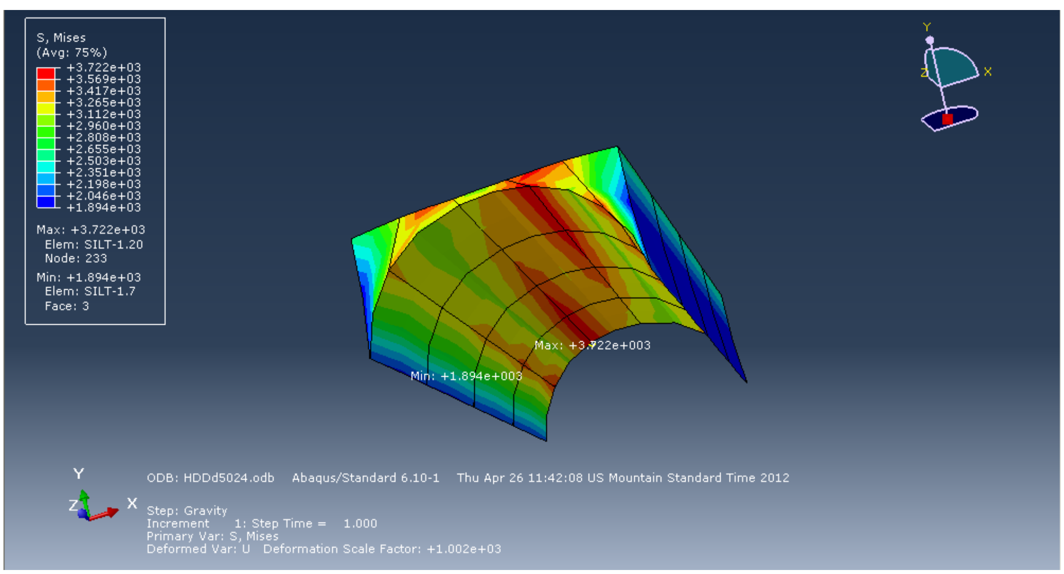
Maximum Soil Stress over the Pipeline in OT: **5.044kpa**

Maximum Soil Stress over the Pipeline in HDD: **3.902kpa**


CRITICAL H/D RATIO: 0.39, **D50MM**

Maximum Soil Stress over the Pipeline in OT: **5.074kpa**

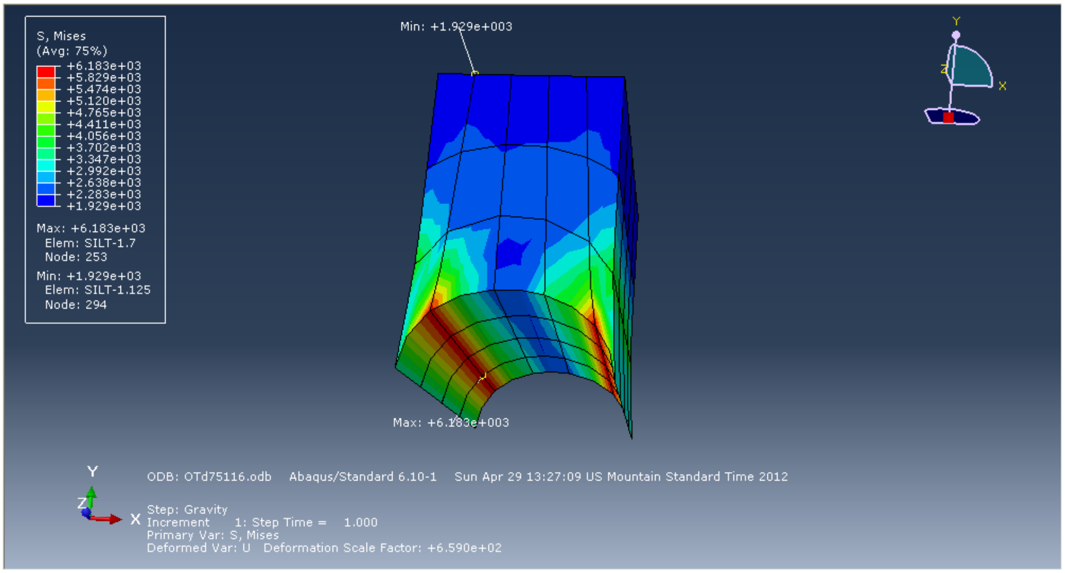


Maximum Soil Stress over the Pipeline in HDD: **3.722kpa**

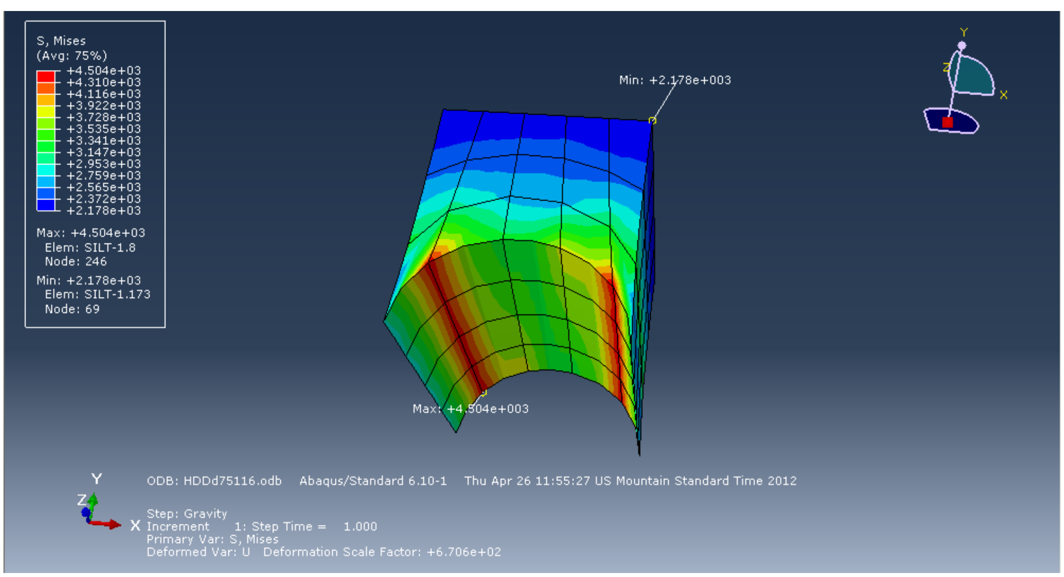


CRITICAL H/D RATIO: 1.30, **D75MM**

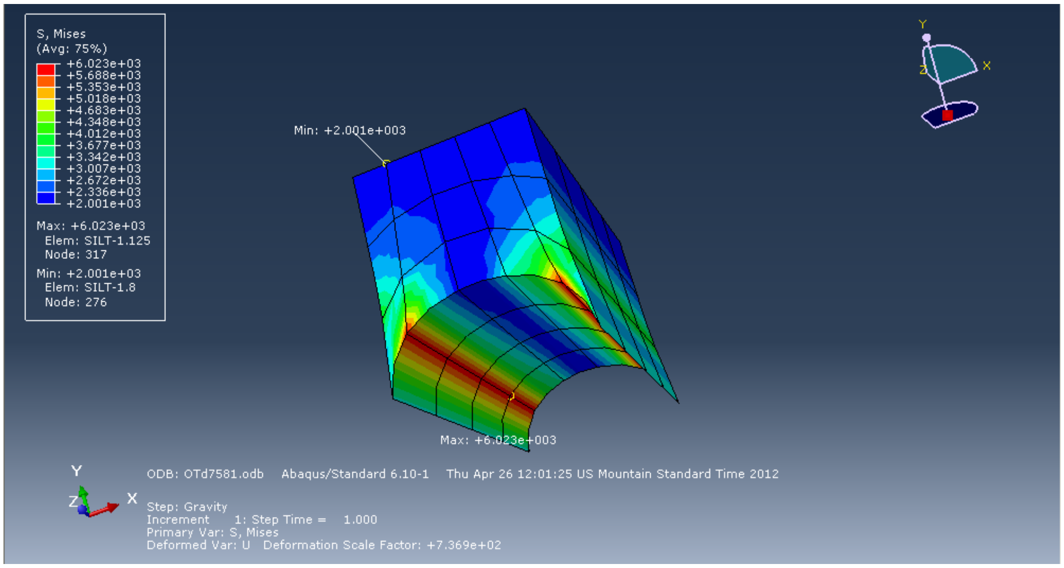
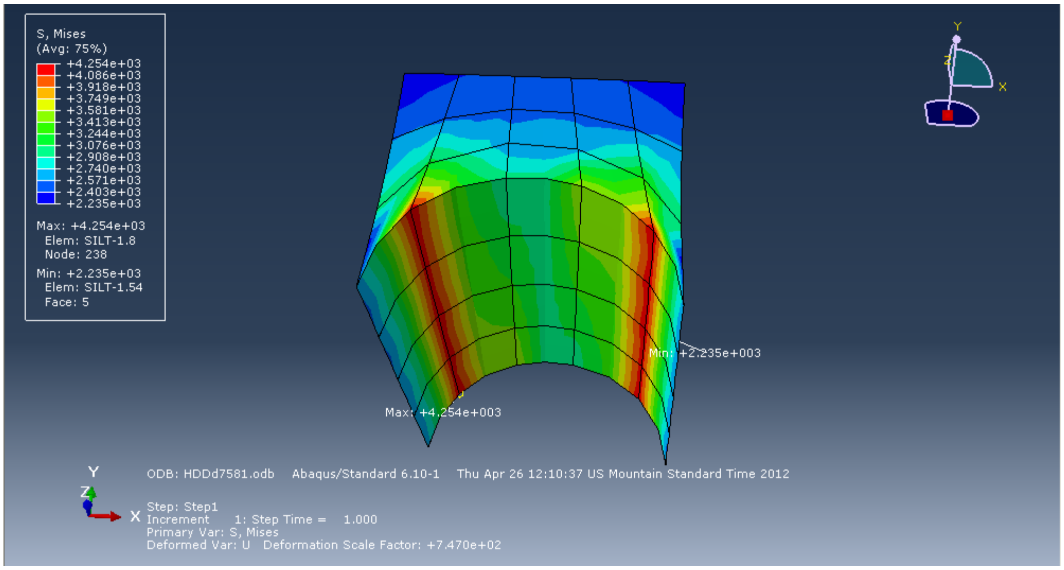
Maximum Soil Stress over the Pipeline in OT: **6.183kpa**



Maximum Soil Stress over the Pipeline in HDD: **4.504kpa**

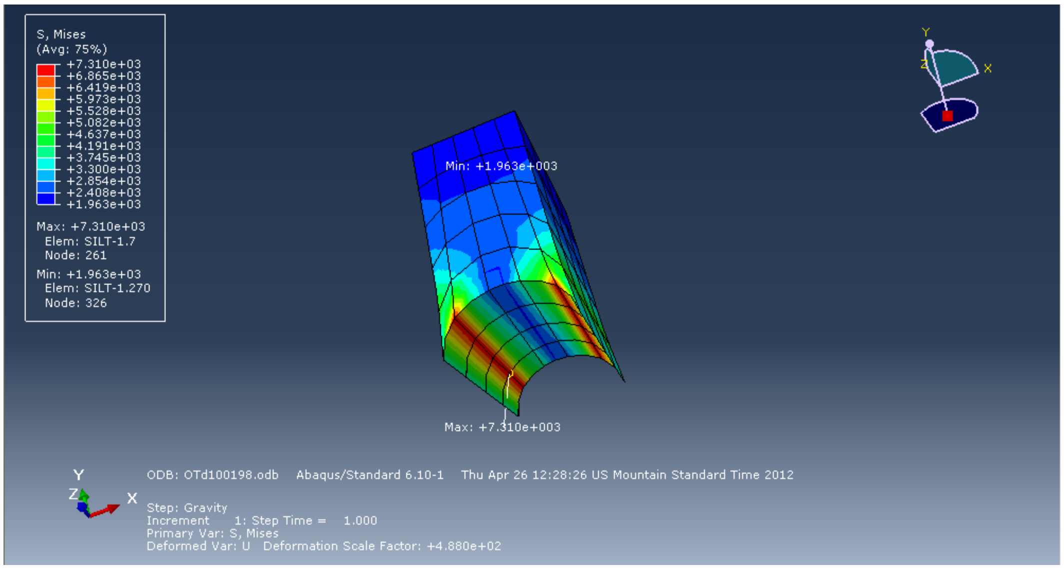


CRITICAL H/D RATIO: 0.91, **D75MM**

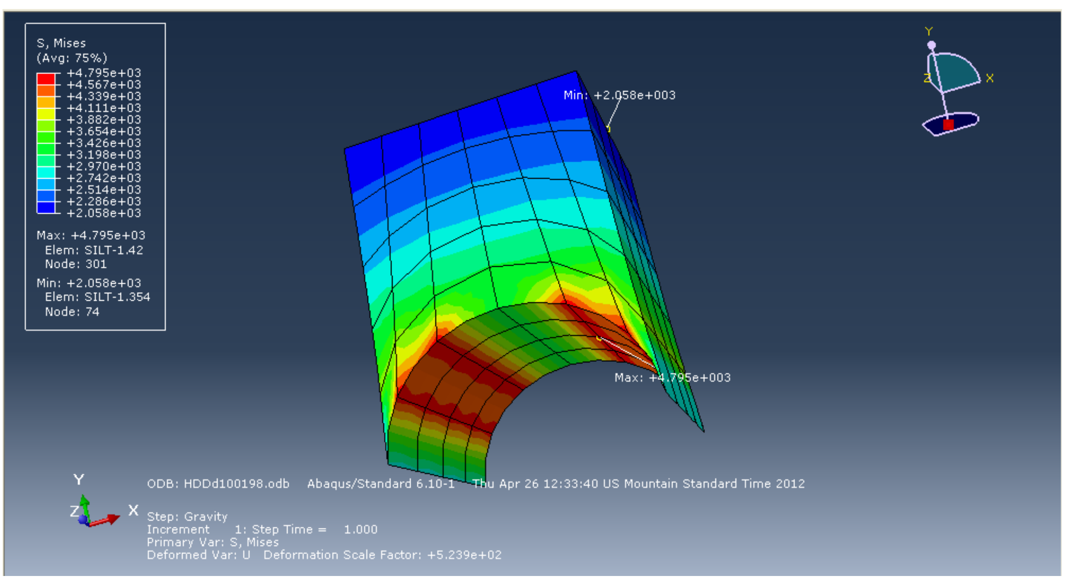
Maximum Soil Stress over the Pipeline in OT: **6.023kpa**

Maximum Soil Stress over the Pipeline in HDD: **4.254kpa**


CRITICAL H/D RATIO: 1.73, **D100MM**

Maximum Soil Stress over the Pipeline in OT: **7.310kpa**

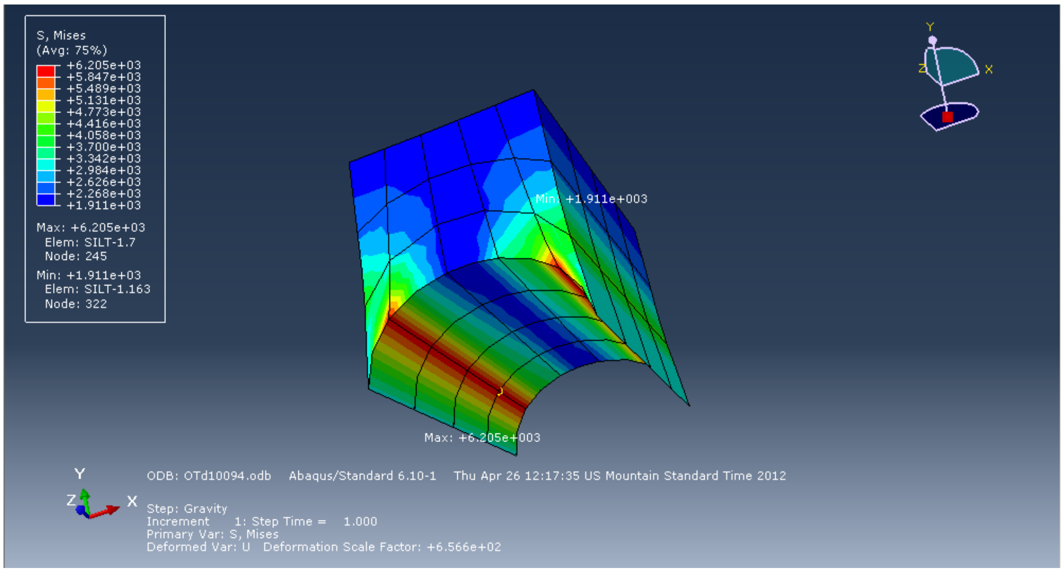


Maximum Soil Stress over the Pipeline in HDD: **4.795kpa**

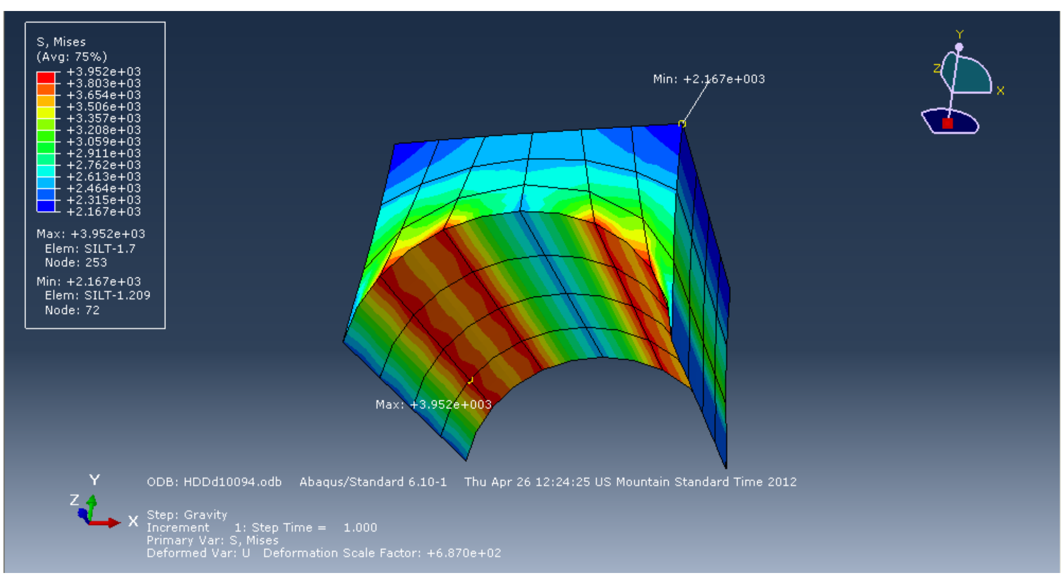


CRITICAL H/D RATIO: 0.82, **D100MM**

Maximum Soil Stress over the Pipeline in OT: **6.205kpa**



Maximum Soil Stress over the Pipeline in HDD: **3.952kpa**

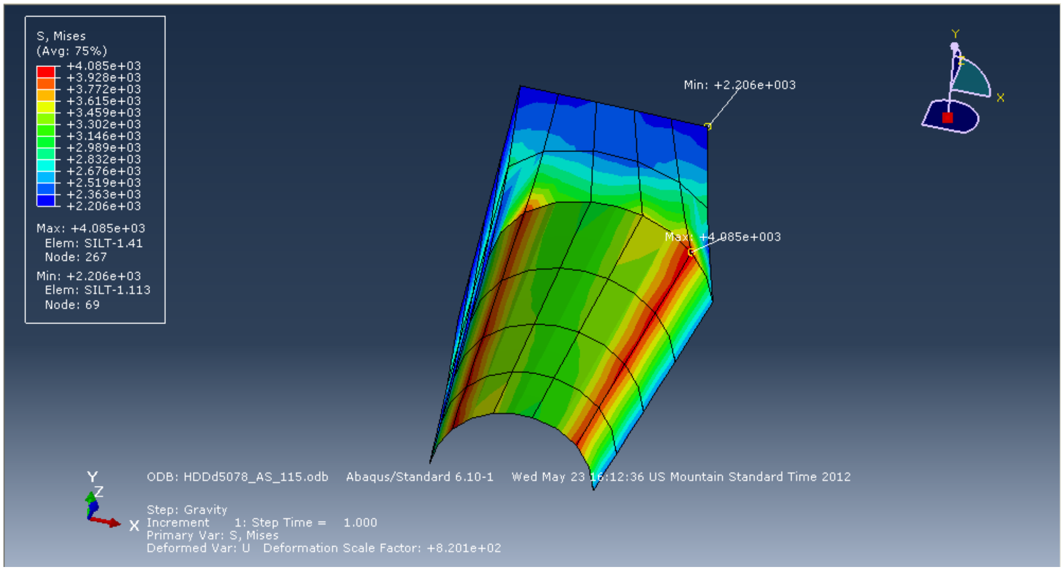


APPENDIX D

[FEM] CHANGING ANNULAR SPACE DENSITY: D50MM

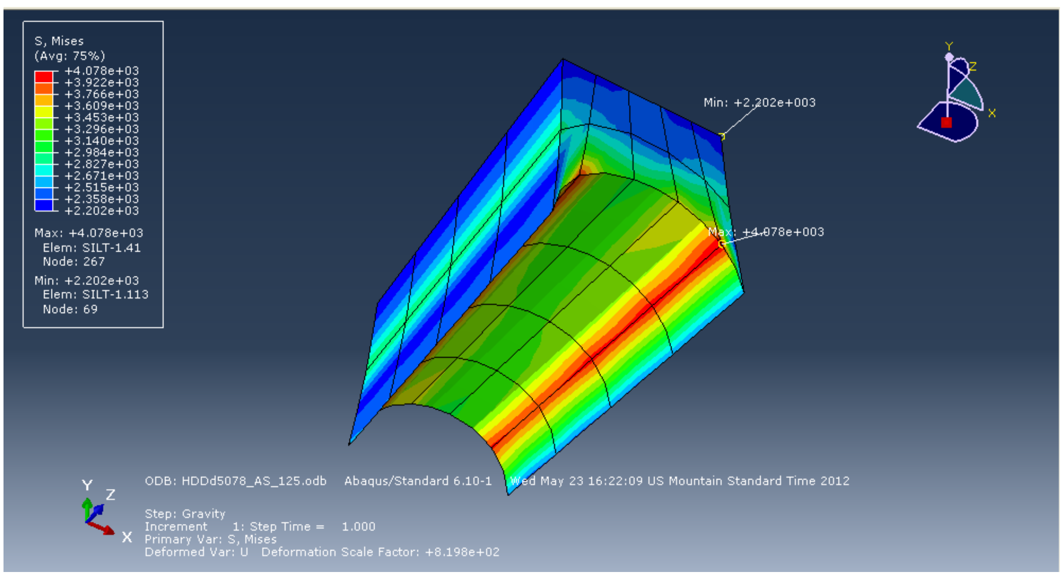
DENSITY: 1.15 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: <u>**4.085kpa**</u>



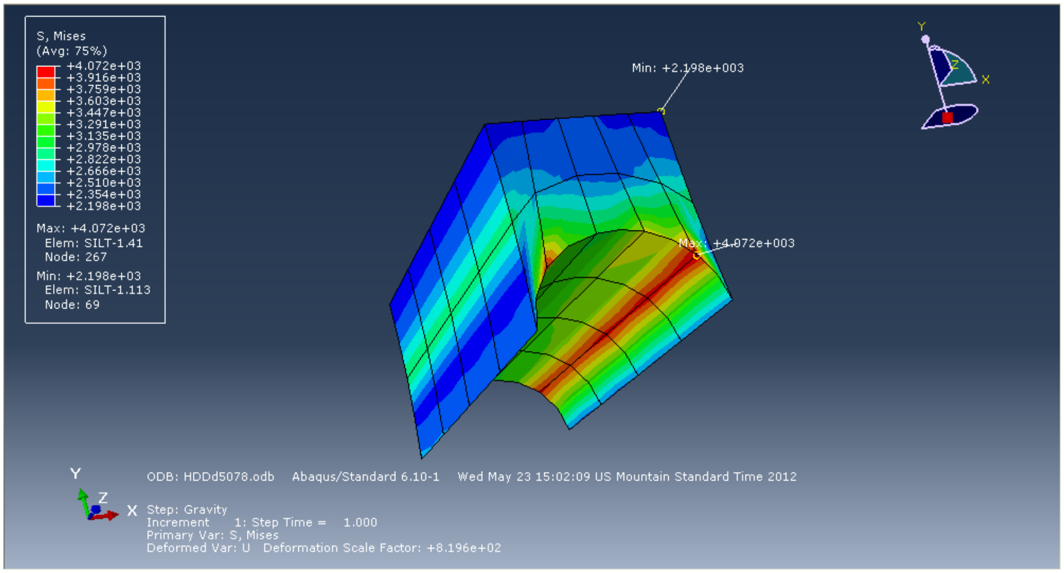
DENSITY: 1.25 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: <u>**4.078kpa**</u>



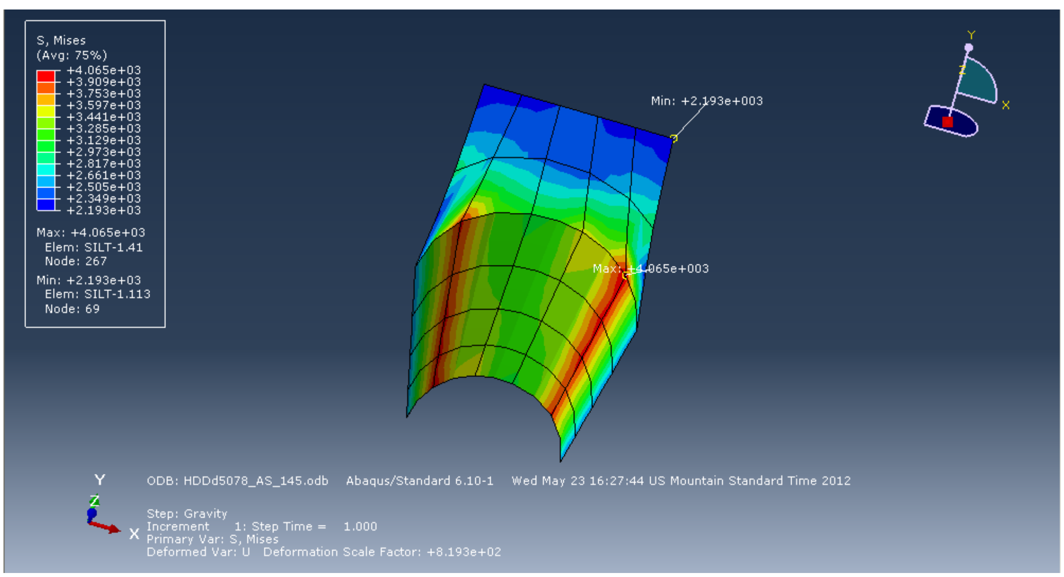
DENSITY: 1.35 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.072kpa**

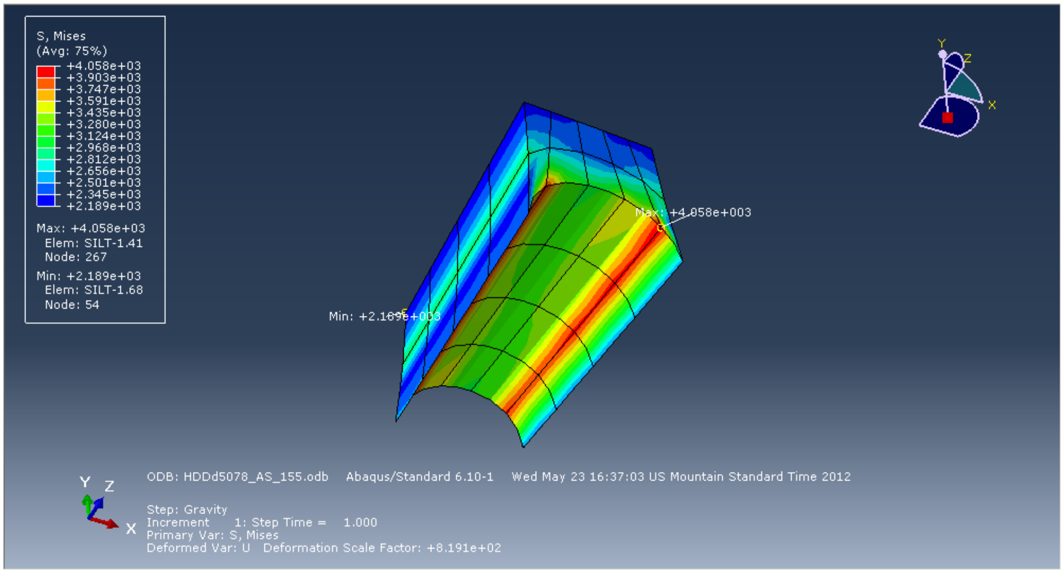


DENSITY: 1.45 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.065kpa**



DENSITY: 1.55 TON/M^3

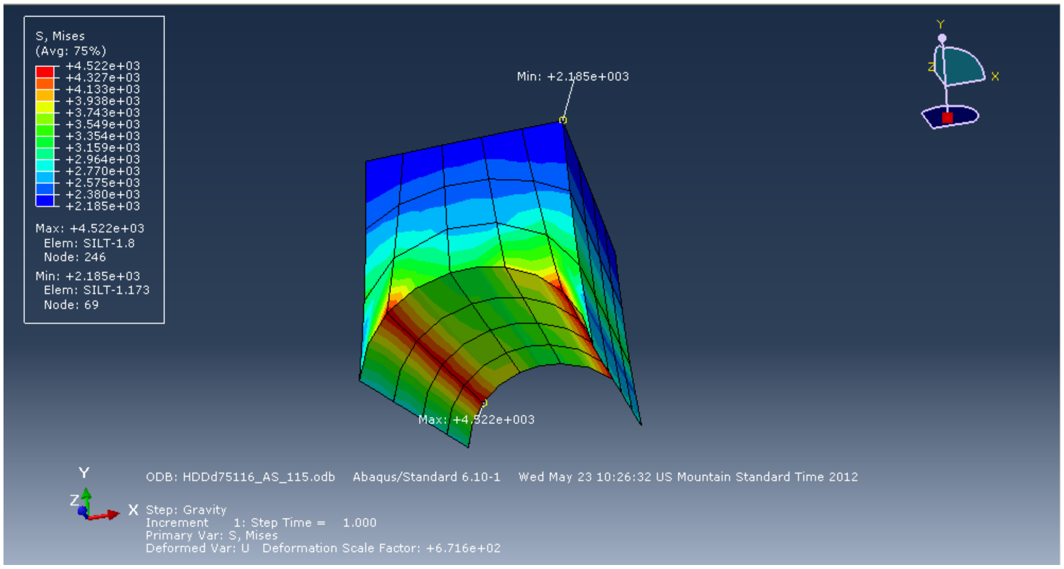
Maximum Soil Stress over the Pipeline in HDD: **<u>4.058kpa</u>**


APPENDIX E

[FEM] CHANGING ANNULAR SPACE DENSITY: D75MM

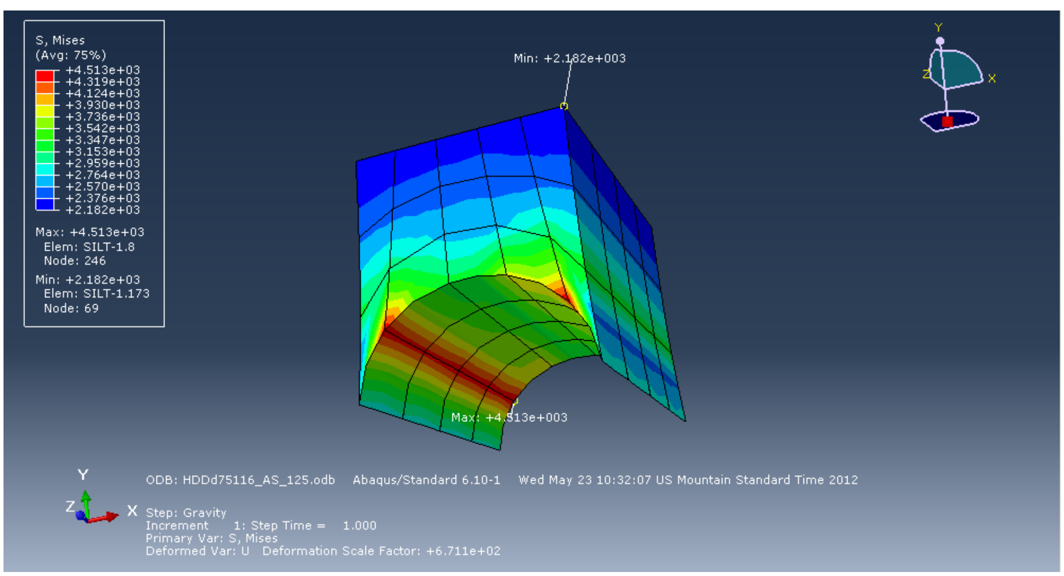
DENSITY: 1.15 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.522kpa**



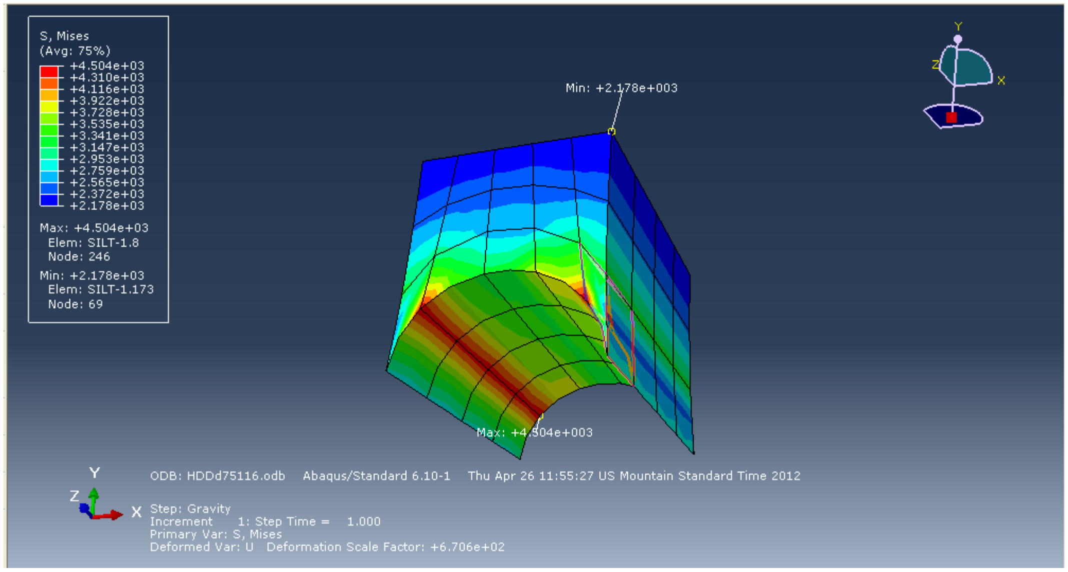
DENSITY: 1.25 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.513kpa**



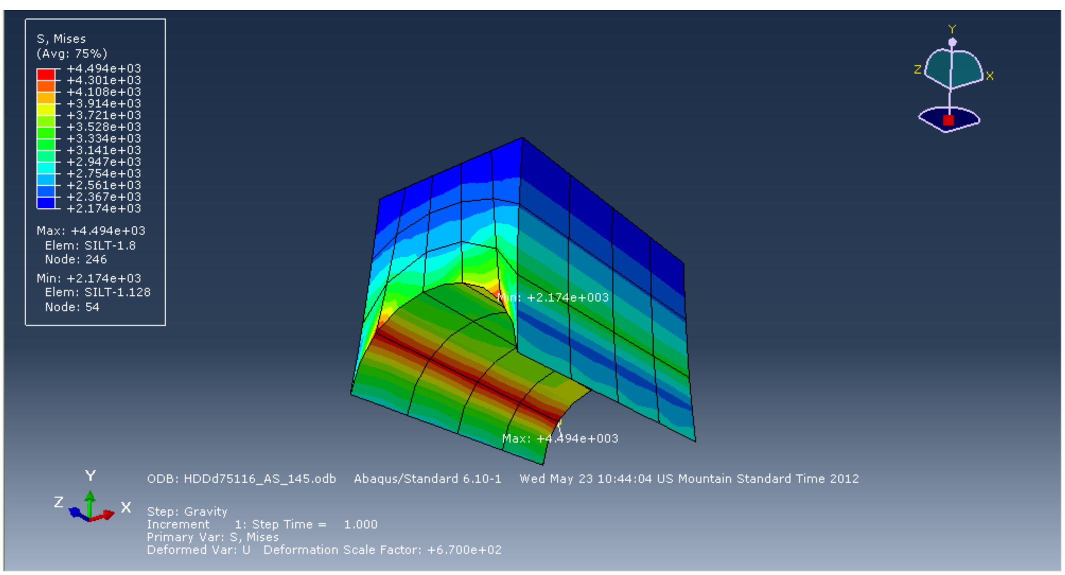
DENSITY: 1.35 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **<u>4.504kpa</u>**



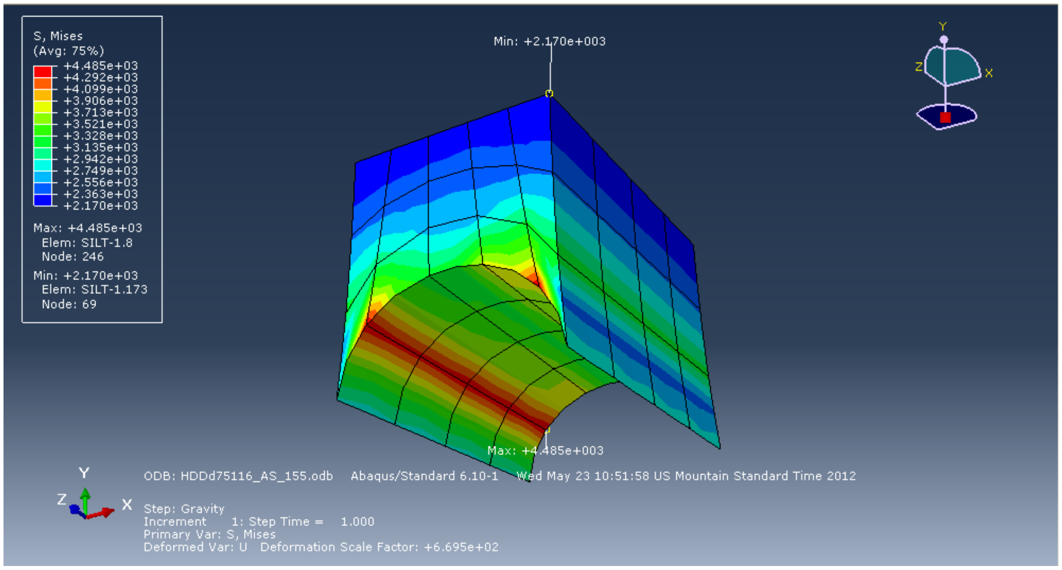
DENSITY: 1.45 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **<u>4.494kpa</u>**



DENSITY: 1.55 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.485kpa**

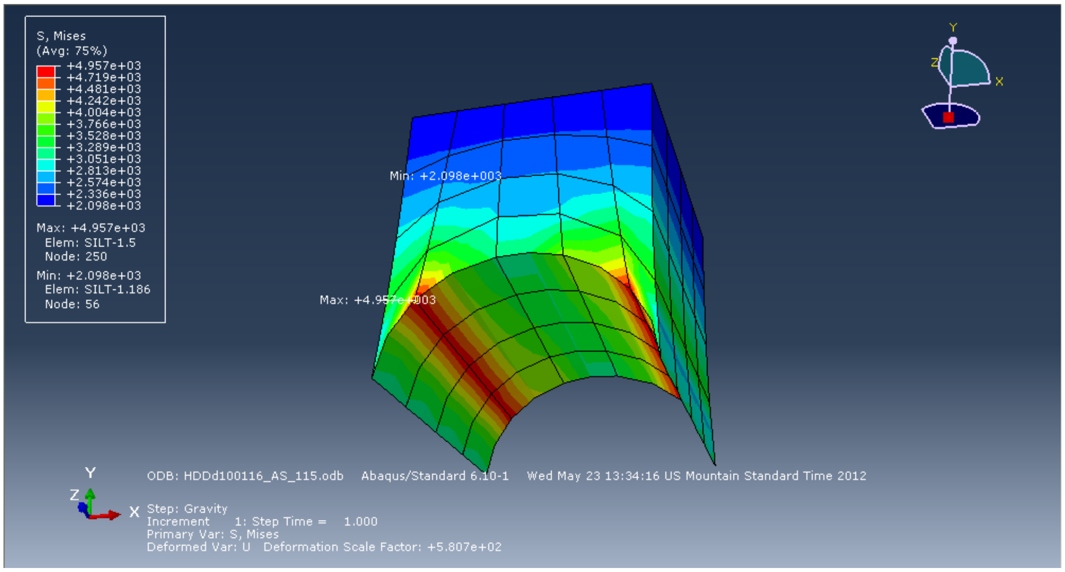


APPENDIX F

[FEM] CHANGING ANNULAR SPACE DENSITY: D100MM

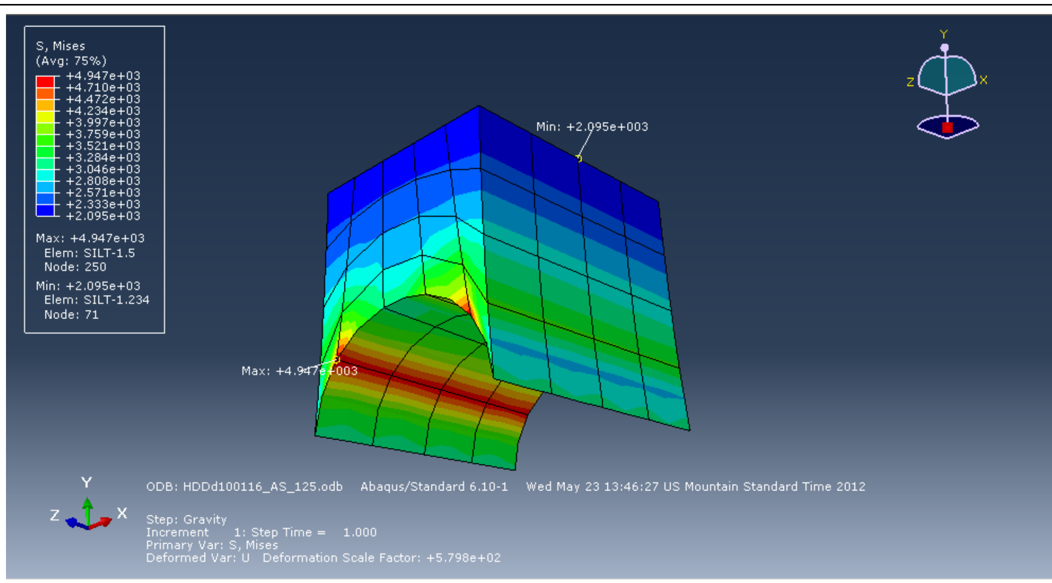
DENSITY: 1.15 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.957kpa**



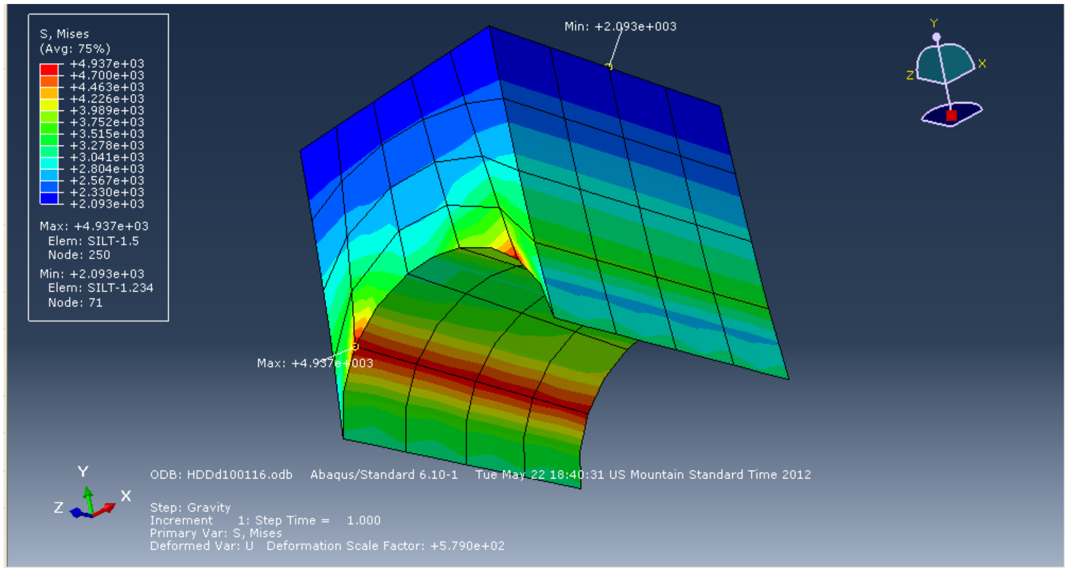
DENSITY: 1.25 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.947kpa**



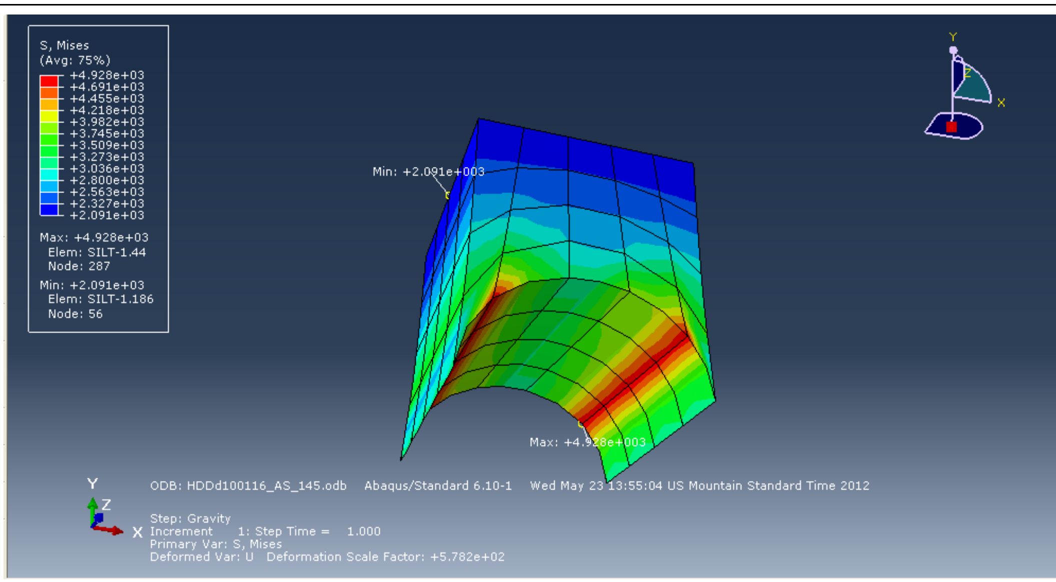
DENSITY: 1.35 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **<u>4.937kpa</u>**



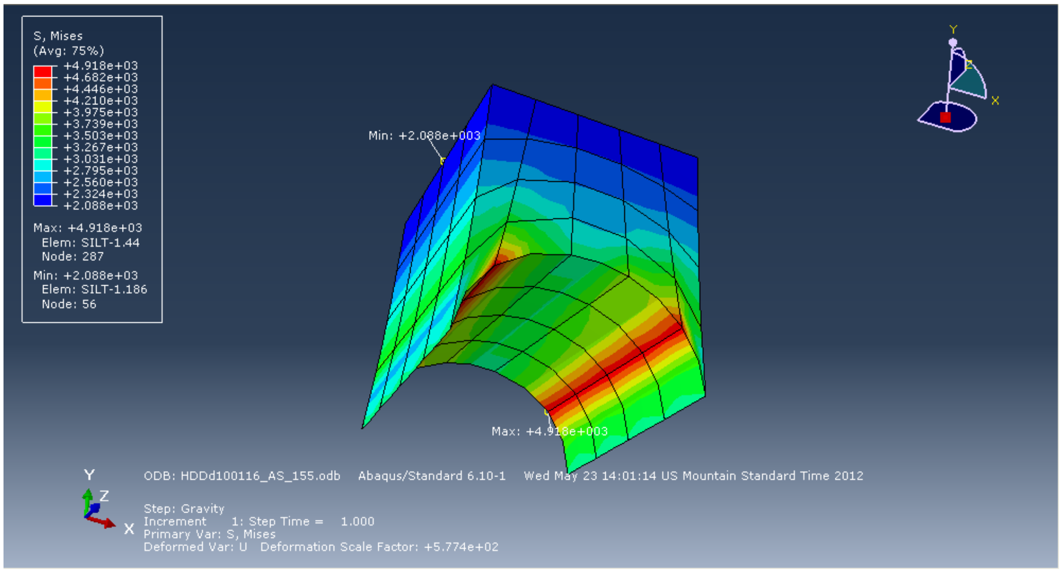
DENSITY: 1.45 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **<u>4.928kpa</u>**



DENSITY: 1.55 TON/M^3

Maximum Soil Stress over the Pipeline in HDD: **4.918kpa**

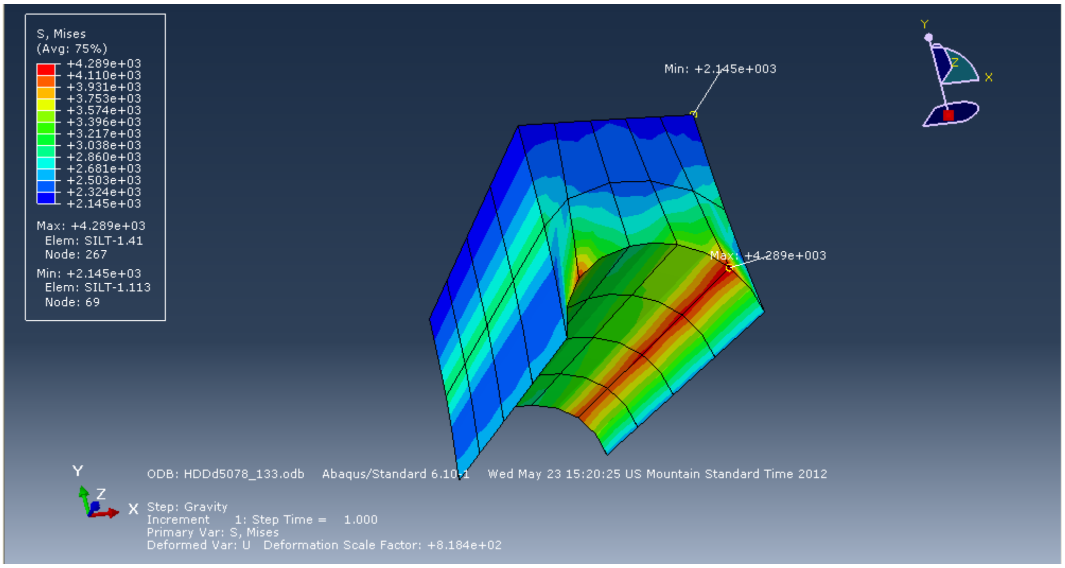


APPENDIX G

[FEM] CHANGING ANNULAR SPACE DIAMETER: D50MM

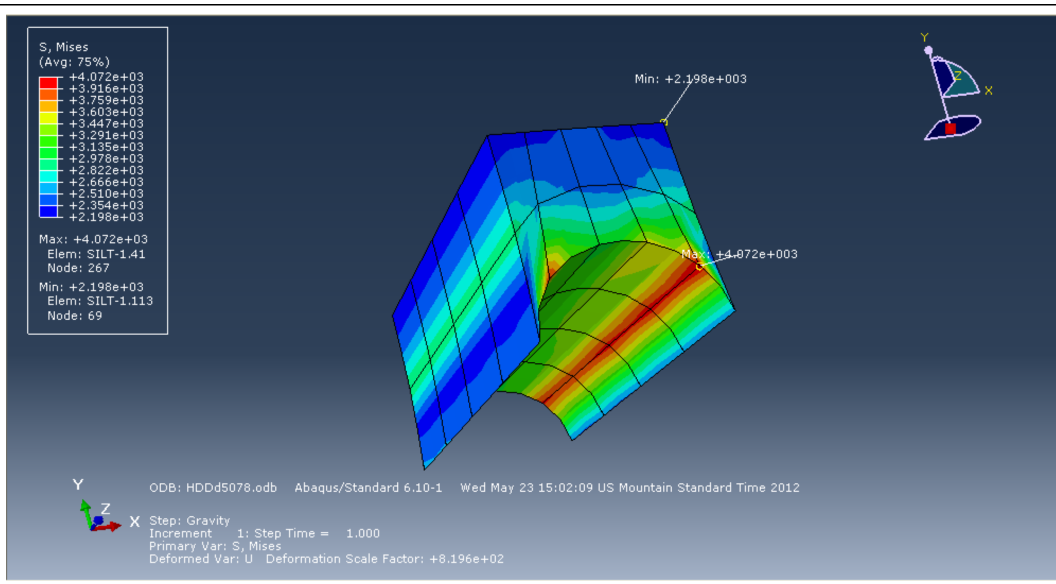
OUTER DIAMETER (OD) × 1.33

Maximum Soil Stress over the Pipeline in HDD: **4.289kpa**



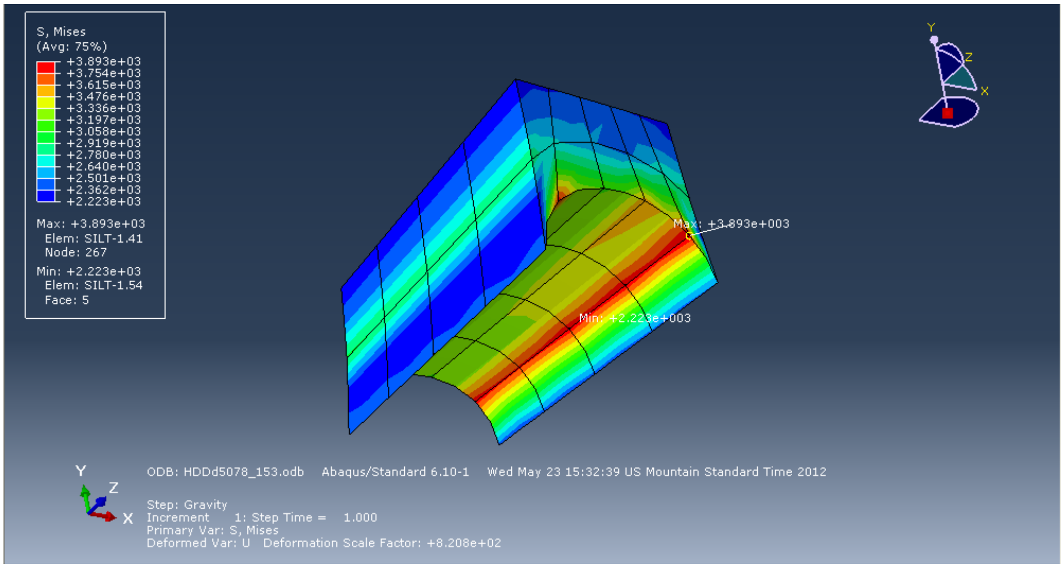
OUTER DIAMETER (OD) × 1.43

Maximum Soil Stress over the Pipeline in HDD: **4.072kpa**



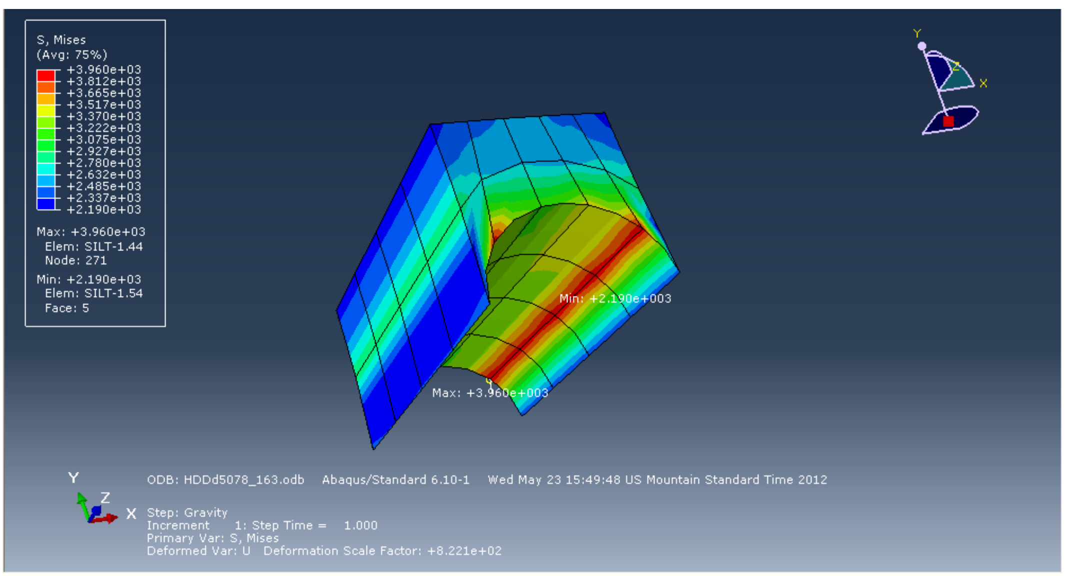
OUTER DIAMETER (OD) × 1.53

Maximum Soil Stress over the Pipeline in HDD: **3.893kpa**



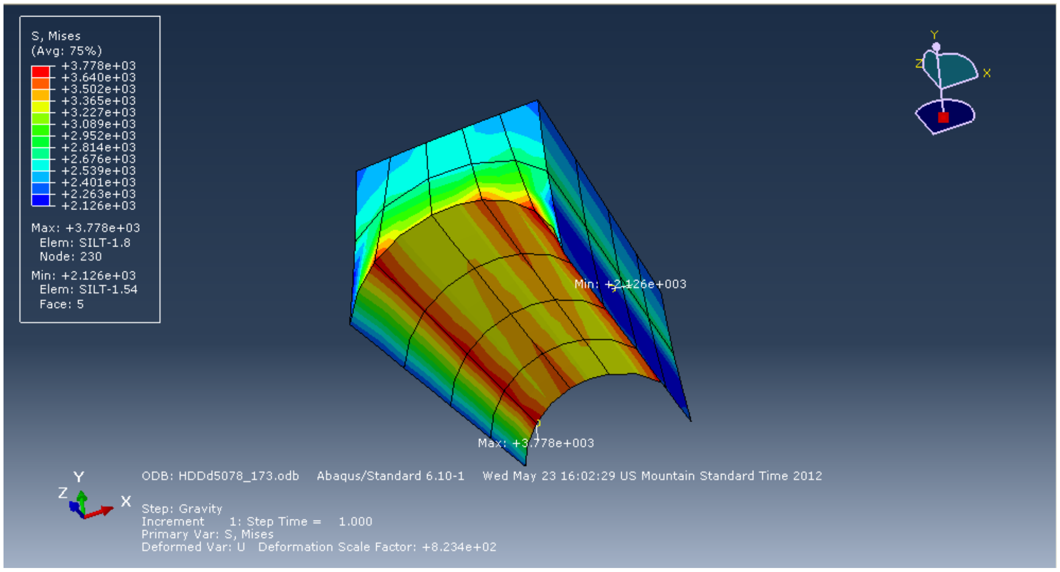
OUTER DIAMETER (OD) × 1.63

Maximum Soil Stress over the Pipeline in HDD: **3.963kpa**



OUTER DIAMETER (OD) × 1.73

Maximum Soil Stress over the Pipeline in HDD: **3.778kpa**

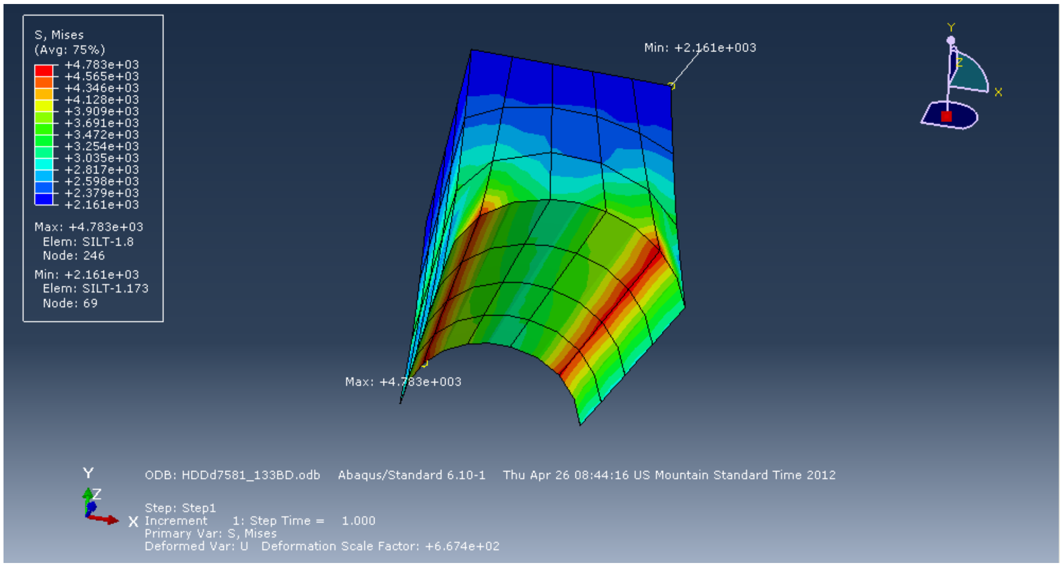


APPENDIX H

[FEM] CHANGING ANNULAR SPACE DIAMETER: D75MM

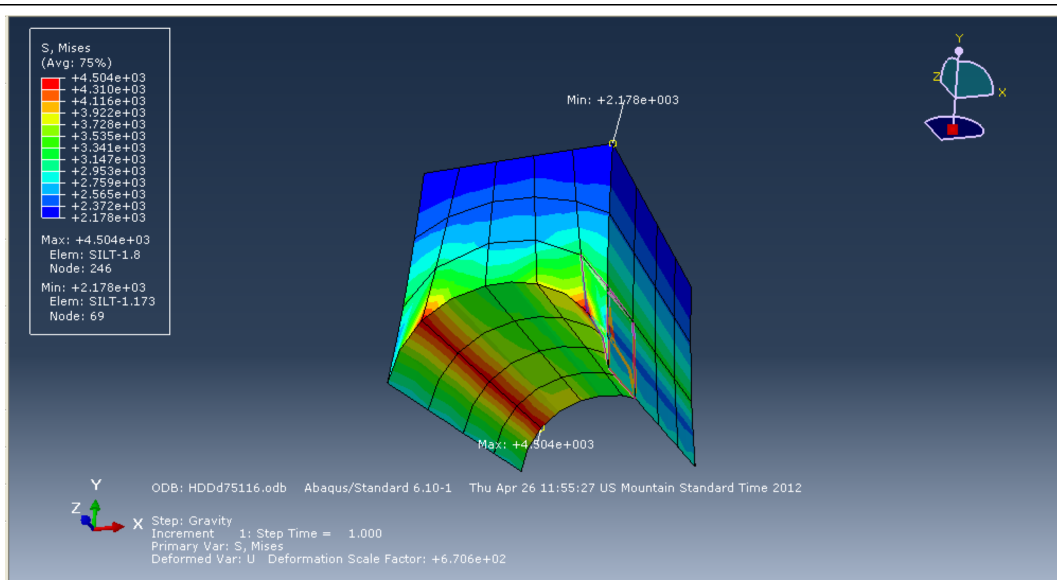
OUTER DIAMETER (OD) × 1.33

Maximum Soil Stress over the Pipeline in HDD: **4.783kpa**



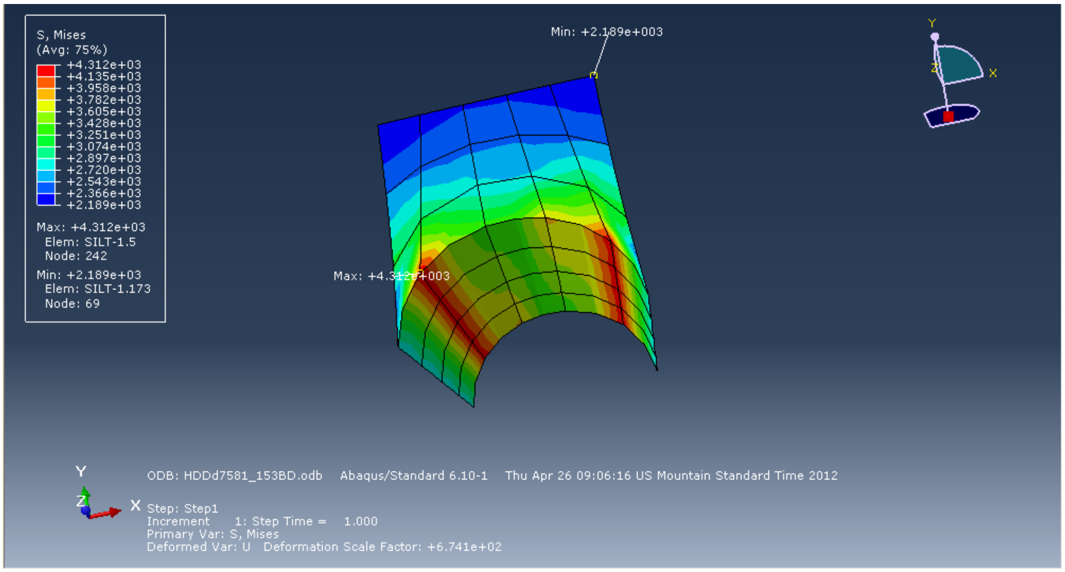
OUTER DIAMETER (OD) × 1.43

Maximum Soil Stress over the Pipeline in HDD: **4.504kpa**



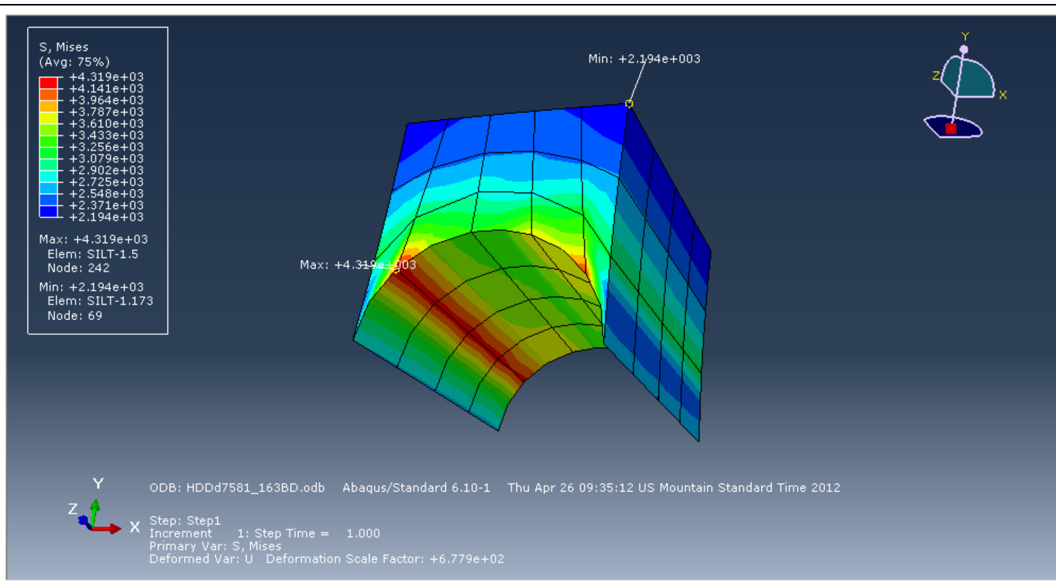
OUTER DIAMETER (OD) × 1.53

Maximum Soil Stress over the Pipeline in HDD: **<u>4.312kpa</u>**



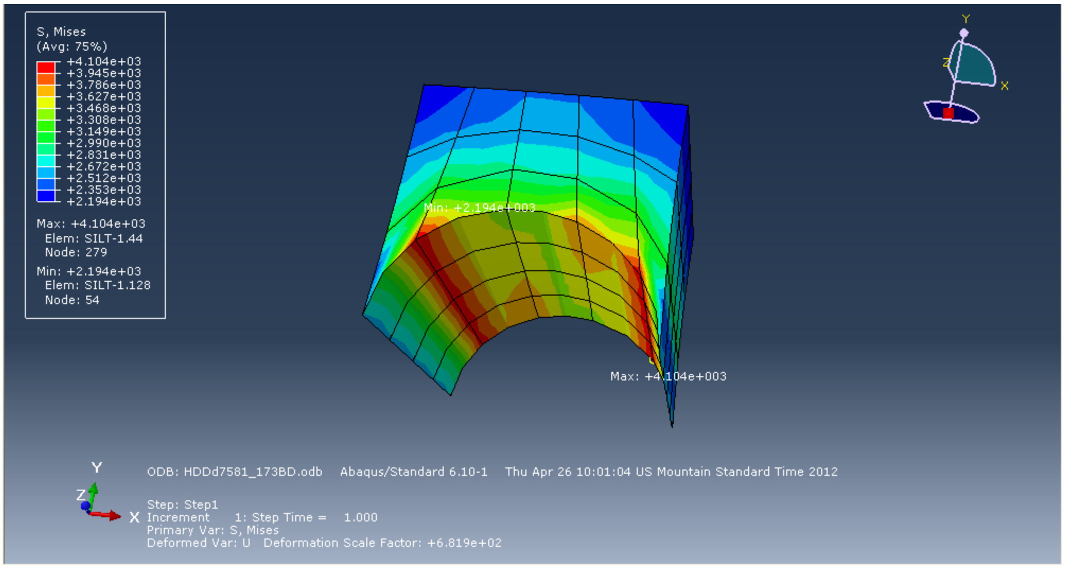
OUTER DIAMETER (OD) × 1.63

Maximum Soil Stress over the Pipeline in HDD: **<u>4.319kpa</u>**



OUTER DIAMETER (OD) × 1.73

Maximum Soil Stress over the Pipeline in HDD: **4.104kpa**

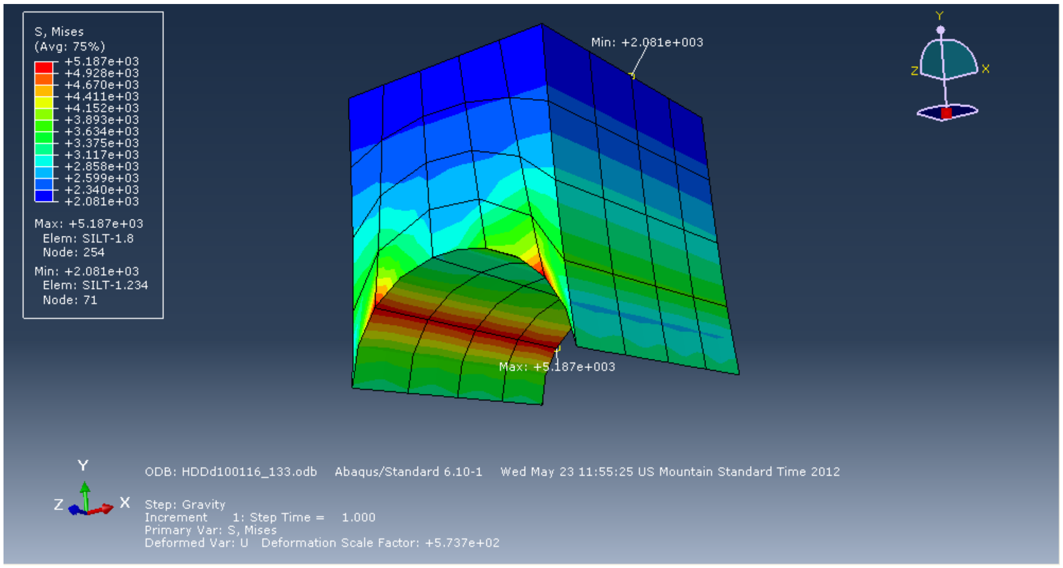


APPENDIX I

[FEM] CHANGING ANNULAR SPACE DIAMETER: D100MM

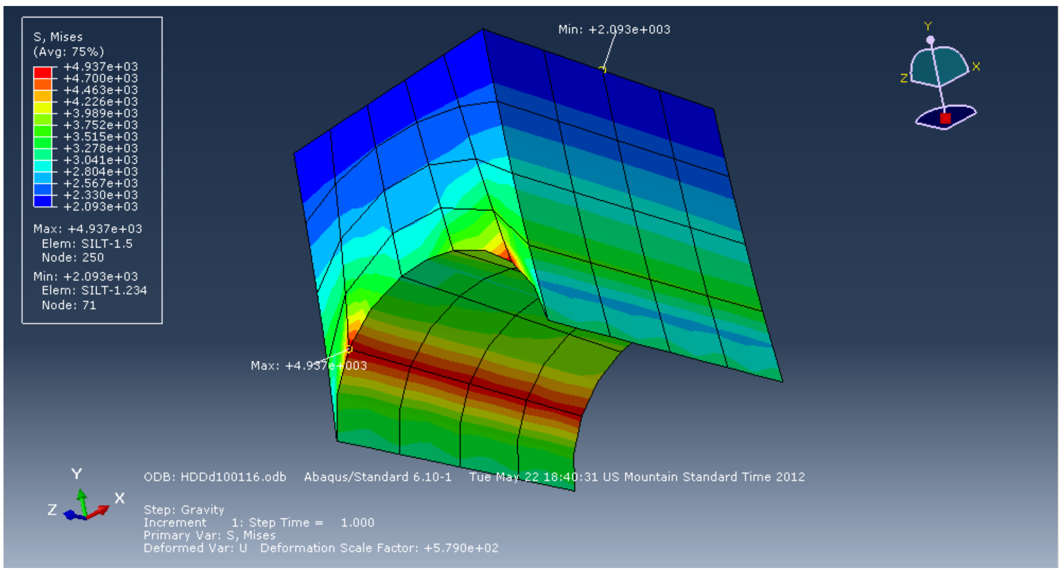
OUTER DIAMETER (OD) × 1.33

Maximum Soil Stress over the Pipeline in HDD: **5.187kpa**



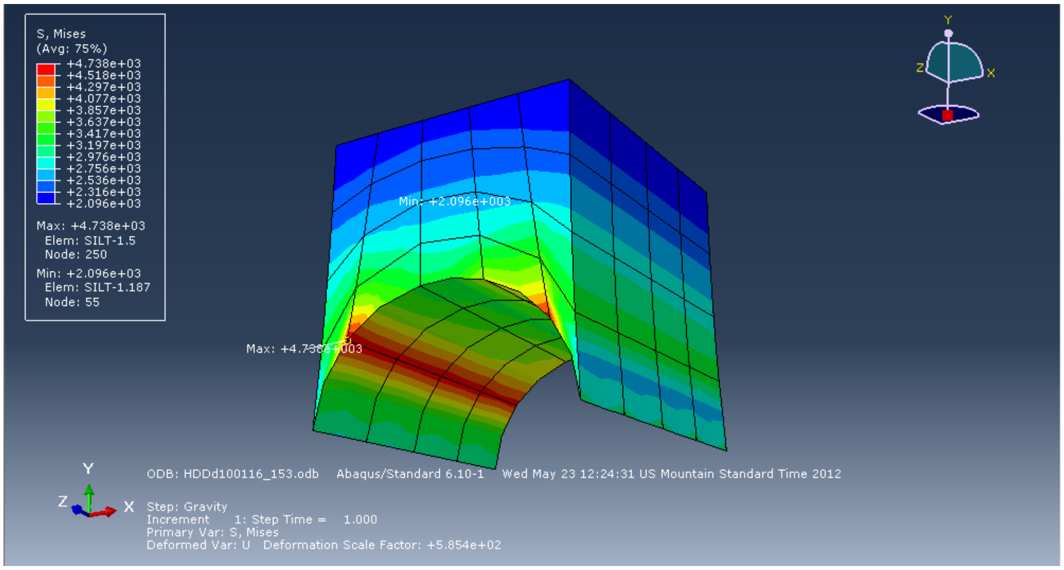
OUTER DIAMETER (OD) × 1.43

Maximum Soil Stress over the Pipeline in HDD: **4.937kpa**



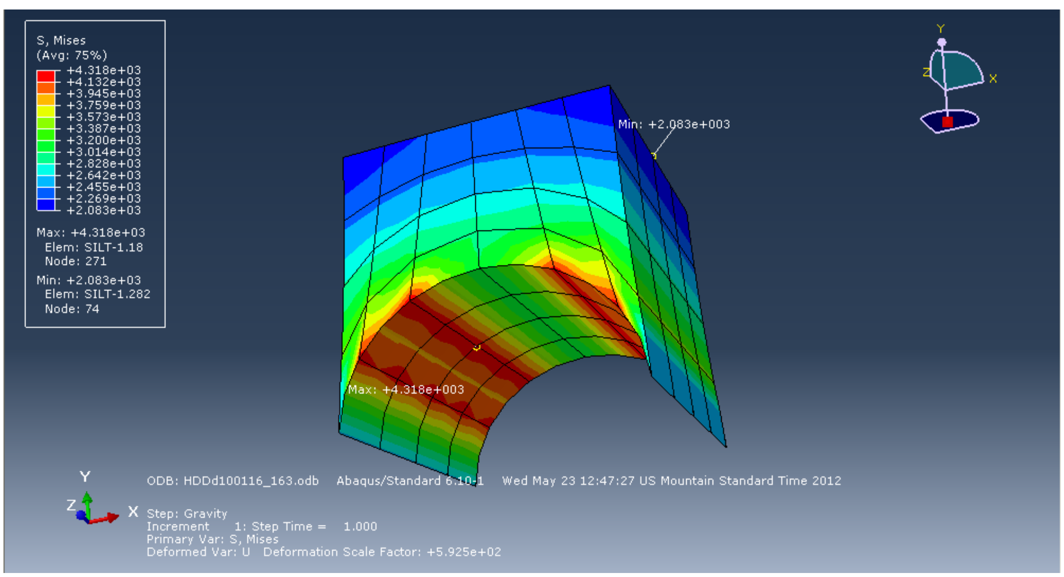
OUTER DIAMETER (OD) × 1.53

Maximum Soil Stress over the Pipeline in HDD: **4.738kpa**



OUTER DIAMETER (OD) × 1.63

Maximum Soil Stress over the Pipeline in HDD: **4.318kpa**



OUTER DIAMETER (OD) × 1.73

Maximum Soil Stress over the Pipeline in HDD: **<u>4.204kpa</u>**