Pipeline-backfill-trench interaction during large lateral displacements

by

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In the name of God

Dedicated to

my parents, and

my wife

Abstract

Subsea pipelines, particularly in shallow areas, are usually buried inside trenches backfilled with pre-excavated material as cost-effective protection against the environmental, constructional, and operational loads. The design of buried pipelines against potential lateral displacements is a challenging task that is usually simplified by assuming a uniform soil surrounding the pipeline. However, the remolded backfill and its lower stiffness compared with the native ground can significantly affect the failure mechanisms around the moving pipe and the mobilized lateral soil resistance. Having a trench backfilled with a material softer than the native seabed soil will lead to a complicated pipe-soil interaction problem which has not been entirely explored in the literature. In this study, the lateral pipeline-backfill-trench interaction and the resultant soil failure mechanisms were investigated by centrifuge models (in partially drained conditions) and also numerical simulations (in undrained conditions). Particle image velocimetry (PIV) analysis was conducted to capture the interactive soil displacements and failure mechanisms during centrifuge tests. It was observed that the interactive effects of pipeline, backfill, and trench precede their individual shear strengths.

The advanced numerical simulations were developed by using the Coupled Eulerian-Lagrangian (CEL) approach with two different Eulerian materials behaving in undrained conditions. The numerical simulations in undrained conditions show a good agreement with the previously conducted centrifuge tests in terms of lateral load-displacement response

and failure mechanisms. The investigated parameters are pipe roughness, pipe weight, pipe initial embedment into the trench-bed, backfill strength properties, soil strain-softening, native soil tension cut-off, and burial depth. The effects of influential parameters are comprehensively examined using the developed numerical model, and the results show good agreement with some previously conducted centrifuge tests.

The study revealed the significance of the pipeline-trenchbed interaction in the mobilization of the lateral soil resistance and several other mechanisms not yet addressed in the literature. As a result, several new research avenues were identified, and the ground was prepared for proposing solutions to improve the prediction of the lateral response of buried pipelines in the near future.

Keywords: Lateral pipe-soil interaction; buried pipeline; p-y response; centrifuge testing; trenching and backfilling; large deformation finite element analysis; numerical modeling; Coupled Eulerian-Lagrangian method

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Nomenclature

CEL	Coupled Eulerian-Lagrangian
c_u (s_u)	Undrained shear strength of soil
c_v	Coefficient of consolidation
D	Pipe diameter
E	Young's modulus
F_h	Horizontal force
H	Burial depth up to the pipe centerline
H/D	Burial depth ratio
K_{su}	Shear strength gradient
LVDT	Linear Variable differential Transformer
N_c	Horizontal bearing capacity factor for clay
N_q	Horizontal bearing capacity factor for sand
PIV	Particle Image Velocimetry
PPT	Pore Pressure Transducer
P_{ult}	Ultimate soil resistance
p-y	Load-displacement curve
RITSS	Remeshing and Interpolation Technique with the Small Strain
S_{um}	Mudline intercept
vD/c_v	Normalized pipe velocity
y/D	Normalized lateral displacement
α	Interface roughness factor
Δ_p	Required displacement to reach the ultimate resistance

δ_{rem}	Inverse of the clay sensitivity
Υ	Soil unit weight
Υ_{sat}	Saturated unit weight
Υ_{sat}	Submerged unit weight
ν	Poison's ratio
ξ	Accumulated absolute plastic shear strain at the Gauss point
ξ_{95}	ξ for the soil to undergo 95% remolding
$\dot{\Upsilon}_{ref}$	Reference shear strain rate
μ	Rate of strength increase per decade

CHAPTER 1. Introduction

1.1 Background and motivation

Pipelines are widely used for hydrocarbon and water long-distance transportation. There are about 3.5 million kilometers of pipeline in 120 countries of the world. The United States, by 65%, Russia by 8%, and Canada by 3% of the whole length of the world pipelines are the first three countries of having the highest length of the pipelines. Submarine pipelines continue to be buried to reduce the risk of hydrodynamic force and increase the stability of pipeline section; protecting the pipeline section from the geo-hazards and external damage due to anchors, heavy dropped objects or fishing gear; and improving other pipeline structural performance, such as free span, lateral buckling, and insulation performance (Bai and Bai 2014). Even though the pipelines are protected to a great extent when they are buried inside a trench, they still require attention regarding the possible relative displacement between the pipeline and soil, causing pipe-soil interaction. Pipeline-soil interaction is a significant aspect of a pipeline system as it may have a large influence on the structural integrity of the pipeline during installation and operation. Buried pipelines may encounter various geo-hazards that impose differential ground movement on buried pipelines. Landslides, ice gouging, earthquakes, fault movements, and external impacts of anchors are some of the potential scenarios causing lateral loading and, subsequently, large deformation which buried pipelines have to be designed for. The imposed loadings and the resulting large deformations may exceed serviceability and ultimate limit states. Subsea trenching and backfilling by re-using the excavated material is usually a cost-effective solution to protect the pipeline against lateral displacements. Depending on the trenching methodology, the construction procedure, and the environmental loads, the backfill material may undergo different degrees of remoulding and disturbance. This process causes the backfill material to be much softer than the native ground, with a wide range of shear strengths ranging from negligible to almost native soil strength values. The difference between the stiffness of the backfill and native material can significantly affect the failure mechanisms around the pipe, and the resultant lateral soil resistance (Pipeline Research Council International (PRCI) 2003).

However, the less-explored interaction between the pipeline, backfill, and the native ground (trench walls) have caused the design standards to simplify the buried pipe configuration to a uniform soil. The practical incorporation of this simplification needs excavation of an adequately wide trench that results in a high construction cost. This is only to ensure the pipeline response will depend solely on the properties of the controlled backfill material, and not on the stiffer native ground.

In the present study, centrifuge tests were conducted at C-CORE to investigate the lateral soil response to the large displacements of the shallowly and deeply buried pipelines. A mixture of Speswhite kaolin clay and Sil-Co-Sil silt was used to prepare a soft native ground (undrained shear strength less than 25 kPa), which has been observed in shallow waters (e.g., water depth less than 100 m) over different geographical locations (e.g., Bohai Sea (Liu et al. 2013), Mackenzie Delta (Solomon 2003), Changi Bay (Bo et al. 2015), and Persian Gulf (DOT 2011)). The selection of a soft clay as native ground enabled observing the significance of the interactive failure mechanisms in lateral soil resistance, having even a limited difference between the stiffness of the backfill and trench. Trenches were excavated in prepared native ground and backfilled with slurry after locating the instrumented pipe sections. Particle image velocimetry (PIV) was applied to monitor the progressive pipeline-backfill-trench interaction mechanisms.

Using a low pipe displacement velocity, the partially drained condition was adopted to mitigate the impact of the excess pore pressure, magnify the pure effect of pipeline-backfill-trench interaction, and obtain the contribution of the trench to the overall failure mechanisms. The force-displacement (p-y) curves were obtained and compared with corresponding PIV analysis throughout a large course of pipeline displacements (about 4D). It was observed that the interactive effects of pipeline, backfill, and trench preceded the individual strength of backfill or native soil. In other words, the mobilized soil resistance was not solely originated from the shear strengths of backfill and native soil. The pipeline/trench-bed interaction, the trench geometry, and the passive backfill pressure affected the pipe displacement trajectory and consequently, the progressive formation of shear bands underneath and behind the collapsing trench wall. As a result, a lower ultimate lateral soil resistance was achieved compared to the uniform native ground. The partially drained condition magnified the pipeline-backfill-trench interaction effects with more significance in the deeply buried pipe. The current study revealed several important mechanisms, e.g., the significant influence of pipeline/trench-bed interaction on the mobilization of the lateral soil resistance not yet well addressed in scholarly researches.

Several new research avenues were identified, and the ground was prepared for improved and cost-effective design of buried pipelines in the near future.

1.2 Objectives

The main objective of the current study was to investigate the lateral soil resistance against the large displacements of the shallowly and deeply buried pipelines, through performing centrifuge tests and numerical studies. The key objectives of the study can be summarized as follows:

Experimental study:

1. Direct observation of the internal soil deformations and interactive failure mechanisms in the soil surrounding the laterally displaced pipeline.
2. Identifying how the pipeline-backfill-trench interaction contributes to the failure mechanisms and to the mobilized lateral soil resistance, consequently.
3. Obtaining the trenching/backfilling effects on the lateral load-displacement (p-y) curves, and the significance of simplifications that are currently applied by existing design codes.
4. Assessing the significance and practical implications of the pipeline-trenchbed interaction intensity on lateral soil resistance.
5. Evaluation of the effect of backfill shear strength that stemming from different types of the backfilling material on internal soil deformations, interactive failure mechanisms, and consequently the mobilized lateral soil resistance.

6. Assessing the burial depth effect on lateral response of trenched-backfilled pipelines.

Numerical study:

1. Developing an advanced numerical model for large deformation finite element analysis of lateral pipeline-backfill-trench interaction under undrained condition.
2. Verifying the model performance against the published and conducted experimental test results by comparing the failure mechanisms, pipe trajectory, and load-displacement responses in shallow and deep burial conditions.
3. Performing a comprehensive parametric study to investigate the influence of several key parameters including pipe roughness, pipe weight, initial pipe embedment into the trenchbed, backfill strength properties, soil strain-softening, native soil tension cut-off, and burial depth.

1.3 Organizations of the dissertation

The thesis has been prepared on manuscript-based format, comprising nine chapters. Overall the conducted work has been disseminated through five journal and three conference papers. Figure 1.1 illustrates the organization of the thesis with a brief explanation of each chapter given below.

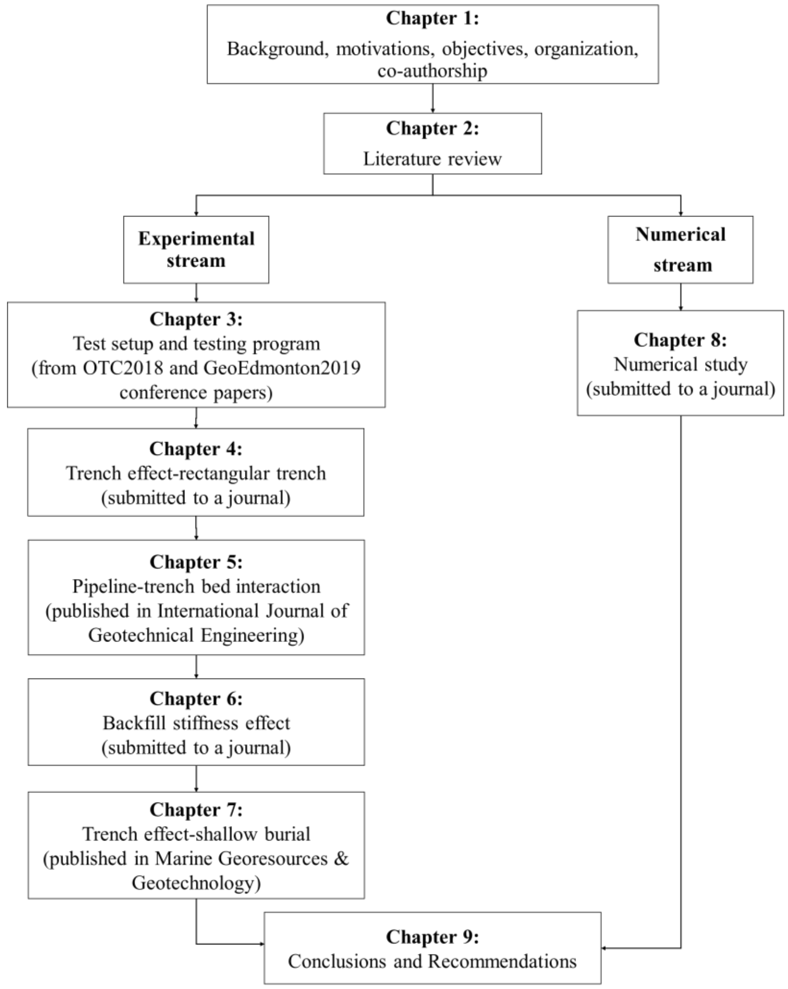

Figure 1.1. Structure of the dissertation

Chapter 1 introduces the research background, the motivation behind the research, the main objectives, novelty, significance, key contributions, as well as the organization of the thesis. Chapter 2 reviews the literature relevant to the study and the current guidelines and recommendations for practice. Relevant literature review is also included in each chapter depending on the chapter topic. Chapter 3 outlines the experimental procedure and testing program that is extracted from two published conference papers (i.e., OTC2018 and GeoEdmonton2019). Chapters 4 to 7 present the results of experimental study. Chapter 4 outlines the trenching/backfilling effect on lateral failure mechanisms around the pipeline deeply buried in clay. Chapter 5 focuses on the influence of pipeline-trenchbed interaction intensity on lateral soil resistance and failure mechanisms. Chapter 6 discusses the influence of different backfilling material and backfilling stiffness on the lateral pipeline response. Chapter 7 presents the trenching/backfilling effect on lateral failure mechanisms around the pipeline shallowly buried in clay. Chapter 8 contains the numerical study including model development, validations, and the results of parametric study. The thesis is closed with Chapter 9 outlining the conclusions and recommendations for future studies.Three conference papers are contained in the appendix A to Appendix C. Any reference to a figure or table with a designation beginning with a letter (i.e., Figure A.l) refers to a figure in that particular appendix (i.e., Appendix A).

1.4 Co-authorship statement

Morteza Kianian (PhD candidate) is the main author for all articles published/submitted within the current research project. Other co-authors are Dr. Hodjat Shiri (Supervisor), Mr. Mehdi Esmaeilzadeh (co-author in one journal and one conference paper), and Masih Allahbakhshi (Co-author in one conference paper). The experiments were mutually conducted by Morteza Kianian and Mehdi Esmaeilzadeh. Morteza Kianian was the main contributor to performing the post-processing of the test results and writing the first draft of all the papers and thesis. Moreover, the numerical simulations were solely performed by Morteza Kianian. Dr. Shiri supervised the project, provided funding, helped in developing the ideas for the experiments and finding knowledge gaps, review and editing the prepared manuscripts. Masih Allahbakhshi had contributed to the preparation of the model pipe and the test box during the initialization tests.

CHAPTER 2. Literature review

2.1 Introduction

New offshore hydrocarbon fields are being developed in deeper areas of the sea. At shallow water depths, the critical point in design is the pipeline lateral stability under current and wave actions. One of the solutions for protecting the pipelines in shallow waters is to bury the pipeline. While in deep water, it is very rare for pipelines to be trenched and buried. Instead, they are often laid on the seabed. In the exposed pipelines, the most critical design concerns are lateral buckling and axial walking (ratcheting) associated with thermal expansion and contraction of the pipeline with successive start-up and shutdown cycles (White and Randolph 2007). The existing literature presents more studies on exposed pipelines than on buried pipelines and also more emphasis on sands than on cohesive soils. Among those investigations around buried pipelines, most of the guidelines and studies address the upheaval interactions (Cathie et al. 2005). In the current chapter, the literature is briefly reviewed separately for exposed and buried pipelines. Common trenching and backfilling techniques are introduced and a discussion is then provided about their impact on the backfill properties and consequently on the pipeline response. Finally the approaches that are used for modelling and pipe-soil interaction are introduced.

- ***Exposed pipelines***

Pipelines are often laid over the seabed in deep areas of the sea. They extensively face cycles of thermal loading that causes axial stresses. The compressive axial stresses consequently lead to lateral buckling. A new cost-effective design method is to relieve the axial stress by controlling the formation of buckles along the pipeline. By laying the pipeline in a snaking pattern on the seabed, buckles are allowed to form at prescribed locations to relieve the axial stress. The imposed initial imperfections, as well as the seabed surface topography, govern the location and the size of the buckles that develop. These buckles must be engineered such that a sufficient length of pipe feeds into the buckle to relieve the axial stress, without generating excessive bending within the buckle. The typical lateral pipe movement within an engineered buckle is several pipeline diameters (Cheuk et al. 2007b). As opposed to exposed pipelines that are very vulnerable to lateral buckling, the buried pipelines are very prone to upheaval buckling. Buried pipelines feel a great lateral constraint (i.e., the trench wall) against the lateral buckling that leads them to buckle in the upward direction.

- ***Buried pipelines***

Pipelines are often buried especially in shallow waters for:

1. reducing the risk of hydrodynamic force and increasing the stability of the pipeline section,
2. protecting the pipeline section from the external damage due to anchors, heavy dropped objects or fishing gears,
3. improving pipeline structural performance, such as free span, lateral buckling, and insulation performance (Bai and Bai 2014).

Pipeline burial would generally provide far better protection than the exposed pipelines. But buried pipelines are vulnerable to the upheaval buckling. The most common pipeline trenching and burying equipment are jetting and plowing. Trenching can be performed before the pipeline lay, which is called pre-lay trenching. There is another methodology in which the after the pipeline is laid on the seabed the trench is excavated when the trenching device rides on top of the pipeline (post-lay trenching).

2.2 Trenching and backfilling

Subsea pipeline trenching practice has been developed in response to the need for protecting the pipelines against the geo-hazards and other external risks that threaten the integrity of pipelines in shallow waters. The burial of the pipelines in shallow waters is categorized into two types. First, trenching over relatively short distances in shore-crossing. This requirement has normally been met by some form of dredging technology, but sometimes by plows and jetting machines. Second, longer trenches in shallow waters of the open sea, which was constructed in the past by jetting, but in the past decades, plows and mechanical cutters have captured part of the market. This may have been a response to dissatisfaction with the high cost and limited protection given by trenches produced by jetting (Palmer and King 2008). Several systems are used to excavate the trenches in the seabed for submarine pipelines. In the following sections jetting and plowing are further discussed as they are the most common techniques in practice.

- ***Jetting***

In jet barge system, a jet sled is pulled along the pipeline by a barge. Water or air is pumped from the barge down hoses to the jets and erodes the seabed, forming a slurry of water and soil. A jet educator system ejects the slurry to one side. The sled carries instrumentation to monitor the forces between it and the pipeline. On the other hand, Jet machine is based on a self-contained machine, supplied with power from the surface by an electrical umbilical. Like the jet sled in the jet barge technique, the machine straddles the pipeline (Palmer and King 2008).

- ***Plowing***

Plows were used many years ago to trench pipelines but gained a bad reputation for poor depth control and sinking. Their modern development began in 1975, with a program targeted on the trenching of a loading line in the Statfjord (Palmer and King 2008). Since then, it has become a popular method of pipeline trenching. The general principle of pipeline plowing has been adopted from the technique used in agriculture to plow fields. The pipeline plow consists of a very large share, on top of which the pipeline rests. The pipeline pulled along (usually by the surface vessel), and as the plowshare passes, the pipeline settles in the trench. If a backfill plow is also employed, this reverses the process by pushing the spoil material back into the excavated trench. The main advantage of the plowing is that it is capable of post-lay trenching over a large range of pipeline sizes up to 24-inch diameter. This is probably the only technique that can dig the trench and backfill afterward in the same pass, which, in turn, reduces the costs dramatically. It should be noted that, however, some operators prefer rock or imported material to be used as backfill. The main disadvantage of this technique is that it has a limitation on the depth of excavation (up to 1.5 m). Other than that, the plowing system can cause damage to pipelines, especially those lines not protected by concrete coating (Bai and Bai 2014). Multi-pass plows have been built with the capability of achieving a trench depth of 2.5m (Paulin et al. 2014).

Pipeline burial can be performed by various trenching and backfilling methods, including the aforementioned methods in the above sections. A variety of factors should be considered to select a trenching method for a specific pipeline route. The water depth range, maximum required trench depth are the primary considerations. If multiple trenching methodologies satisfy the primary considerations, secondary considerations must be used to determine the preferred solution. These include parameters such as seabed geology, backfill method, seabed slopes, and environmental sensitivity (Paulin et al. 2014).

- ***Backfill properties***

Most of the technologies utilize the same technology to backfill the trench as how it was excavated. Following the pipeline installation, the excavated material is normally used to backfill the trench. The properties of the backfill placed in the trench are dependent on the selected method and the construction process of trenching and backfilling. The trenching and backfilling process remolds the excavated material and increases the water content of the backfill. As a result, the backfill will become of less strength and softer than the native ground, with a wide range of shear strengths ranging from negligible strength up to nearly the strength of native soil values. Depending on the trenching/backfilling technique and construction procedure, the backfilling material may be remolded to a different extent. Various backfilling material properties are expected depending on many parameters such as level of soil disturbance, size of clay lumps, potential high energy environment, whether the excavated spoil is left on the seabed or stored on land or barge, the period of exposure before placing in the trench, consolidation time after placing inside the trench, etc. For example, jetting requires cutting, erosion, and fluidization of the soil by the jets. Therefore, the water-clay suspension is expected immediately after jetting. The produced backfill in soft clay seabed is named slurry, which is of very low strength. The slurry regains the shear strength gradually from practically zero to that of a normally consolidated clay (DNV.GL 2017). The resulting backfill from jetting operation is generally homogeneous but jetting may introduce water-filled voids. Jetting in stiff clay may yield to a backfill consisting of lumps of semi-intact clay in a matrix of unconsolidated slurry.

On the other hand, plowing in soft clay results in heterogeneous backfill. The resulting backfill consists of softened and remoulded chunks close to the native water content in a slurry of much higher water content soil (Cathie et al. 2005). As a general rule, the backfill properties are a function of how the trench is excavated and backfilled.

- ***Influence of backfill properties on pipeline response***

The properties of the produced backfill, which is a function of many factors that have significant impacts on the pipeline response to vertical and lateral movements. Cheuk et al. (2007a) conducted a series of centrifuge tests to assess the vertical pressure exerted on a pipeline backfilled with lumpy clay when the pipe was moving upward at a constant velocity. Two different consolidation periods were considered to investigate the potential benefit of having a longer waiting period before putting the pipeline into operation. Results showed that early commissioning of buried pipelines in under-consolidated lumpy fill could lead to a reduction of soil restraint up to 56%, together with a decrease in the stiffness of the response. Bransby et al. (2002) conducted centrifuge tests to investigate the uplift capacity and the load-displacement behavior of pipelines buried in recently liquefied clay. They observed lower undrained uplift capacities than the drained capacities. The term recently liquefied clay means that it may still be consolidating when pipelines are commissioned. They finally proposed a simple method to predict uplift capacity from the average degree of consolidation of the backfill. Wang et al. (2009) also designed several centrifuge tests to measure the uplift resistance of a pipeline backfilled with blocky clay installed into stiff clay by trenching and backfilling. The uplift measurements were conducted approximately three months after installation.

There are a few experimental studies that have investigated the trenching and backfilling effect on large lateral pipe-soil interaction in clay. The most comprehensive experimental research is maybe the work conducted by Paulin (1998). This study was followed by the research conducted in the C-CORE centrifuge using the same methodology (C-CORE, 2003; Phillips et al., 2004). These studies were incorporated into the PRCI (2009) guidelines, which also present practical design recommendations for buried pipes in clays.

2.3 Modeling lateral pipe-soil interaction

A very common way of simulating the pipe-soil interaction is representation of the soil by a series of discrete springs that provide specified resistance per unit length of pipe (Winkler method). Figure 2.1 shows the soil loading on the pipeline that is represented by discrete bilinear or nonlinear springs. Finite element method is the other approach to analyze pipe-soil interaction problems. The maximum lateral soil force per unit length of pipe that can be transmitted to the pipe is:

$$P_{ult} = N_c \, c_u \, D + N_q \, \gamma \, H \, D \qquad (2.1)$$

where c_u is the undrained shear strength of the soil. N_c is horizontal bearing capacity factor for clay (0 for c = 0), and N_q horizontal bearing capacity factor (0 for zero friction angle). Both of the factors are provided by different expressions in ALA (2005) and PRCI (2009). ALA (2005) and PRCI (2009) suggested the following equation limiting the required displacement to reach P_{ult} to 10 to 15 percent of the pipe diameter.

$$\Delta_p = 0.04 \left(H + \frac{D}{2}\right) \leq 0.1D \ to \ 0.15D \tag{2.2}$$

Note that the equation (2.1) is recommended for the buried pipe in the uniform soil without considering the trench effect. Δ_p and P_{ult} are the coordinates of the ultimate point at the load-displacement curve. Bilinear or hyperbolic curve fits can represent the load-displacement curve. The general expression for the hyperbolic p-y relationship is provided here:

$$\frac{P}{P_{ult}} = \frac{y}{0.15\Delta_p + 0.85y} \tag{2.3}$$

where y is lateral displacement, and P is the lateral load on the Pipeline.

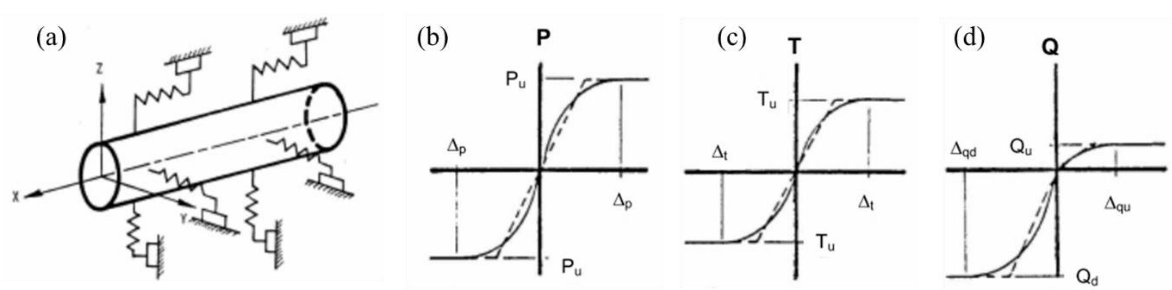

Figure 2.1. (a) Idealized representation of soil with discrete springs (b) transverse horizontal (c) axial (d) transverse vertical (after ALA 2005)

The conventional approaches for assessment of lateral pipe-soil interaction were mainly based on the earlier studies on plate anchors and piles that show a response similar to the pipeline (Hansen 1948, Hansen and Christensen 1961, Rowe and Davis 1982). In this kind of approach, the active and passive soil pressure based on simplified failure surfaces were used to calculate the soil loads on pipes in a closed-form solution. Corrective parameters such as aspect ratios and shape factors were used to equalize the geometrical configuration of pipes and anchors.

2.3.1 Buried pipelines

Buried pipelines may be subject to large forces under the effects of ground movement or large thermal loads. In the literature, most experimental pipeline studies were conducted in the sand. There is a very limited number of pipeline-specific theoretical and experimental models in the literature to predict the ultimate lateral resistance or force-displacement (p-y) curves for pipelines in clay. Many of the proposed models are based on anchor plates because they share behavioral characteristics with pipelines (Mackenzie 1955, Tschebotarioff 1973, Luscher et al. 1979, Rowe and Davis 1982, Das et al. 1985, Das et al. 1987, Rizkalla et al. 1992, Ranjani et al. 1993, Merifield et al. 2001). Many of the other solutions are developed based on piles (Hansen (1948), Poulos (1995), Hansen and Christensen (1961), Matlock (1970), Reese and Welch (1975), Bhushan et al. (1979), Edgers and Karlsrud (1982), Klar and Randolph 2008).

There is an extensive number of publications in the literature on studying the lateral pipe-soil interaction in cohesive and granular material, but only a few studies have investigated the trench effect on lateral response. In this section, for the sake of conciseness, only the key publications that have considered the trench effect on lateral pipe-soil interaction in clay were shortly reviewed. Also, whenever needed, references were made to some of the fundamental works on lateral pipe-soil interaction in uniform soils.

- ***Physical modeling***

Physical modeling is performed to investigate particular aspects of the pipe-soil interaction in prototype-scale. Full-scale testing is somehow the most costly way of physical modeling where all under investigation features of the prototype are reproduced at full scale. Most of the time, because of the convenience and cost-effectiveness, the physical models are constructed at much smaller scales than the prototype. If the model is not constructed at full-scale, then there is a need to transform the measurements in the model to the prototype scale. The concept of using centrifuge is born here, as we need to establish a similitude between the model and the prototype. Most of the difficulties associated with scaling can be avoided if the stresses at corresponding points in the model and the prototype are the same. Using centrifuge is a technique to generate the same levels of stress at the corresponding points in the model and the prototype. More details on centrifuge modeling can be found in Wood (2004).

Many of the proposed or potential models were originally developed for anchor plates and are useful for pipelines because of their relatively similar behavioral patterns. Mackenzie (1955) conducted small-scale model tests on rectangular and strip deadman anchors in a purely cohesive soil and proposed the “breakout factor” as a non-dimensional parameter to define the maximum capacity of anchors. Rowe and Davis (1982) studied the behavior of horizontal and vertical orientated anchor plates in cohesive soil theoretically. Das et al. (1985 and 1987) investigated the maximum pullout capacity of vertical anchors at different embedment ratios, ranging from 1 to 5. Other solutions have been developed based on piles. Hansen (1948) and Hansen and Christensen (1961) were based on piles subjected to lateral earth movements and broadly adopted after that by other researchers regarding lateral pipeline interactions. Most of the experimental studies in the literature regarding pipeline interactions have been conducted in the sand, and there is a very limited number of studies in a cohesive testbed.

Audibert and Nyman (1977) conducted a systematic experimental study on the lateral response of buried pipes. The study was conducted on loose and dense sands, but the initiated approach was used in cohesive soils in later studies as well. The authors proposed a hyperbolic curve fit to the experimental dimensionless load-displacement results. The proposed hyperbolic relationship was in close agreement with the earlier equation derived by Das and Seeley (1975) for lateral anchor response. However, the study did not consider the effect of pipeline-backfill-trench interaction on the lateral p-y response.

Wantland et al. (1979) investigated the effects of the pipe weight, diameter, burial depth, loading rate, and soil properties on lateral pipe resistance in soft clay by conducting a series of field and laboratory tests. Using an upper bound plasticity approach, the authors proposed an equation for ultimate lateral resistance with a load-bearing factor varying with H/D and an average cohesion in a distance of 2D above the pipeline invert. Ng (1994) compared the field test results of laterally loaded pipe in clay with the existing empirical equations (Rowe and Davis (1982) Randolph and Houlsby (1984)). The author noted the significance of pipeline-backfill-trench interaction in the evaluation of lateral response. Ng (1994) proposed modification factors (F_{ch} and F_{ac}) to incorporate the pipeline-backfill-trench interaction effects on load-bearing capacity (N_c).

Paulin (1998) conducted a series of centrifuge tests to investigate the lateral pipeline-backfill-trench interaction in the cohesive testbed. The key objectives of the program were to study the effects of trench width, burial depth, interaction rate, backfill properties, and the stress history of the soil on p-y curves. The authors observed that the pipeline-backfill-trench interaction mitigates the ultimate load applied to the pipeline. The magnitude of mobilized lateral soil resistance in partial drained and drained conditions was higher than undrained tests. PRCI (2003) and Phillips et al. (2004) reported other tests on lateral pipeline response using the same testing facility. The pull-out load results were presented in terms of the percentage of the maximum load, and no absolute value for the results was released.

Phillips et al. (2004) and C-CORE (2003) investigated the trench effect on soil resistance against pipeline lateral displacement by centrifuge experiments and numerical simulation. They adopted the same configurations as Paulin (1998). The authors observed that the presence of a trench mitigates the pipe response to lateral soil movement. A reasonable agreement was observed between the measured undrained interaction factors and those predicted by Hansen and Christensen (1961) and Rowe and Davis (1982) for uniform cohesive soil. Also, the authors showed that an increase in the trench width increases the required pipeline displacement to the ultimate lateral load. The study showed that the peak load occurs after the pipe touches the trench wall and is controlled by the native soil strength. Phillips et al. (2004) did not evaluate the magnitude of reduction in the pipe response due to the presence of trench and postponed it for future investigations. The undrained shear strength parameter is commonly used in design practice to assess pipe-soil interactions. However, this neglects the rate dependency of the pipeline response. In real conditions, drained or partially drained situations commonly occur, where the relative displacement rate between the pipe and soil is quite low. Under these conditions, the pore pressure would have enough time to dissipate during the pipe movement and achieve some level of consolidation. Moreover, in many geographical locations, silt fractions are found in natural offshore soft clays (e.g. Gulf of Mexico, Schiffman 1982). The presence of silt in clay increases the coefficient of consolidation of the soil and moves the loading condition toward partially drained and even fully drained. The coefficient of consolidation is highly variable and depends on the degree of initial consolidation and the load generating consolidation (PRCI 2009). These effects and their impact on the p-y response of the pipeline have not been well explored.

Oliveira et al. (2010) conducted 1-g and centrifuge tests using a type of clay obtained from Guanabara Bay at shallow burial depth ratios (lower than 200%). Based on the obtained visualized displacement fields, the research group assumed two simplified circular failure surfaces, which were defined only by geometrical parameters, and proposed Eq. (2.4) for a normalized horizontal force which, in shallow depths, is in agreement with Eq. (2.5), recommended by ALA (2001). The Eq. (2.4) is found as a function of (H/D), and curve fitting parameters ‘a’ to ‘d’ are respectively 6.752, 0.065, -11.063, and 7.119.

$$N_c = 5\ \text{arctan}(\frac{H}{D}) \tag{2.4}$$

$$N_c = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3}\ \leq 9 \tag{2.5}$$

- ***Numerical modeling***

There is a fewer number of studies on the lateral response of buried pipelines in clay in comparison with exposed pipelines (Ng, 1994; Audibert and Nyman, 1977; Paulin, 1998; Oliveira et al., 2010; C-CORE, 2003; Phillips et al., 2004). Merifield et al. (2001) used two numerical procedures that were based on finite element formulations of the upper and lower bound theorems of limit analysis to propose lower and upper bound equations for clay resistance in vertical plate anchor. The equation for ultimate lateral resistance of a plate anchor in a uniform cohesive soil is:

$$P_y = N_{co} D c_u + \gamma D H \tag{2.6}$$

where H is the burial depth to the center of the pipe, γ is the unit weight of soil, and the term N_{co} depends on the burial depth of the pipeline and is given as follows:

$$N_{co} = 2.46\ ln\left(\frac{2(H + D/2)}{D}\right) + 0.89 \quad \textit{Lower bound} \tag{2.7}$$

$$N_{co} = 2.58\ ln\left(\frac{2(H + D/2)}{D}\right) + 0.98 \quad \textit{Upper bound} \tag{2.8}$$

$$N_{co} + \frac{\gamma' H}{c_u} \leq 10.47 \tag{2.9}$$

The limiting value of 10.47 in Equation (2.9) reflects the transition from shallow to deep behavior which depend not only on the burial ratio but also on the ratio of $\gamma' H/c_u$. This implies that if the ratio of $\gamma' H / c_u$ is large enough, then the pipe will behave as a deeply buried pipeline.

- ***Theoretical approaches***

The basic theoretical solutions are based on (1) earth pressure theories, (2) bearing capacity theories; and (3) P-y curves based on a beam on elastic foundation theory. The earth pressure theory concept has typically been adopted for pipelines despite the differences in geometry of the problem, principally that the pipeline does not extend to the ground surface in the manner of a wall, and the pipeline surface is curved rather than flat. Bearing capacity theories were initially developed for loads applied downward onto a half-space of soil (ASCE committee 2014).

There are some guidelines based on the previous researches and empirical equations for lateral pipe-soil interaction by ALA (2005), PRCI (2009), ASCE committee (2014). Almost all of the recommended formulations lack the trench effect in their equations for lateral interaction. PRCI (2009) and DNV.GL (2017) noted the effect on the lateral response of the pipe without implementing it on their proposed formulations. PRCI (2009) recommended making use of the strength properties of backfill until the relative horizontal displacement between the pipeline and the soil exceeds the distance between the pipe and the trench wall. DNV.GL (2017) has implemented the trench and backfill effect into the uplift guidelines.

2.3.2 Exposed pipelines

There are numerous researches on the lateral response of pipelines laid on soft clay seabed using analytical (Randolph and White, 2008), physical (Dingle et al., 2008; Cheuk et al., 2007) and numerical (Merifield et al., 2008; Merifield et al., 2009; Dutta et al., 2015) modeling.

- ***Physical modeling***

Cheuk et al. (2007b) conducted a series of full-scale plane strain tests for pipe penetration into a very soft clay testbed (Kaolin clay). They also proposed a simple upper bound solution to model the observed response. The solution was able to capture the observed experimental trends, including the growth of the active berm and collection of dormant berms. This approach was the first attempt to quantitatively model the mechanisms underlying the response during large-displacement lateral sweeps of an on-bottom pipeline, accounting for the growth of soil berms. Dingle et al. (2008) performed centrifuge tests and used the Particle Image Velocimetry technique. They correspondingly observed the soil deformations along with the pipeline vertical/lateral load-displacement response and pipe trajectory to get further insight into the soil flow mechanisms during pipe penetration into the very soft clay testbed.

- ***Numerical modeling***

Pipeline penetration into the seabed and lateral breakout have been investigated numerically by many researchers. Merifield et al. (2008) used finite element of the shallowly embedded pipeline to investigate the response to vertical and lateral loading. These analyses have been compared with collapse loads calculated using the upper-bound theorem of plasticity and were used to construct yield envelopes defining the limiting combinations of vertical and horizontal load. Dutta et al. (2015) evaluated the competency of the Coupled Eulerian-Lagrangian method for modeling the partially embedded pipelines. The results were also compared with the previous centrifuge tests conducted by Dingle et al. (2008).

- ***Theoretical approaches***

Randolph and Houlsby (1984) proposed a plasticity solution for the laterally loaded piles. Assuming a perfectly plastic cohesive material, the calculation of ultimate lateral resistance at depth reduces to a plane strain condition in plasticity theory, in which the load is calculated on a long cylinder, which moves laterally through the infinite medium. With these assumptions, they proposed the exact non-dimensionalized ultimate resistance (that is called load factor) for perfectly rough and smooth piles. Randolph and White (2008) then proposed a yield envelope using Martin’s mechanism, which has been already developed by Martin and Randolph (2006) for both uniform and linearly varying undrained shear strength profiles.

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CHAPTER 3. Test setup and testing program

3.1 Introduction

The previous chapter presented a literature review on modeling strategies of lateral pipe-soil interaction considering buried and exposed pipelines. In this chapter, the first section overviews centrifuge modeling. Then experimental procedures and testing details are presented, including soil sample preparation, model preparation, and installation of instruments. Some details are then provided on the soil preparation, facilities, equipment, instrumentation, visualization system. During the experiments where necessary, modifications were made to improve the experimental setup and test conduct as the experimental program progressed.

Buried pipelines may be subject to large bending and tensile loads under the effects of ground movement or large thermal loads. Ground movement can arise from differential soil settlement, fault displacement or lateral spread displacement in earthquakes, landslide displacement, frost heave or thaw settlement, etc. (ALA 2005). Figure 3.1 shows the pipeline bending zone under the strike-slip fault movement and the resulting lateral distributed load on the free body diagram of the pipeline.

3.2 Overview of centrifuge modeling

Using centrifuge for modeling gravity-dependent problems has been proven to be reasonably an efficient technique. Centrifugal acceleration is used to simulate gravity

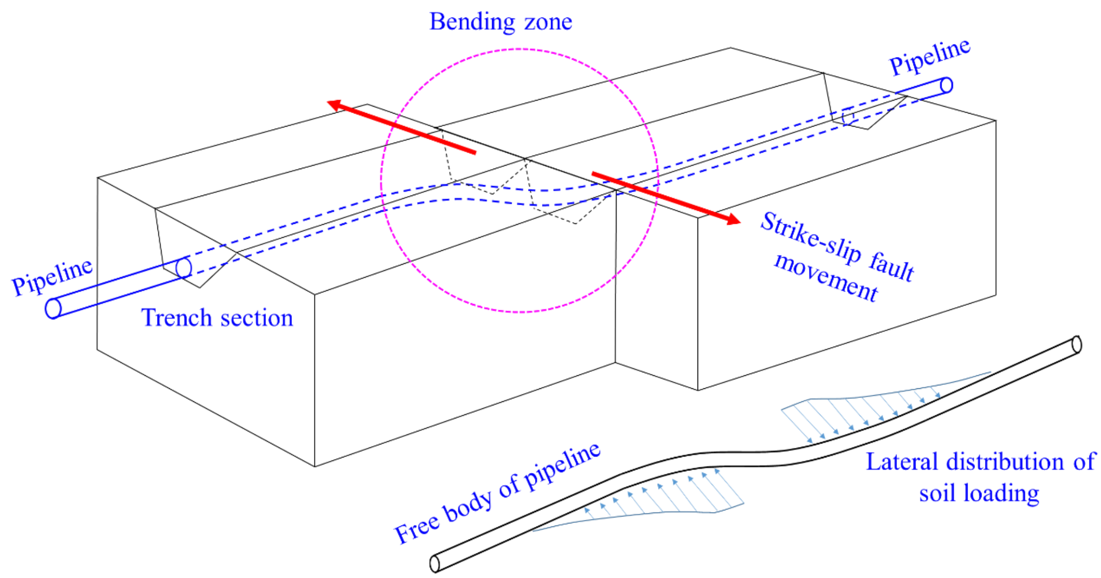

Figure 3.1. Buried pipeline under strike-slip fault movement and lateral distribution of soil loading

and facilitates correspondence of stress levels between model and prototype. The same levels of stress at the corresponding soil depth provide similitude in other parameters between model and prototype by scaling laws. Therefore, centrifuge modeling is offering a capability for accurate modeling of geotechnical phenomena. Such accurate modeling increases general understanding and permits calibration and verification of other numerical and theoretical models (Paulin 1998). Note that there may be some parameters or processes in the centrifuge modeling that could not be scaled by the gravity level. If those parameters were significantly important in the results of the model, then the results obtained from the centrifuge modeling would not be acceptable or at least accurate. Wood (2004), in a chapter, has discussed the centrifuge modeling and centrifuge model preparation. He also has reviewed the dimensional analysis in an individual chapter and described the scaling laws for centrifuge and small-scale modeling.

In this study, a centrifuge testing program was conducted to investigate the response of buried pipelines to large lateral displacements. The interactive and progressive failure mechanisms both in the backfilling and the native soil were obtained through direct observation from a transparent sheet mounted in the sidewall of the test-box. A range of tests was conducted using a fully stocked test instrument set up to capture the influence of various parameters including the undrained shear strength of the backfill and the native soil, trench geometry, burial depth.

3.3 Testing program

The testing program comprised five series of tests involving the lateral pipeline-backfill-trench interactions in clay during large lateral displacements (up to 4D) at a centrifuge acceleration of 19.1g. Two similar pieces of pipe with different configurations were pulled in opposite directions and tested in each run resulting in ten tests in total. In addition, three series of tests (six pipe tests) were conducted in dry loose sand. However, the main purpose of the project was conducting tests in the cohesive testbed, and the preliminary sand tests were only set to calibrate the test setup. The details of the interactive failure mechanisms were directly monitored from a transparent observation window mounted on the side of the test box. Two digital cameras were used to capture high-quality images for post-processing and particle image velocimetry (PIV) analysis. In each clay test, the fully instrumented model pipe sections were located on the bottom of the excavated trenches and backfilled with different backfilling materials. Two vertical actuators with pulleys and horizontal cables were used to pull the pipes in opposite directions with pre-determined velocities, while the pipes were free to move vertically over the initial course of displacement. The testing schedule was defined to maximize the amount of required high-quality data obtained. Table 3.1 gives a summary of the conducted testing program.

Table 3.1. Summary of the conducted testing program

Test	Pipe	Test name	Burial depth ratio (H/D)	Trench backfill type	Trench wall	Model displacement rate (μm/s)	Normalized velocity vD/c_v	Normalized pulling distance
Test 1	Pipe 1	T1P1	3.5	Chunk	Inclined (32°)	8.96	0.43	2.61
	Pipe 2	T1P2	3.5	Slurry	Vertical	9.09	0.43	3.03
Test 2	Pipe 1	T2P1	3.5	Loose sand	Vertical	9.29	0.43	3.6
	Pipe 2	T2P2	3.5	Slurry	Inclined (62°)	9.16	0.43	3.5
Test 3	Pipe 1	T3P1	1.5	Slurry	Vertical	9.44	0.43	3.93
	Pipe 2	T3P2	1.5	Chunk	Inclined (36°)	9.23	0.43	3.82
Test 4	Pipe 1	T4P1	1.5	Slurry	Vertical	3	0.14	3.93
	Pipe 2	T4P2	1.5	Chunk	Inclined (32°)	3.01	0.14	3.87
Test 5	Pipe 1	T5P1	3.5	Slurry	Vertical	2.98	0.14	3.71
	Pipe 2	T5P2	3.5	Chunk	Inclined (38°)	3.01	0.14	3.85

Figure 3.2 shows the contribution of each experiment in the relevant chapters of the thesis.

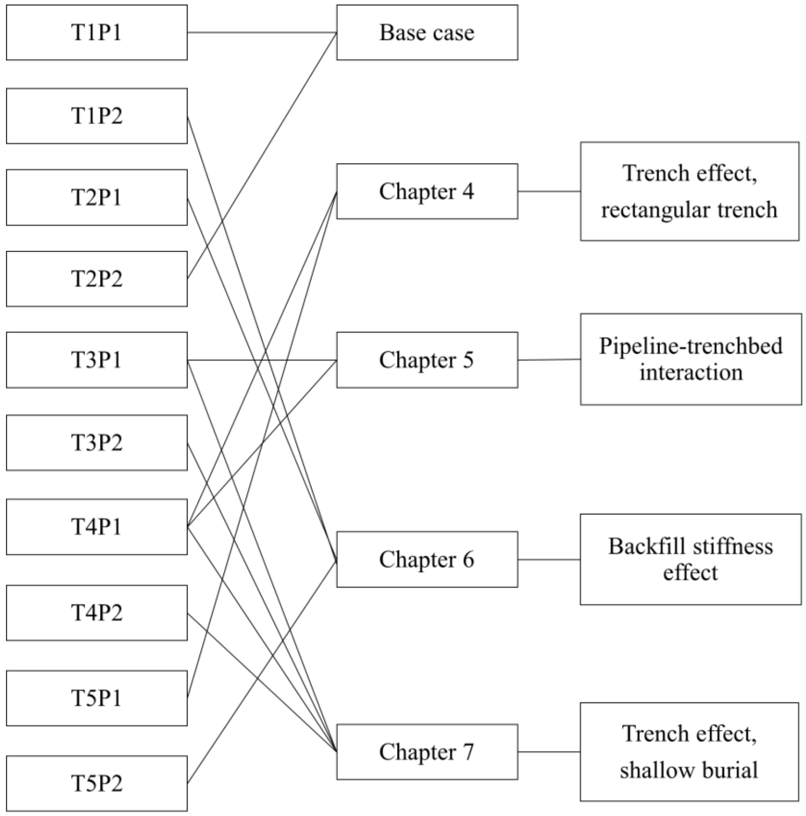

Figure 3.2. Contribution of tests in different chapters

3.4 Experimental setup and testing procedure

3.4.1 Modeling considerations

The main objective of the testing program was to investigate the pipeline-backfill-trench interactions and its impact on the force-displacement responses of pipelines during large lateral deformations. For this purpose, it was essential to monitor the interactive and progressive soil failure mechanisms around the pipe and interpret its impact on the measured p-y responses and the ultimate loads exerted on the pipelines. Therefore, a plane-strain container with an Acrylic side window was used to monitor the failure mechanisms for further PIV analyses. The plane strain assumption originated from a practical fact that the lateral and vertical displacement of the pipeline takes place over a very long section of the pipeline. The effects of variations in burial depth, trench geometry, and backfill properties were other objectives of this study to ensure that the results could be confidently scaled up to full-scale conditions. Figure 3.3 shows a sample schematic view of the test setup, where two pieces of model pipes were backfilled inside excavated trenches in a pre-consolidated soil bed and pulled apart over large displacements (3-4D) using horizontal cables driven by vertical actuators. Figure 3.3 illustrates the boundary conditions normalized to the pipe diameter using dotted circles.

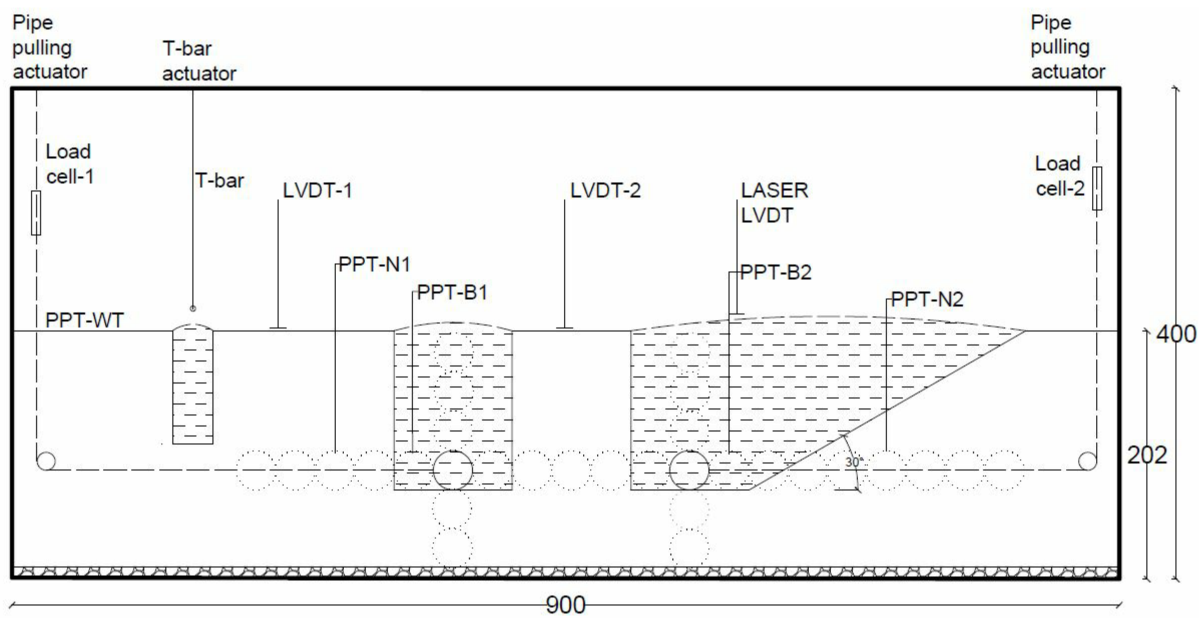

Figure 3.3. Schematic view of test setup (cohesive testbed); Instruments are coded; all dimensions are in mm

The soil sample was consolidated to effective stress of 400 kPa and was unloaded sequentially. This level of consolidation yielded soft clay with an undrained shear strength profile in native soil (15-25 kPa). Three main types of backfill with various geomechanical properties were developed to model the significant difference between the strength of the native material and the backfill. The model pipe size was dictated by the dimensions of the internal pore pressure transducers that had to be incorporated inside the pipe to measure the pipe-soil interface pressure or suction in the rear of the pipe during pipeline displacement. The minimum possible bending radius of the cable connected to the pressure transducer imposed a minimum nominal pipe diameter of 32 mm to accommodate the transducer. The acceleration level was set to about 19.1g to model a real pipe diameter 610 mm, as targeted by the industry sponsor. This pipe size was the same as the earlier tests conducted in sand (Burnett 2015), representing the size range of export pipelines. Different burial ratios (H/D) ranging from 1.4 to 3.8 were tested to ensure covering shallow to deep burial conditions. Rectangular and trapezoidal trenches were considered with a fixed bottom width of 3D and top with varying from 3D to 10D depending on the side angle of the trench wall (90°, 60°, and 30°). The trench wall behind the pipe was kept vertical, assuming a minor effect on the lateral pipe response moving in opposite directions.

A range of instruments was used to monitor the testing program, such as pore pressure transducers (PPTs), strain gauges, load cells, linear variable differential transformer (LVDTs), T-bar, actuators and vertical drive motion controllers, digital cameras, markers, and artificial textures.

3.4.2 Soil preparation

Different procedures were used to prepare the native soil bed and various backfilling materials to simulate realistic field conditions. A mixture by weight of 50% white kaolin clay and 50% Sil-Co-Sil silt was added by a sufficient amount of water to form a slurry with a nominal moisture content of 70%. The mix was left for an hour or some to completely soak before mixing for about a half-hour followed by 3 hours mixing under a vacuum of 60-70kPa for de-airing. The mixture was poured into the container, closely observing to ensure it is homogeneous and free of lumps. The container was placed in the consolidometer and the top edge was checked and leveled to be horizontal. Incremental loads were applied to the soil over a week or so and directly monitored by the load-cell of a hydraulic jack.

After achieving the desired stress level (400 kPa), the soil sample was sequentially unloaded up to 100 kPa with an open drainage valve. Below 100 kPa, the flow of water into the sample was restricted by closing the base drain and removing excess water at the soil surface. After removing the box from consolidometer, the removable sidewall of the box was removed by sliding parallel to the opposite sidewall. Before installing the transparent window, the exposed side surface of the soil sample was artificially seeded by dark Frasier river sand using a regular salt pourer. This texture provided by artificial seeding allows both macroscopic and grain-scale deformation features to be identified by PIV analysis (Stanier and White 2013). The Acrylic sheet was carefully installed on the side of the box with a face-to-face approaching direction.

3.4.3 Trenching the soil bed

Shaving blades with desired side angles were used to cut the trenches and T-bar site. Shaving blades were attached to an adjustable shaft traveling inside a horizontal guide frame mounted on the top edge of the box (Figure 3.4). Samples were extracted from shaved material to determine the average water content. The height of the shaving arm was adjusted to ensure that the spring line of the pipe will be at the desired elevation from the prepared bottom of the testing box. To locate the pulling cables, 3 mm wide openings were created using narrow steel blades. The desired dimensions of the trenches were controlled by using marks on the internal surface of the steel rear wall and direct measurements through the transparent front wall. Figure 3.5 and Figure 3.6 show a sample of excavated soil bed, where trenches with vertical and inclined walls have been tested. The trench depth was kept the same for both of the pipes in a test. Trenches with three different side angles were created (i.e., 30°, 60° and 90°). To better simulate the real condition, the surfaces of the trench walls and trench bottom was slightly patterned using a wet canvas to prevent having a slippery smooth surface between the trench and backfill.

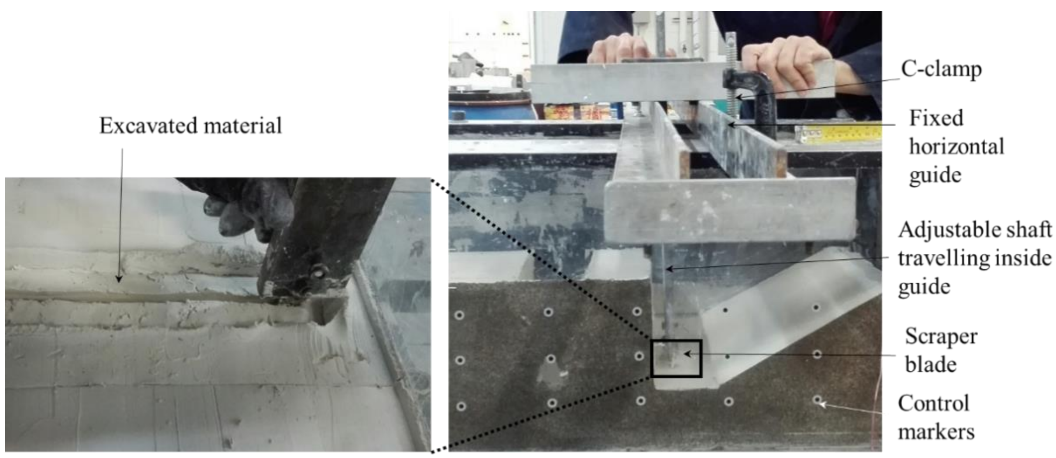

Figure 3.4. Excavating trench bottom using blade

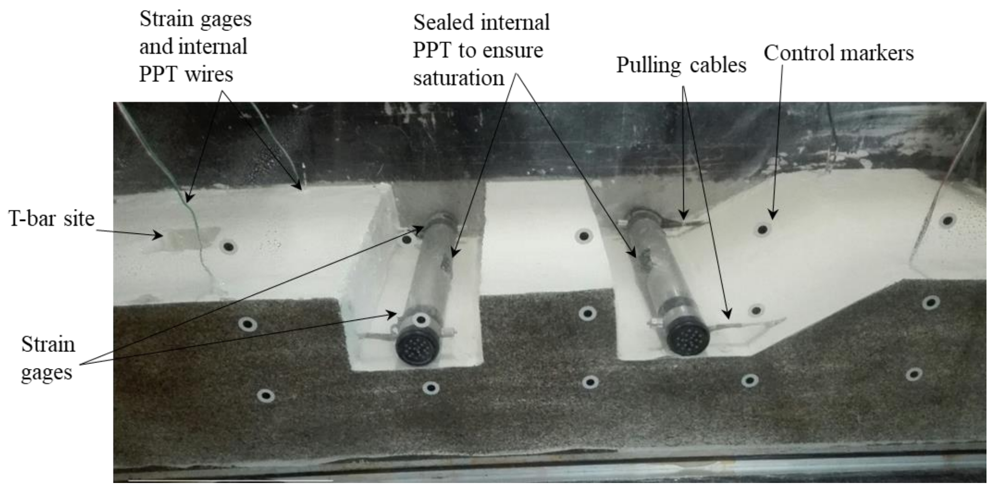

Figure 3.5. Box front view; pipes installed inside two excavated trenches before backfilling

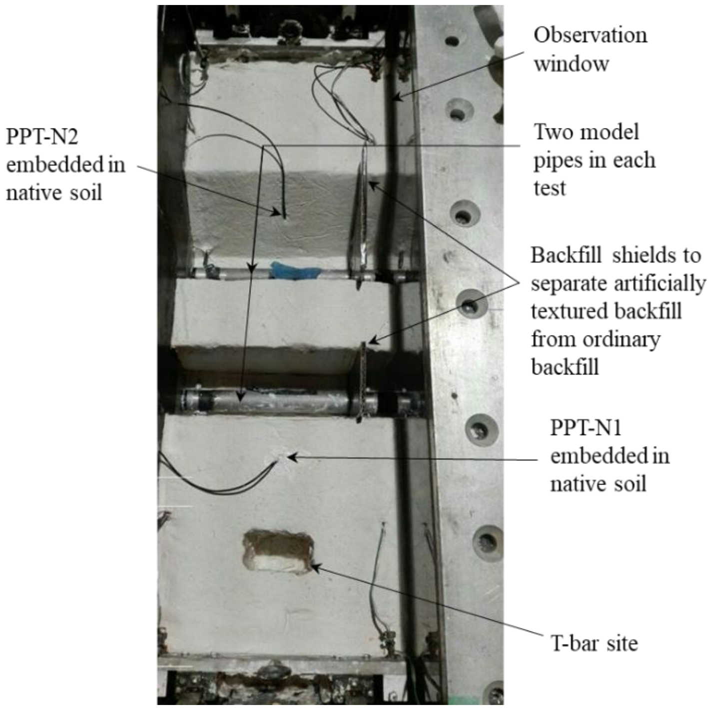

Figure 3.6. Top view of the instrumented box before backfilling

3.4.4 Backfilling material

The excavated material is usually used for backfilling the trenched pipeline. Depending on the trenching and backfilling technique, and construction condition, the backfilling material may be remolded to a different extent. Various backfilling material properties are expected depending on many parameters such as level of soil disturbance, size of clay lumps, potential high energy environment, whether the excavated spoil is left on the seabed or stored on land or barge, the period of exposure before placing in the trench, consolidation time after placing inside the trench and etc. In this study, in addition to silica sand, a range of cohesive backfills was reproduced from shaved native material including very soft slurry and chunk materials with various strengths. Different preparation methods were used to model a range of backfilling conditions and backfill properties. This enabled the preparation of fairly soft backfills representing the strength difference between the real native soil and backfill material. Table 3.2 shows the summary of the testbed and backfilling material.

Table 3.2. Soil properties of the cohesive testbed

Test	Pipe	Test name	Trench backfill type	Trench backfill ID	T-bar site backfill	T-bar site backfill c_u (kPa)	Native c_u at pipe depth (kPa)	Native soil water content before test (%)	Native water content after test (%)	Native soil void ratio	Saturated unit weight Υ_{sat} (kN/m^3)
Test 1	Pipe 1	T1P1	Chunk	T1B1	Slurry	<< 1	16 - 19	32.04	32.97	0.864	18.33
	Pipe 2	T1P2	Slurry	T1B2							
Test 2	Pipe 1	T2P1	Loose sand	T2B1	Chunk	2 - 3.7	16 - 19.5	30.81	31.11	0.815	18.56
	Pipe 2	T2P2	Slurry	T2B2							
Test 3	Pipe 1	T3P1	Slurry	T3B1	NA	NA	17.5 - 20	31.24	31.47	0.825	18.51
	Pipe 2	T3P2	Chunk	T3B2							
Test 4	Pipe 1	T4P1	Slurry	T4B1	Slurry	<< 1	17.5 - 20	31.99	31.98	0.838	18.45
	Pipe 2	T4P2	Chunk	T4B2							
Test 5	Pipe 1	T5P1	Slurry	T5B1	Chunk	2.5 - 4.5	17 - 20.5	30.12	32.13	0.842	18.43
	Pipe 2	T5P2	Chunk	T5B2							

3.4.4.1 Slurry

To investigate the influence of different backfills on the pipeline response, a trenched but unburied base case was required. In reality, the trench may be naturally filled with fine sediments under the environmental loads action in the relatively shallow water, where seabed currents are sufficient to induce transport (Cathie et al. 2005). Also, the excavated material deposited into the spoil heaps and then left exposed to free water for a long period before backfilling causes the soil to become fluidized and produce a slurry. This kind of natural backfill is a soft slurry that has no or very low strength. A mixture of shaved native soil material and the water was used to create the backfilling slurry with water content about 100%, which is about three times the liquid limit of the native soil. The in-flight T-bar test showed almost zero undrained shear strength after inflight consolidation. However, the test results showed that despite low strength, a mechanism called pipeline-trenchbed interaction contributes to the response. Kianian and Shiri (2019) which is chapter 5, has more details on this mechanism. Figure 3.7. shows a top view of the backfilled soil sample.

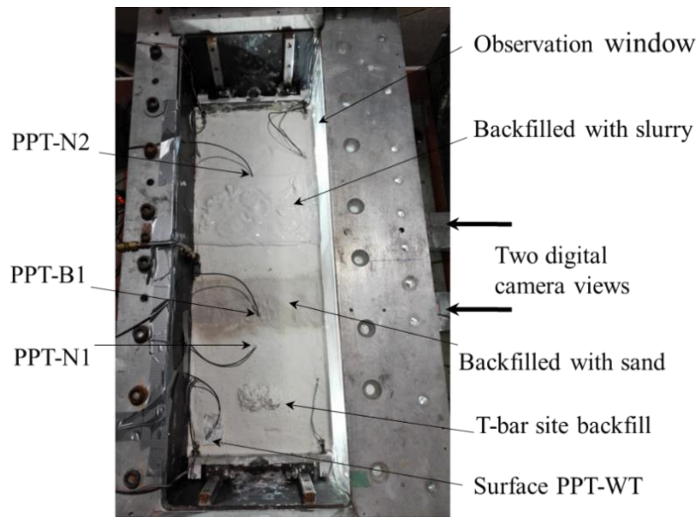

Figure 3.7. Top view of the instrumented box after backfilling

3.4.4.2 Chunk of native soil

The chunks of around 25 mm were excavated from native soil and exposed to water for several hours. This backfill was heterogeneous and consisted of softened and remolded or semi-remolded chunks. The water content was kept slightly higher than the in-situ consolidated soil. The preparation process of this backfilling type can simulate the jet cuttings excavated and deposited inside the trench in a matrix of slurry while using the jetting technique. This backfill can also be taken as an attempt to model the backfills produced by mechanical excavation or backfilling techniques like plowing, backhoe and clamshell bucket. Four different chunky materials were produced and tested in this program.

3.4.4.3 Silica Sand

The granular purchased material may be used for backfilling of the pipelines in many cases. Fine Silica sand (D60 = 0.205 mm; D30 = 0.14 mm; D10 = 0.103 mm.) was used as backfilling material in one test (T2P1) to investigate the pipeline response surrounded by granular cohesionless materials. The silica sand was poured inside the trench after locating the pipe. The sand backfill achieved an extent of densification by water filling the test box and in-flight period for consolidating native soil.

A T-bar penetrometer (Stewart and Randolph 1994) was used to obtain the undrained shear strength profile of the native and backfilling material. A T-bar bearing factor of 10.5 was considered for deep penetrations. But for shallow depths, a reduced bearing factor arising from the soil buoyancy and shallow failure mechanism mobilized before the full flow of soil around the bar (White et al. 2010) was used to translate the measured bearing resistance to the undrained shear strength.

3.5 Instrumentation

The model pipe, backfilling and native soil was fully instrumented to ensure sufficient and reliable data will be recorded during the testing program. Table 3.3 provides more detailed information about the test instrumentation.

Table 3.3. Test instrumentation

Instrument name	Location	Description	Total number used per test
Internal PPT	Inside the pipe sensing the rear of pipe pore pressure	Druck PDCR81	1 per pipe
PPT holder, water plug and O-rings	Inside the pipe	Nylon	1 per pipe
Pore Pressure Transducer (PPT)	In backfill and native soil and at surface of soil	Druck PDCR81	2 per pipe
Strain gauge	At reduced section of pipe. One full Wheatstone bridge	Shear gauge which has been calibrated to shear force at reduced section of pipe	2 per pipe
Load cell	Connected to pulling cable measuring total pulling force including all frictions	3.5 kN capacity	1 per pipe
T-bar	T-bar site	Head bearing area: 30×7.4 mm^2	1 per test
Digital camera	In front of the viewing window	10.10 megapixel	1 per pipe
LVDT	Native soil surface	Linear Variable Displacement Transducer	2 per test
Laser LVDT	Backfill surface	There was malfunction because passing through water	1 per test
Control marker	Inner side of transparent window	Inner circle diameter: 6.27 mm; Outer diameter: 12.24 mm	18 per test
Sand for artificial seeding	Sprinkled on native soil and mixed with backfill just beside the window	Fraser River sand	NA
End caps & O-ring	The end of the pipes	Nylon	2 per pipe

Miniature pore pressure transducers (PPTs) were used to record the pore pressure variation in different spots of the test box. The internal PPT was installed inside the pipe facing the rear of the pipe to measure the suction force mobilization behind the pipe during the displacement. The curvature of the data acquisition cable connected to this PPT dictated the minimum diameter of the model pipe (i.e., 31.75 mm). Each backfill material equipped with one PPT and two more PPTs was installed in native soil with the locations shown in Figure 3.3. The external PPTs were kept in position using supports on two I-beams carrying the actuators. These external PPTs were used to monitor the state of soil equilibrium assessing the soil drainage conditions under various pipeline displacement rates throughout the moving path. The external PPTs could be also used for monitoring the variation of the water table.

The strain gauges were installed in the reduced cross-section of the pipes to capture the lateral pipe response (Figure 3.8). The strain gauges were calibrated to measure the shear force at the reduced sections. Calibration factors were extracted by a simple analysis of load distribution along the pipe.

In addition to direct monitoring of surface variation of the soil surrounding the pipes via acrylic sheet, appropriate numbers of linear variable displacement transformers (LVDTs) were also used to measure the soil surface movement. The measuring shafts of the LVDTs rested on Plexiglas pads. These pads were penetrating into the slurry backfill with low strength, so laser LDVTs were replaced in the tests with slurry backfill. The clarity of the filled water inside the test box was not sufficient for traveling the laser beam and recording the surface movements.

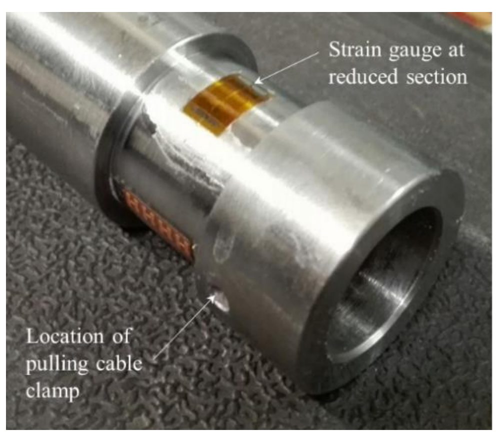

Figure 3.8. Shear strain gauge installed at reduced section

3.6 Visualization and monitoring

Two Canon EOS DIGITAL Rebel XTi still cameras operating in continuous shooting mode were used to capture images of the moving pipes end cap and surrounding soil through the observation window. Each camera was intended for one pipe individually. Two cantilever beams fixed the cameras to the centrifuge swinging platform. Tight cables were used at the end of cantilever beams to secure the cameras at a higher g-level.

The acrylic window on one side of the test box enabled the direct recording of soil failure mechanism, pipe trajectory, and lateral pipe response. The continuously captured high-quality images were used in particle image velocimetry (PIV) analysis to measure the displacements and obtain strains at any point observable from the window.

The PIV analysis was conducted using GeoPIV software originally developed by White et al. (2003) and further developed by Stanier et al. (2016), where the subsets of the image field were tracked and compared with the reference image as the pipes were being pulled. Black and white circle markers with the dimensions and layout shown in Figure 3.5 were attached to the transparent window as the reference points in PIV analysis. Because of physical limitations in testing facilities and the actuators, the digital cameras couldn't be synchronized and moved with the movement of the pipe. To limit the slight effect of varying observation sight over the large lateral displacement in PIV analysis, a calibration sheet was used. This enabled the correction of image distortion because of noncoplanarity of the images and object planes, and the nonlinear fish-eye and barrelling effects. During the tests with model pipe nominal moving velocity of 0.01 and 0.003 mm/s, 25 and 83 second shutting intervals were used to capture images at 0.25 mm increments which is appropriate relative to total displacement domain and ensure sufficient capturing of the soil failure mechanisms.

3.7 Preliminary sand tests for setup calibration

The preliminary experiments were conducted in dry sand to find out the possible bugs in the test configuration. As the consolidation process in clay is time-consuming, the preliminary tests were chosen to be conducted in dry sand to make sure that the test configuration will properly work for clay.

3.7.1 Testing setup and procedure

The testing program consisted of three series of tests engaging the pipeline-soil interaction in dry sand through large lateral displacements. The buried pipes were pulled in opposite directions over a large course of displacements (2.5 to 3.0D). In tests with deferent g-levels, the pipes were pulled in two individual stages. The sand was placed inside the box without any densification process. However, there would have been some slight levels of densification during the loading of the box onto the platform and during centrifuge running.

Two model pipes were pulled in opposite directions and tested in each run resulting in six sand tests in total. The instrumentation of pipe-2 was not ready at the time of testing, therefore, the results of pipe-2 were not measured during the experiments. The assumed uniform lateral distributed force due to pipe-soil interaction was obtained using two shear strain gauges which were installed at two sections of the pipe. These two strain gauges measured all the shear force developed between the locations of the strain gauges. The internal dimensions of the testing box were 0.9 m by 0.3 m wide by 0.4 m high. The testing box was designed to simulate plane strain conditions, as an infinitely long buried pipeline would experience similar conditions in the field. Both pipe and sand are restrained at two sides of the box and during lateral movement of the pipe, sand cannot flow out of the plane. The testbed was prepared using silica sand. Table 3.4 shows some details of the preliminary tests. The sand particle size analysis shows that the sand is poorly graded, having D_{50} = 0.19 mm and coefficient of uniformity C_u = 1.96.

Table 3.4. Summary of the testing program

Test	Pipe	Test ID	Scale	Model pipe diam (mm)	Prototype pipe diam (mm)	Prototype depth (m)	Burial ratio, H/D	ϒ (kN/m3)	Confining pressure (kPa)	Resistance (kN/m)	Normalized resistance
Test 1	Pipe 1	T1P1	19.06	31.75	605.2	1.20	2.0	13.5	16.24	63.93	6.50
	Pipe 2	T1P2	19.06	31.75	605.2	1.20	2.0	13.5	16.24	-	-
Test 2	Pipe 1	T2P1	19.06	31.75	605.2	0.60	1.0	13.5	8.15	33.69	6.83
	Pipe 2	T2P2	7.95	31.75	252.4	0.25	1.0	13.5	3.40	-	-
Test 3	Pipe 1	T3P1	7.95	31.75	252.4	0.72	2.8	13.5	9.66	20.03	8.21
	Pipe 2	T3P2	19.06	31.75	605.2	1.72	2.8	13.5	23.16	-	-

3.7.2 Comparison with published studies

Figure 3.9 compares the test data with some other experimental studies and guidelines including, ALA (2005), PRCI (2009) and ASCE committee (2014). All the data presented in Figure 3.9 are selected from the previous experiments executed in loose sand testbeds. ALA (2005) predicts closer results to the present experiments. There are many sources of discrepancies, including friction angles, sand types, sand densities, and experimental procedures associated with test setups and their side effects on the produced results.

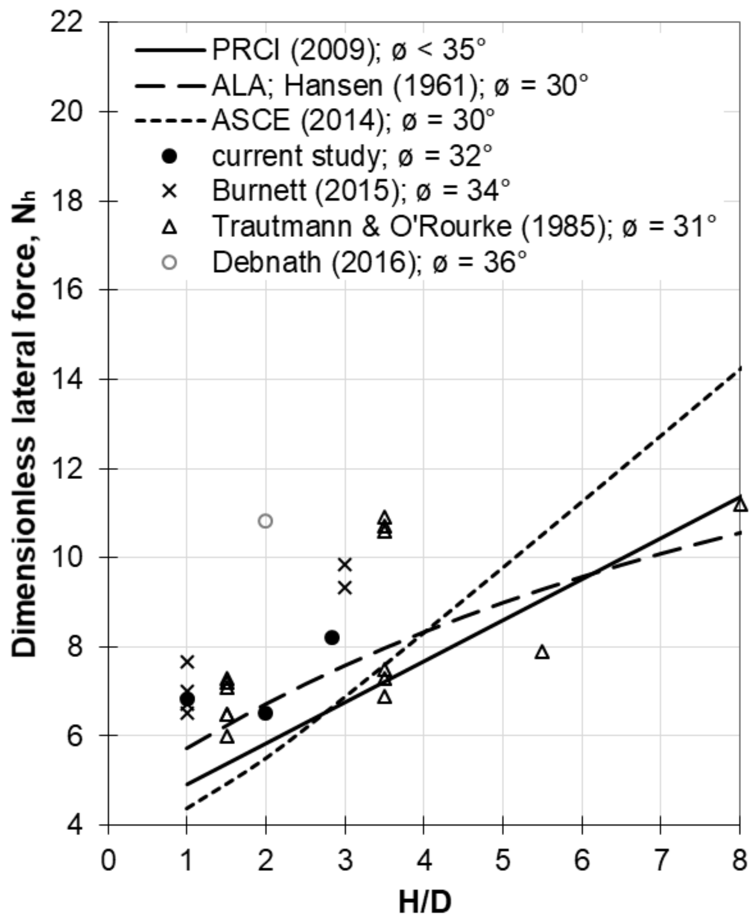

Figure 3.9. Comparison of the normalized resistance with several published test results and guidelines for loose sand

The results of the current study are comparable with the results of the full-scale experiments conducted by Burnett (2015). There were several differences between the current study and Burnett (2015) including small scale modeling using centrifuge, as well as the sand type and relative density. Table 3.5 describes some of the differences in the sand type, friction angle, scaling, pipe diameter, burial ratio, and relative density conditions.

Table 3.5. Lateral pipe–soil interaction experimental studies in sand

Experimental study	Sand type	Relative density condition	H/D	Pipe diameter (mm)	scale	Friction angle
Current study	Dry silica sand	Loose	1, 2, 3	252 & 605 prototype	7.95 & 19.06	32
Debnath (2016)	Dry silica sand	Dense and loose	2	609 prototype	13.25	32
Burnett (2015)	Dry synthetic olivine sand	Loose and dense	1, 3, 7	254 & 610	1	32.7 - 35.4
Karimian et al. (2006)	Moist & dry Fraser River	Medium dense	2.75, 1.92	324 & 457	1	32 - 34
Trautmann and O'Rourke (1985)	Dry Cornell filter sand	Loose to dense	1.5, 3.5, 5.5, 8, 11	102 & 325	1	31, 36, 44
Audibert and Nyman (1977)	Carver sand	Loose to dense	1 to 24	25, 63.5, 114.3, 228.6	1	35

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CHAPTER 4. Experimental study of trench effect on lateral failure mechanisms around the pipeline buried in clay

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Abstract

Subsea pipelines in shallow areas are usually buried inside trenches backfilled with pre-excavated material as a cost-effective protection against the environmental, constructional, and operational loads. The design of buried pipelines for potential lateral displacements is a challenging aspect that is usually simplified by assuming a uniform soil surrounding the pipe. However, the remolded backfilling material and its lower stiffness compared with the native ground can significantly affect the failure mechanisms around the moving pipe and the mobilized lateral soil resistance. In this study, the lateral pipeline-backfill-trench interaction and the resultant soil failure mechanisms were investigated by centrifuge tests. Particle image velocimetry (PIV) analysis was conducted to capture the interactive soil displacements and failure mechanisms. The partially drained condition was adopted to magnify the pipeline-backfill-trench interaction effects, and the significance of the burial depth was also investigated. It was observed that the interactive effects of pipeline, backfill, and trench precede their individual shear strengths. The study revealed the significance of the pipeline/trench-bed interaction in the mobilization of the lateral soil resistance and several other mechanisms not yet addressed in the literature. As a result, several new research avenues were identified, and the ground was prepared for proposing cost-effective solutions to improve the prediction of the lateral response of buried pipelines in the near future.

Keywords: Lateral pipe-soil interaction; p-y response; large deformation; centrifuge testing; trenching and backfilling

4.1 Introduction

Subsea pipelines that are widely used for the development of offshore fields can be subjected to large lateral displacements due to environmental, operational and accidental loads (e.g., ground movement, ice gouging, drag anchors, etc.). Subsea trenching and backfilling by re-using the excavated material is usually a cost-effective solution to protect the pipeline against lateral displacements. Depending on the trenching methodology, the construction procedure, and the environmental loads, the backfill material may undergo different degrees of remoulding and disturbance. This process causes the backfill material to be much softer than the native ground, with a wide range of shear strengths ranging from negligible to almost native soil strength values (Paulin et al. 2014).

The difference between the stiffness of the backfill and native material can significantly affect the failure mechanisms around the pipe, and the resultant lateral soil resistance (Pipeline Research Council International (PRCI) 2003). Figure 4.1 schematically shows the potential scenarios that may happen depending on the relative backfill/native soil stiffness.

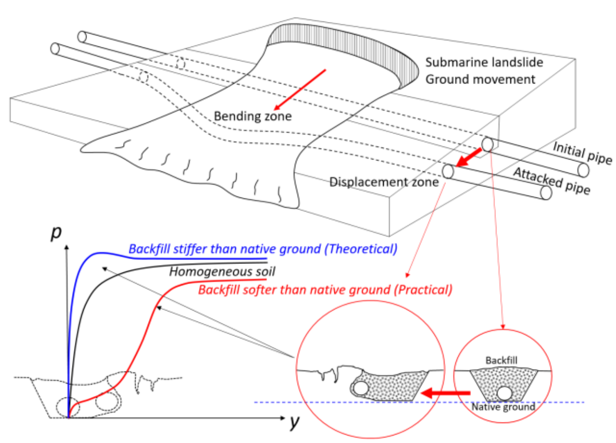

Figure 4.1. The lateral response of trenched and backfill pipeline to subsea geo-hazards

However, the less-explored interaction between the pipeline, backfill, and the native ground (trench walls) have caused the design standards to simplify the buried pipe configuration to a uniform soil. The practical incorporation of this simplification needs excavation of an adequately wide trench that results in a high construction cost. This is only to ensure the pipeline response will depend solely on the properties of the controlled backfill material, and not on the stiffer native ground (Kouretzis et al. 2013).

In the present study, centrifuge tests were conducted at C-CORE to investigate the lateral soil response to the large displacements of the shallowly and deeply buried pipelines. A mixture of Speswhite kaolin clay and Sil-Co-Sil silt was used to prepare a soft native ground (undrained shear strength less than 25 kPa), which has been observed in shallow waters (e.g., water depth less than 100 m) over different geographical locations (e.g., Bohai Sea (Liu et al. 2013), Mackenzie Delta (Solomon 2003), Changi Bay (Bo et al. 2015), and Persian Gulf (DOT 2011)). The selection of a soft clay as native ground enabled observing the significance of the interactive failure mechanisms in lateral soil resistance, having even a limited difference between the stiffness of the backfill and trench. Trenches were excavated in prepared native ground and backfilled with slurry after locating the instrumented pipe sections. Particle image velocimetry (PIV) was applied to monitor the progressive pipeline-backfill-trench interaction mechanisms.

Using a low pipe velocity, the partially drained condition was adopted to mitigate the impact of the excess pore pressure, magnify the pure effect of pipeline-backfill-trench interaction, and obtain the contribution of the trench to the overall failure mechanisms. This was motivated by reviewing the observation made by Paulin (1998), where the higher pipeline velocities (undrained condition) caused the backfill to become less of a factor in interaction. In addition, in many geographical locations, silt fractions are found in natural offshore soft clays (e.g., the Gulf of Mexico, Schiffman (1982) that tends the consolidation characteristics of clay towards partially drained and even fully drained conditions. Other compositional and depositional fractions may also show a similar effect. Also, Paulin (1998) did not investigate the burial depth effect in the partially drained condition that was covered in the current study as a remaining knowledge gap. Therefore, the main objective of the current study was set to qualitatively investigate the lateral pipeline-backfill-trench interaction mechanisms in clay through direct observation under partially drained condition.

The force-displacement (p-y) curves were obtained and compared with corresponding PIV analysis throughout a large course of pipeline displacements (about 4D). It was observed that the interactive effects of pipeline, backfill, and trench preceded their individual effects, and the mobilized soil resistance was not solely the result of the shear strengths of backfill/native soil. The pipeline/trench-bed interaction, the trench geometry, and the passive backfill pressure affected the pipe displacement trajectory and consequently, the progressive formation of shear bands underneath and behind the collapsing trench wall. As a result, a lower ultimate lateral soil resistance was achieved compared to the uniform native ground. The partially drained condition magnified the pipeline-backfill-trench interaction effects with more significance in the deeply buried pipe. The current study revealed several important mechanisms, e.g., the significant influence of pipeline/trench-bed interaction on the mobilization of the lateral soil resistance not yet well addressed in scholarly researches. Several new research avenues were identified, and the ground was prepared for improved and cost-effective design of buried pipelines in the near future.

4.2 Experimental modelling considerations and testing program

There are only a few studies that have investigated the trenching/backfilling effect on lateral pipe-soil interaction (e.g., Ng 1994; Paulin 1998; PRCI 2003; and Phillips et al. 2004). However, the only systematic experimental study on the lateral performance of trenched/backfilled pipeline was conducted by Paulin (1998) and extended later by PRCI (2003), and Phillips et al. (2004). Overall, these studies investigated the effects of different backfills, soil stress history, trench geometry, pipe size, interaction rate, and burial depth through undrained, partially drained, and drained conditions. However, they could not directly observe internal soil displacements and failure mechanisms. This motivated the current study to set the observation of the failure mechanism as the main objective that was magnified by adopting a partially drained condition. The testing program was conducted at the C-CORE centrifuge facility located at the St. John's campus of the Memorial University of Newfoundland (Newfoundland and Labrador, Canada). The setup initialization tests were conducted in loose sand and validated against the full-scale tests published by Burnett (2015) (Kianian et al. 2018a). Two tests with shallow (T4P1, H/D = 1.45) and deep (T5P1, H/D = 3.70) burial condition were conducted to investigate the depth effect as well. Very soft slurry backfill with rectangle trenches was used via partially drained condition (Normalized velocity, $vD/c_v = 0.14$, based on Phillips et al. (2004)) to purely capture the trench effect on lateral pipe response. It is worth mentioning that the current study did not intend to quantify the effects of various parameters affecting the lateral pipe-soil interaction that might need a large number of tests. Rather, the study explored the interactive internal soil displacements and failure mechanisms via two tests resulting in the interpretation of the observed trends in p-y responses.

In order to optimize the testing program and effectively use the existing test data, the test setup was developed in a similar approach undertaken by Paulin (1998) and Popescu et al. (1999). The significant advantage of the current test set up compared to the earlier studies was the use of a transparent observation window and PIV analysis that enabled direct capturing of failure mechanisms and soil displacements beside the lateral p-y responses (see Figure 4.2).

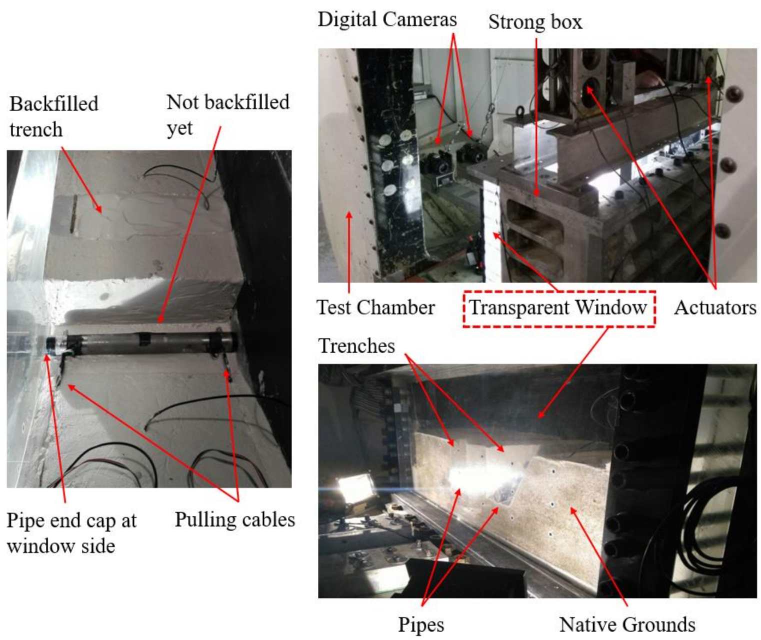

Figure 4.2. Transparent window and digital cameras for PIV analysis

The real pipe size was selected as 24” with an external diameter of 610 mm. The pipe size was a technical requirement of the project’s industry sponsor to keep the continuation of their earlier full-scale studies in sand conducted by Burnett (2015) at Queens University. Because of technical restrictions in accommodating the internal pressure transducer, the minimum possible pipe diameter was selected as 32 mm to guarantee that the cable of the transducer will not be folded. Combining this requirement with the prototype pipe size, the spinning acceleration was set on 19.1g. The internal walls of the strongbox were lubricated to eliminate the friction between the end caps and the box walls as much as possible. A trial pipe pulling was conducted without soil bed, and the pulling load was measured to ensure that the friction has been properly suppressed. In order to eliminate the friction between the cable and the soil, two strain gauges were installed on each pipe to purely capture the lateral soil resistance right in front of the pipe. In addition, load cells were installed on pulling cables to double-check the results obtained from the strain gauges. The load cells captured the friction forces as well, including the friction between the end caps and the box walls, the friction between the cable and pulley, and the friction between the cable and soil. The results presented throughout the paper were obtained from strain gouges.

The dimensions of the strongbox (900 × 400 × 300 mm, L × H × B) provided sufficient margins to prevent boundary effects. To obtain the in-flight undrained shear strength of the backfill and native ground, a T-bar penetrometer was mounted on the strongbox. The summary of test setup preparation is explained in the next section, and the full details can be found in Kianian et al. (2018b). Table 4.1 summarizes the details of the experiments.

Table 4.1. Summary of conducted experiments in shallow and deep trenches

Test ID	Burial depth ratio, H/D	Trench backfill type	Trench wall	Model displacement velocity (μm/s)	Normalized velocity $V_n = vD/c_v$	Total pipe movement
T5P1	3.70	Slurry	Vertical	2.98	0.14	3.75D
T4P1	1.45	Slurry	Vertical	3.01	0.14	3.95D

It is worth mentioning that centrifugal acceleration simulates the gravity and allows for correspondence of stress fields between model and full-scale permitting an accurate geotechnical modelling. However, to verify the scale effects, modelling of model tests at different "g" levels can be conducted to ensure the applicability and accuracy of centrifuge modelling. Paulin (1998) conducted modelling of model tests and attempted to model the same prototype condition at 1:25, l:50 and 1:100 scales for the lateral response of trenched/backfilled pipelines in clay. The study yielded acceptable results with most of the interaction curves within a bandwidth of 0.5-1 normalized lateral loads. These results provided sufficient relaxation to avoid repeating the scaling studies in the current research work, which has used a test set up almost identical to Paulin (1998). In addition, comparisons were made between the results obtained from the current study and a range of large and small-scale studies, and the resultant analytical and empirical solutions that will be further discussed in the later sections. Moreover, selection of an identical test set up to the earlier studies (e.g., Paulin 1998; PRCI 2003; and Phillips et al. 2004) with a single set of instruments, material mix, testing personnel, etc. were assumed sufficient to ensure the repeatability and reliability of the conducted tests.

4.3 Test setup preparation and testing procedure

The test apparatus was designed to conduct two separate tests at the same time. Figure 4.3 shows a schematic view through the transparent window. The model pipes were backfilled inside the excavated trenches in a pre-consolidated soil bed.

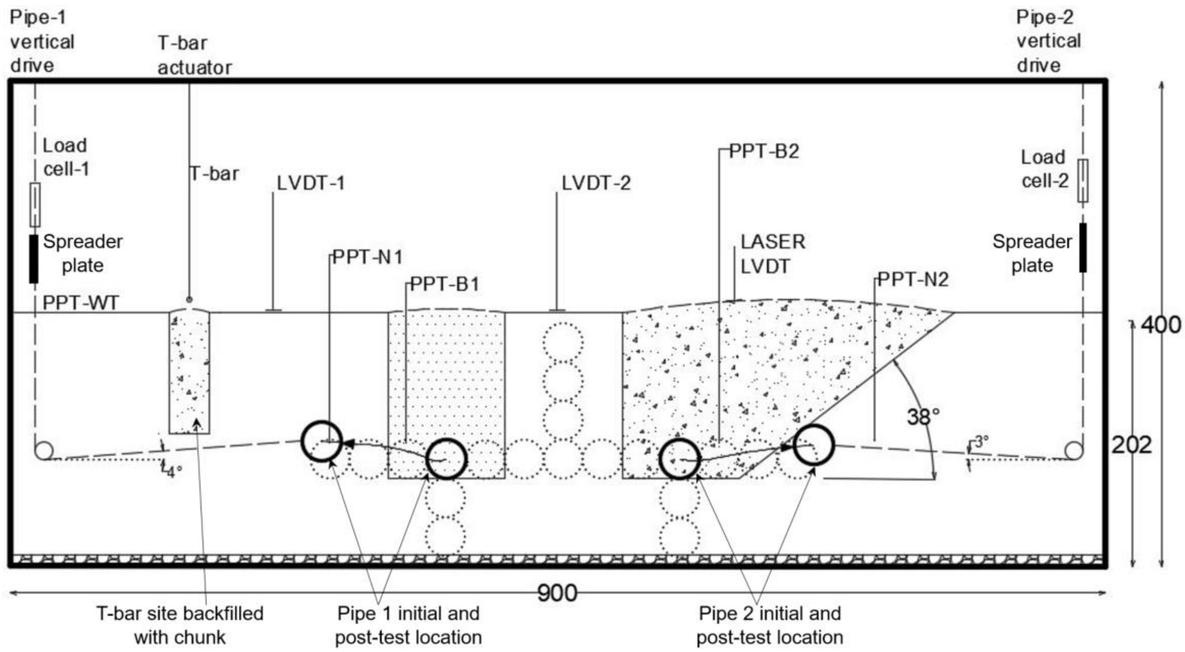

Figure 4.3. Sample schematic view of test setup and instruments (dimensions are in mm)

The pipes were horizontally pulled in opposite directions using pulling cables passed through the fixed pulleys. The pipes were pulled under displacement control condition but were free to displace vertically. Releasing of the vertical restraints enabled the pipes to slightly move upward at the beginning of test following the path of least resistance, and allowed to capture the significant effect of the pipeline/trench-bed interaction on lateral soil resistance. However, the pipes tended to move vertically downward with larger pipe displacements as the result of the pipe weight, bearing stress, and pulling cable. This configuration was in a similar context with the reality, where the laterally loaded/deflected pipe tends to return to its ideally straight configuration and develop a vertical load component. For a precise evaluation of the purely lateral soil resistance, the measured lateral forces are better to be calibrated against the instantaneous pipe trajectory. This would improve the accuracy of the current by $\pm 0.4\%$. Figure 4.4 shows the summarized sequence of test setup preparation.

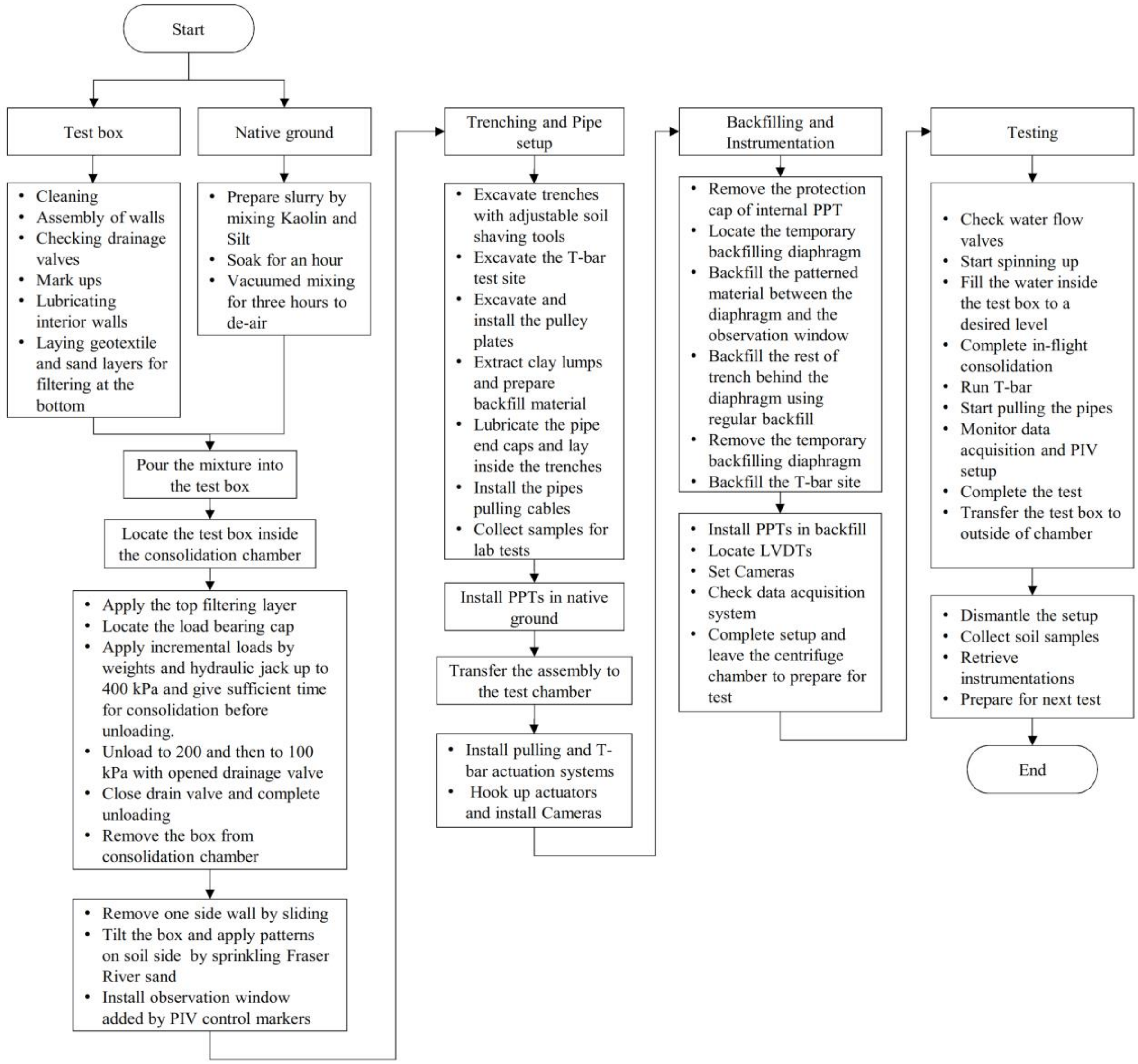

Figure 4.4. The sequence of test setup preparation and testing procedure

To prepare the native ground, Speswhite kaolin clay and Sil-Co-Sil silt were mixed by 50%-50% in weight with a sufficient amount of water to form a slurry with a nominal moisture content of about 70%. The native soil bed was incrementally consolidated up to the effective stress of 400 kPa and then was unloaded to 100 kPa with an open drainage valve. While removing the load, the water flows into the sample was restricted by closing the base drain and removing the excess water on top of the soil surface. This level of consolidation yielded a clay with an undrained shear strength of 15 to 25 kPa.

Trenches 3D wide were excavated using a blade with an adjustable side angle that was mounted on a guide beam sitting on the strongbox. The burial depth ratio (H/D) was defined as the initial ratio of the pipe springline depth to the pipe diameter. A 2D clearance was considered between the trench bottom and the lower drainage layer in the bottom of the test box to ensure that there would be no boundary effects. Table 4.2 shows a summary of the backfilling and native material prepared and tested in this study.

Table 4.2. Soil properties

Test ID	Trench backfill type	T-bar site backfill	T-bar site backfill c_u (kPa)	Native c_u at pipe SL (kPa)	Native water content before and after the test (%)	Υ_{sat} (kN/m3)
T5P1	Slurry	Slurry	<< 1	17.6	32.04 - 32.97	18.45
T4P1	Slurry	Slurry	<< 1	16.0	30.12 - 32.13	18.43

The model pipe was fabricated from a stainless steel pipe (31.75 mm in diameter) and instrumented with two sets of strain gauges, one internal pore pressure transducer (facing the rear of pipe), two strings of pulling cables, two rubber end caps (both lubricated, on patterned in window side) (see Figure 4.5).

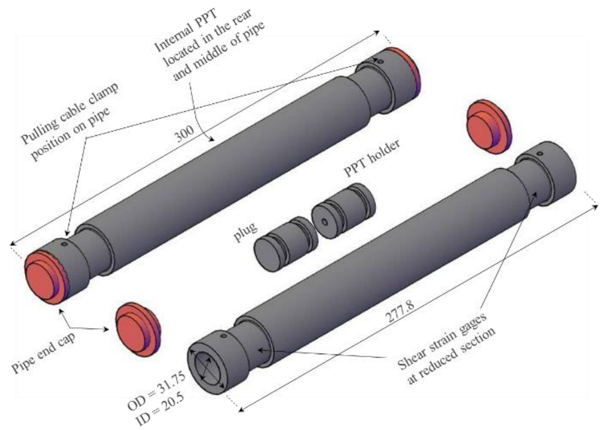

Figure 4.5. Pipe design and instrumentation

Several instruments were used along with 3 parallel strong data acquisition systems (each has 8 individually configurable inputs) to monitor the testing program. The instruments included pore pressure transducers (PPTs), strain gauges, load cells, conventional and laser linear variable differential transformers (LVDTs), T-bar, vertical drive motion controller, digital cameras, markers, and artificial textures. This enabled the full recording of progressive failure mechanisms and the development of shear bands in backfill and native soil, the lateral force-displacement response of the pipeline, the suction force variation behind the moving pipe, and the pore pressure variation both in the backfill and native ground. Figure 4.6 shows the key steps of conducted tests.

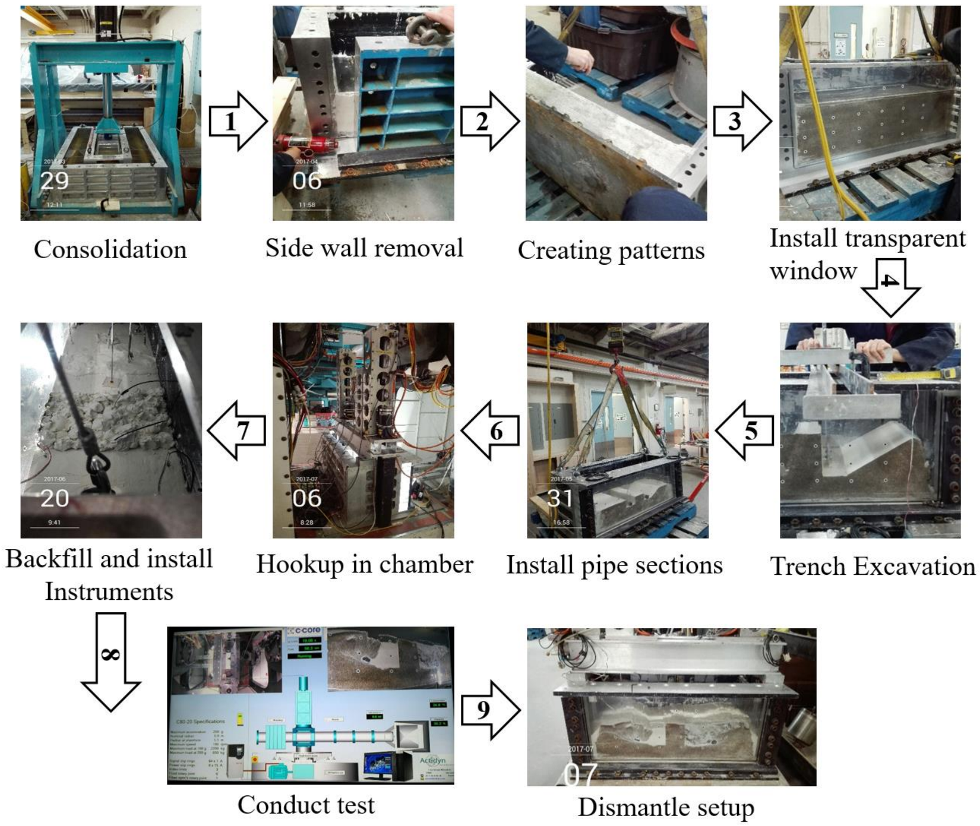

Figure 4.6. Visualized key steps of test setup preparation procedure

The pipeline displacement rate was set sufficiently low (vD/c_v = 0.14, partially drained based on Phillips et al. (2004)) to consolidate the surrounding soil, eliminate the effect of excess pore pressure and magnify the effect of pipeline-backfill-trench interaction.

4.4 Characterization of soil strengths

A T-bar penetrometer (Stewart and Randolph 1994) was used to assess the strength profile of the backfill and native ground in-flight. The T-bar head bearing area was 30 × 7.4 mm^2. Due to limited space on the test box and having access to only one T-bar actuator, the T-bar test site had to be designed in a way to allow assessing the backfill and native ground strengths in a single run. A T-bar site was excavated in the native ground and filled with backfill material in each test. The surface dimensions of the T-bar site was 75 × 50 mm^2, and the depth was selected to be deeper than the tested trench depth. These dimensions were used to properly obtain the undrained shear strength profile over the trench depth and the native ground involved in the failure mechanisms and also to ensure the prevention of the boundary effects in different directions. The T-bar was first penetrating to the backfill material and then continued to penetrate into the native ground. A T-bar test with no excavation in native soil was also conducted to capture the pure shear strength profile of the native soil. Figure 4.7 shows the schematic configuration of the T-bar site and arbitrary shear strength profiles.

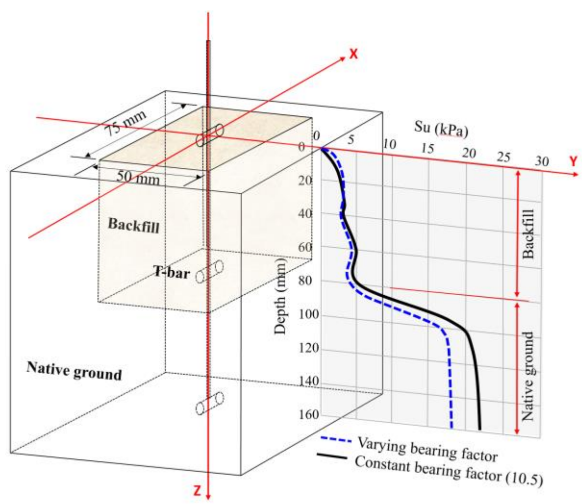

Figure 4.7. Schematic configuration of the T-bar site

A T-bar factor of $N_{kt} = 10.5$ was used to convert the measured unit bearing resistance, q, to the local undrained strength, S_u. This factor has been recommended by Stewart and Randolph (1994) based on the plasticity solution earlier proposed by Randolph and Houlsby (1984). For shallow depths, a reduced bearing factor considering the effect of soil buoyancy and a shallow failure mechanism mobilized prior to the full flow of soil around the bar was used to transform the measured bearing resistance to the undrained shear strength (White et al. 2010). The reduced bearing factor increases the shear strength profile that was obtained by the factor 10.5. This profile is considered for shallow penetrations. Figure 4.8 shows the undrained shear strength profile for the conducted tests outlined in Table 4.1. The good correlation between the shear strength profiles of the native ground from different tests shows that the native soil conditions were kept fairly similar between the tests.

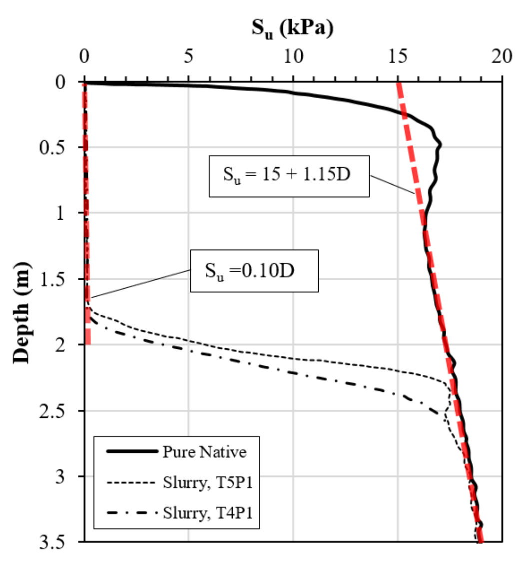

Figure 4.8. Undrained shear strength profiles and linear curve fits

Approximate linear S_u profiles were fitted for both the backfill and native soils to facilitate the back-analysis of the test data, as shown in Figure 4.8. Table 4.3 shows the magnitudes of mudline intercept, S_{um}, and the shear strength gradient, k_{su}, obtained from the proposed linear fits.

Table 4.3. Linear curve fits of undrained shear strength profiles in model scale

Soil Type	S_{um} (kPa)	K_{su} (kPa/m)
Native	15.0	1.15
Slurry	0.0	0.10

The undrained shear strength in slurry backfills is almost negligible. The native soil located underneath the backfill material showed a slightly softer response in the initial stages of penetration. This is due to the slight water dissipation from the backfill to the native soil. By increasing the penetration, the plots of overlaid native soil strengths are gradually matching the profile of pure native soil.

4.5 Test results

4.5.1 Lateral load-displacement response

Figure 4.9 shows the force-displacement responses against the normalized lateral displacement (y/D) of the conducted tests with the key configuration and test parameters. It is usually appropriate to normalize the load with the pipe diameter and undrained shear strength to obtain the interaction factor. In this case, the lateral load can be either normalized by the undrained shear strength of backfill, native ground, or both. However, normalizing the load by the strength of the backfill will be only appropriate for the backfill region and will provide no meaningful information for the native ground. Same will happen for the backfilling region if the load is normalized by the strength of the native ground. In case of normalizing the load both by the undrained shear strength of backfill and native soil within corresponding regions, significant discontinuities will happen in p-y response, particularly with the complex form of interaction factors in the transition zone near the trench wall. Therefore, the load response was not normalized for a proper presentation of p-y response.

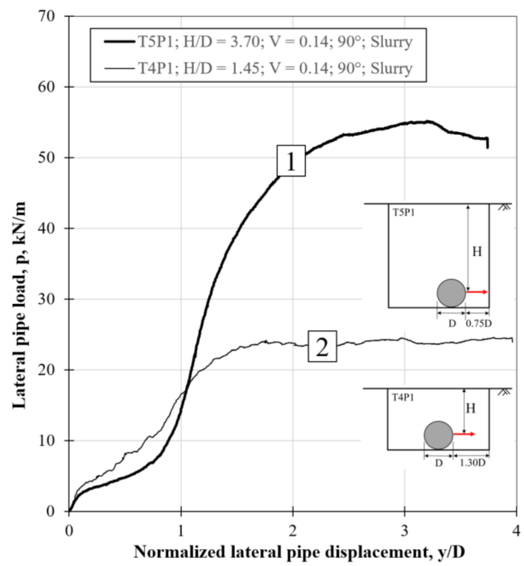

Figure 4.9. The lateral load-displacement response

Several important trends in the lateral p-y response were observed and can be explained by accurate PIV analysis. At the initial pipe displacement point, the magnitude of the load is slightly increased showing a relatively high stiffness at the beginning and continued by a softer response while the pipe is moving in the slurry backfill. By approaching the trench wall (native ground), the response gets stiffer, and the load is dramatically increased by a steep transition slope, which increases its incline with further penetration into the native ground. The p-y results show that the burial depth ratio (H/D) has a significant effect on the p-y response; the deeper the burial depth, the larger the lateral resistance, as reported by earlier studies as well (Karal 1983; Altaee and Boivin 1995; Paulin 1998). It is challenging to determine a displacement corresponding to the ultimate soil resistance since the load is continuously increasing. The load drop in T5P1 that happens at 3.20D displacement cannot be guaranteed that it is the ultimate resistance or a sign of a boundary effect due to a large global failure.

Figure 4.9 shows a lateral load of about 5 kN/m for the pipe inside the slurry, which is much larger than what is expected. It will be shown in later sections that by reviewing the PIV results the source of this load mobilization is the pipeline/trench-bed interaction, which significantly affects the lateral soil resistance in larger pipe displacements. The investigation of the pipe/trench-bed interaction and its influence on ultimate lateral soil resistance is one of the contributions of the current study. It will be shown that future comprehensive investigations are needed in this area to improve the current state of knowledge about the lateral response of trenched/backfilled pipelines.

Figure 4.10 presents the comparison of the test results with the p-y curves predicted by the existing design codes (i.e., PRCI 2009; ALA 2005; and ASCE committee 2014). The test results were examined against both the undrained and drained solutions, irrespective of the applicability. The soil strength parameters for the undrained condition were extracted from Table 4.3, and the drained parameters were adopted from the tri-axial tests (Paulin 1998).

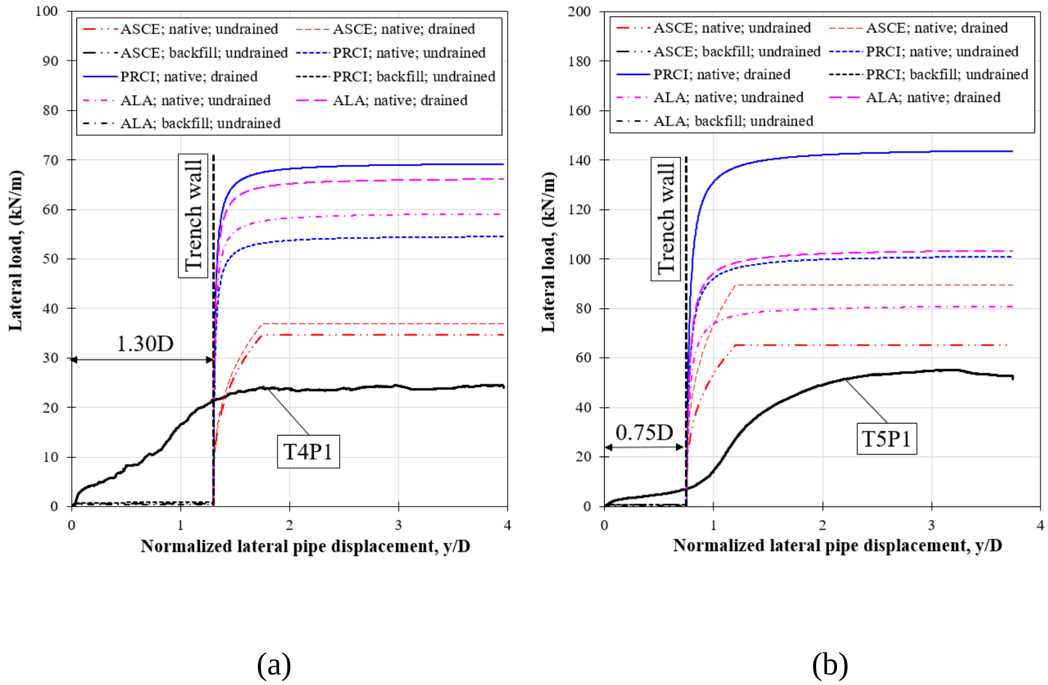

Figure 4.10. The comparison of the p-y responses between the test results and design codes

The results of partially drained tests did not fit inside the undrained and drained boundaries predicted by ALA (2005), PRCI (2009), and ASCE committees (2014). Paulin (1998) observed similar incongruities between the undrained and drained test results and the prediction made by analytical and empirical solutions that built the basis of the equations proposed by design codes. These design codes overestimate the ultimate load for a pipe moving inside the backfill or when approaching the trench wall and underestimate the lateral load for the pipe penetrating into the trench wall. It will be shown in later sections by PIV analysis that the significant reduction of the soil resistance in the native ground is the result of trenching that releases the passive pressure against the collapsing trench wall. Also, the larger soil resistance inside the backfill is primarily the result of the pipe/trenchbed interaction, not the shear strength of backfilling material. Neither of these effects are taken into consideration by design codes. The pipe/trench-bed interaction would comprise the initial vertical pipeline embedment, the shear strength of the native material at the trench bed, and potential suction forces between the pipe and the trench bed. Paulin (1998) referred to this potential interaction but couldn't further discuss it due to indirect monitoring of the internal soil deformations.

The interaction factors or the normalized lateral loads of the conducted tests against the burial depth ratio were compared with some of the key published studies shown in Figure 4.11. A close agreement was observed between the current results and the prediction made by Rowe and Davis (1982) (immediate breakaway) and Paulin (1998). The solution provided by Rowe and Davis (1982) and also some of other researchers (Hansen 1948; Hansen and Christensen 1961; Smith 1962; Ovesen 1964; Sokolovskii 1965; Kosteyukov 1967; Ovesen and Stromann 1972; Neely et al. 1973; Das and Seeley 1975) has been originally proposed for plate anchors but has been widely applied to pipelines due to a similar fashion. The current test results were also fairly close to the predictions made by Tschebotarioff (1973) and Wantland et al. (1979). The later study used an upper bound plasticity approach to predict the ultimate lateral resistance with a load-bearing factor varying with H/D and an average cohesion in a distance of 2D above the pipeline invert. However, the upper bound plasticity solution proposed by Merifield et al. (2001) and the equations recommended by the ALA (2005) overestimated the interaction factor for deeply buried pipes. This was because they did not address the effect of the pipeline-backfill-trench interaction on the failure mechanisms. Merifield et al. (2001) used two numerical procedures based on finite element formulations of the upper and lower bound theorems to propose equations for lateral clay resistance against plate anchor. The authors set a lower bound limiting value of 10.47 to reflect the transition from shallow to deep behaviour. A similar overestimation trend was observed in the predictions made by Oliveira et al. (2010).

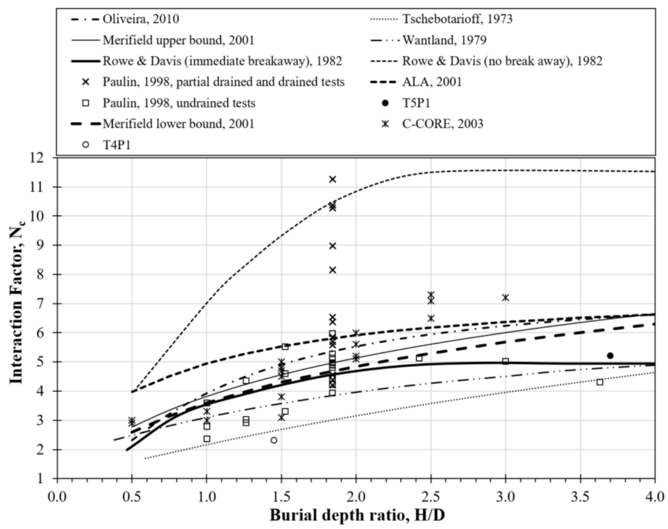

Figure 4.11. Interaction factor against the burial depth ratio

Overall, the design codes and the plasticity solutions that consider uniform soil strata, ignore the highly different stiffness between the backfill and the native soil. Therefore, they underestimate the lateral load inside the trench and in the transition zone and overestimate the ultimate response. The basis of the observed trends will be discussed in subsequent sections by reviewing the results of PIV analysis.

4.5.2 Pore pressure variations

PPTs were placed along the pipeline's moving path inside the backfill and native ground to assess the soil drainage conditions. These PPTs were also used to monitor the depth of water in the sample in order to determine the position of the water table. Figure 4.12(a), (b), and (c) show the variation of pore pressure against the pipe displacement in the backfills (PPT-B series), native ground (PPT-N series), and right in the rear of the pipe (Internal PPT). The location of PPTs was shown earlier in Figure 4.3. Figure 4.12(a) shows the pipeline-backfill interface pressure/suction variation that has been measured by the PPTs installed inside the pipes. The trend of internal PPTs indicate an initial increasing of the pore pressure followed by the dissipation of the excess pore pressure and the development of a slight suction force behind the pipe. The magnitude of this suction is quite limited due to the low displacement rate of the pipe in a partially drained test condition. The initial increasing of the pore pressure within the pipe displacement less than one diameter, may have been due to the competence of the backfill, where the excess pore pressure developed in front of the pipe may have been transferred by the backfill to the rear of the pipe. The PPT was damaged in test T5P1 and did not give a proper pore pressure variation.

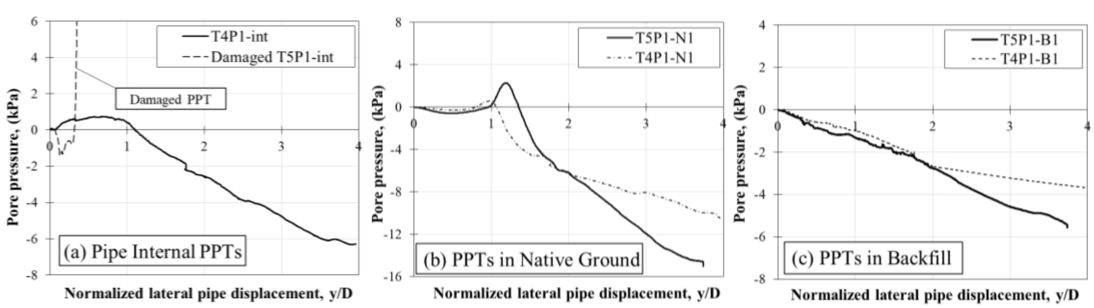

Figure 4.12. Variation of pore pressure in (a) the rear of the pipe (b) native ground (c) backfill.

Figure 4.12(b) illustrates the PPT measurements in the native ground. Overall, after a slight decrease and then increase, the excess pore pressure continued to dissipate with time and are slightly affected by the pipe interaction with the trench wall. The PPTs inside the backfills were installed on an elevation to allow the passing of the moving pipe from underneath the PPT. Figure 4.12(c) shows a contentious reduction of the pore pressure and the development of a slight suction force that might also be affected by the trench wall collapsing into the backfill.

4.6 Observed deformation and failure mechanisms

The internal soil deformations and interactive failure mechanisms were observed by using a transparent acrylic window on one side of the test box and PIV analysis. High-quality images were continuously captured by two cameras and post-processed by GeoPIV-RG software (originally developed by White et al. (2003) and further improved by Stanier et al. (2016))(see Figure 4.2).

Zoning of the observations

Different interactive mechanisms were observed in various stages of the pipe displacement, where the earlier stages significantly affected the later interaction mechanisms. In order to effectively discuss the observed interactive mechanisms, the pipeline displacement was divided into three different assessment zones (I, II, and III), shown in Figure 4.13, based on changing the key soil displacement mechanisms. Zone I covered the pipeline movement inside the backfill, from the initial pipe position to a second point inside the backfill, at which the rotational backfill flow around the moving pipe stopped. At this stage, the distance of the pipe-front from the wall was approximately 0.65D for the shallow trench (T4P1) and 0.5D for the deep trench (T5P1). In this zone, the pipeline has neither touched the trench wall nor even closely approached it. The pipe interacts with the backfill, but the pipeline/trench-wall interaction has not yet been effectively started. Zone II covers the area in which the pipe starts to interact with the trench wall without direct contact. This zone ends when the pipe arrives at the initial position of the untouched trench wall. The length of this zone was 0.65D for T4P1 and 0.5D for T5P1. Zone III represents the area in which the pipeline moves beyond the initial wall position and fully penetrates into the trench wall. It is worth mentioning that the dimensions of the assessment zones were selected intuitively and may need to change for different soil properties and pipeline configurations.

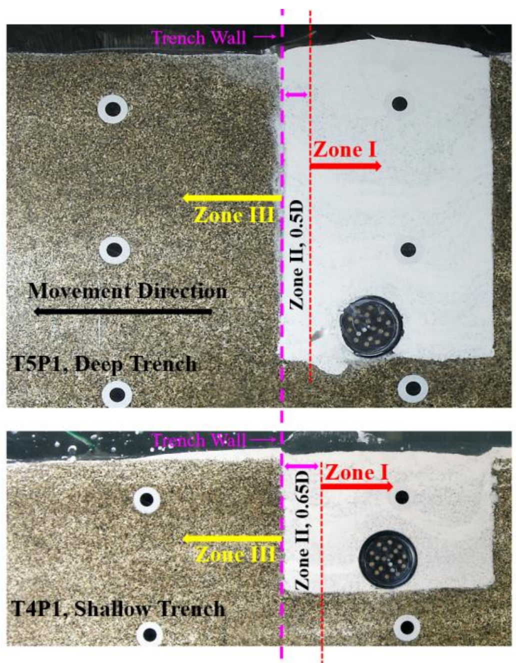

Figure 4.13. Observation zones based on key soil displacement mechanisms

Observations in Zone I

Two main mechanisms were observed in this zone: i) pipeline-backfill interaction ii) pipeline/trench-bed interaction. A complex interfering mechanism was also recorded when the mechanism "i" and "ii" interacted at the end of zone I. Figure 4.14 shows samples of the PIV analysis in Zone I.

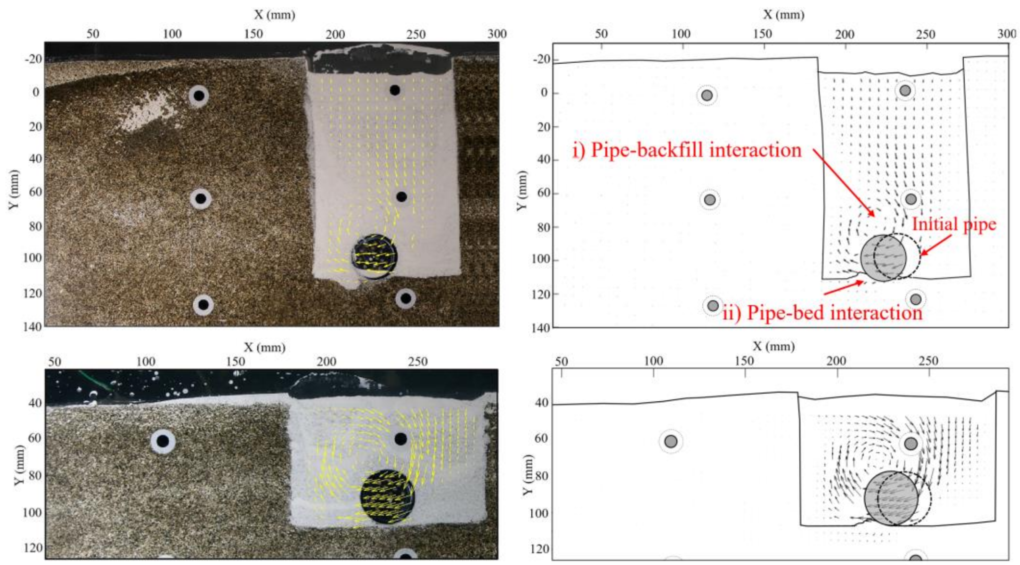

Figure 4.14. Sample PIV analysis results in Zone I (~ 0.25D pipe displacement)

A close investigation of recorded videos and PIV results demonstrated that the pipeline-backfill interaction (i) comprises loops of eccentric spiral soil flows with rotational circles around the moving pipe. These spiral flow surfaces emanate from a point above the pipe and move horizontally with the pipe until the failure surface touches the trench wall. From this stage, with a further displacement of the pipe towards the trench wall, the spiral flow starts to contract with a varying ratio that depends on its distance to the wall. The closer to the wall, the smaller the failure circle will be. Figure 4.15 schematically shows the spiral flow mechanism with the corresponding dimensions observed in the conducted tests.

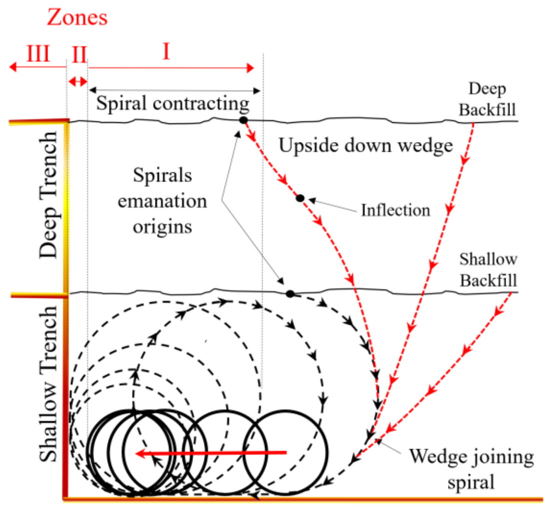

Figure 4.15. Contracting spiral and wedge mechanisms in shallow and deep backfills

Also, Figure 4.15 shows the difference between the emanation origin (EO) in shallow and deep trenches, where the EO is tangential to the surface of backfill in a shallow trench and sharply crosses the backfill surface in the deep trench. There is an additional upside-down wedge failure on top-right of the spiral flow with side legs remaining asymptotic to the spiral surface at the rear of the moving pipe. The far failure surface (in the rear of pipe) is still a kind of logarithmic spiral, but there is an inflection point in the near failure surface (above the pipe) of the deep backfill. The observations show that rotational spiral flow is dominants and the size of failure circles are only governed by the distance to the trench wall, not the near failure surface. The comparison of these flow mechanisms with obtained lateral p-y curves shows that the vertical flow wedge has almost no influence on the mobilized rotational shear against the pipe's movement inside the slurry backfill. However, the larger soil column on top of the deeply buried pipe in T5P1 applied more overpressure on the pipe, resulting in greater friction between the pipeline and trench bottom. Looking at progressively contracting spiral flows, one may expect dissipation of the lateral load on the moving pipe section due to the reduction of the magnitude of mobilized rotational shear stress in the backfill. However, the obtained p-y responses show a continuous increase in lateral load (see Figure 4.9). Therefore, while the pipeline is inside the trench, the lateral load on the pipeline does not seem to be governed by a spiral flow mechanism.

A closer investigation of the recorded videos and PIV results showed a second mechanism that is significant in the assessment of the lateral soil resistance. This mechanism is a result of the interaction between the pipeline and the trench-bed. The mechanism is schematically shown in Figure 4.16, where the pipe slightly penetrated into the trench bed during the inflight consolidation due to pipe weight and the bearing stress.

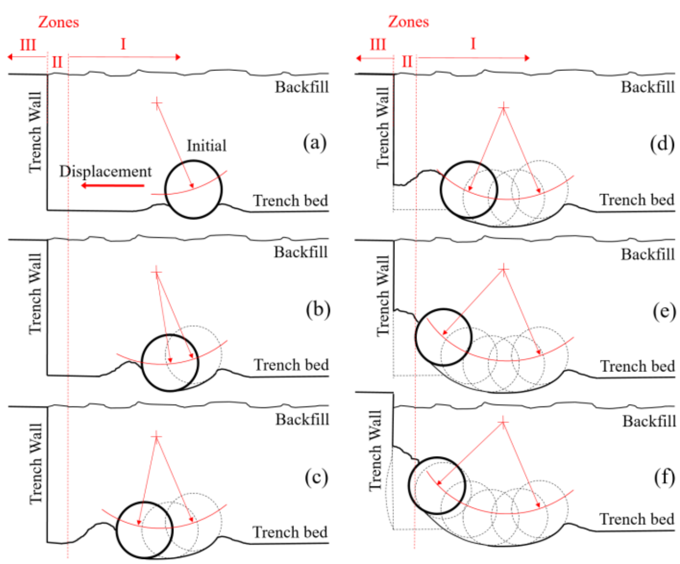

Figure 4.16. Schematic presentation of pipe-bed interaction in Zone I

This initial embedment resulted in the creation of small soil berms in the front and rear of the pipe. Due to a minor penetration of the slurry backfill into the native soil around the internal surface of the trench, these small soil berms were barely seen in the tests, but the recorded videos confirmed their existence and contribution. As the pipeline moved laterally, the front berm was successively developed pushing the pipeline upward into the backfill that had a lower strength. The upward movement was accelerated as the pipe further approached the trench wall, where the front berm was stuck between the pipe and trench wall and was compressed to the trench corner. In addition, the squeezed soil berm that is stiffer than the backfill intervened and stopped the rotational failure in front of the pipe, which was considered to be the starting point of the Zone II.

Considering the low magnitude of the shear strength in the slurry backfill, this second mechanism (mechanism "ii") is the main contribution to the p-y curves in the Zone I (see Figure 4.17). A closer look at the p-y responses of the test T5P1 and T4P1 in the Zone I showed that the resistance in T4P1 started earlier and achieved a higher value compared to T5P1. Then the p-y curve enters into Zone II with a slope less than T5P1 (see Figure 4.17). This is due to the test configuration and the relative elevation of the pipeline springline and the pulley. The elevation of the pipe springline in the test T4P1 (Shallow trench) is +2.4 mm above the elevation of the pulley invert, i.e., the point at which the horizontal cable turns to be vertical and attached to the actuator. Therefore, the cable pulls down the pipe slightly, while it is laterally displaced in Zone I. As shown in Figure 4.17, this causes the pipe to first slightly move downward and then shift upward over about 1.0D horizontal displacement of pipe (negative part of plot No.4 in Figure 4.17). The longer shear zone underneath the pipe and the larger volume of mobilized soil berm in front of the pipe cause the p-y response of T4P1 to be higher than T5P1 (plot No. 3 in Figure 4.17, the area less than 1.0D). In T5P1, the pipe springline is -5.1 mm below the pulley invert. The cable slightly pulls up the pipe and shortens the shear zone underneath the laterally moving pipe (negative part of plot No.2 in Figure 4.17). This results in a mobilized lateral force less than test T4P1 (plot No.1, the area less than 0.75D).

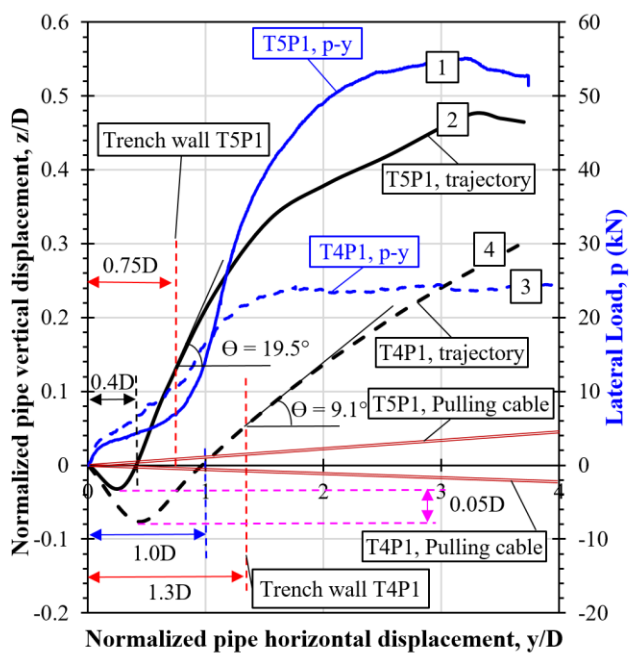

Figure 4.17. Pipe trajectories and p-y curves

It is worth mentioning that these mechanisms did not end in Zone I and have a significant impact on later stages of soil deformation mechanism through Zone II and III that will be discussed in later sections of this study.

The lateral and breakout soil resistance in partially embedded pipelines has been widely investigated in the past using numerical (Merifield et al. 2008; Wang et al. 2010; Chatterjee et al. 2012), experimental (Cheuk et al. 2007; White and Randolph 2007; Dingle et al. 2008), and analytical (Das and Seeley 1975) methods. In all of these studies, the pipeline is partially exposed to the water or air on its top. In case of the buried pipelines, the water or air on top of the pipe is replaced with soft backfilling soil. This pipeline-backfill-trench bed interaction would significantly affect the berm development, lateral soil resistance, and the lateral breakout by changing the magnitude of the passive pressure on top of the pipe and the berm. To date, there have been no studies that investigate the backfill effect on berm development, the lateral resistance and break out of the buried pipes.

Also, in the published studies, the trench bottom width is believed to have no effect on the lateral pipe-soil interaction (Paulin 1998), which is not in agreement with the findings of the current study. The current study shows a high potential for a significant effect of the trench width on lateral soil resistance. Figure 4.18 shows a typical lateral response for partially buried pipelines (DNV.GL (2017))

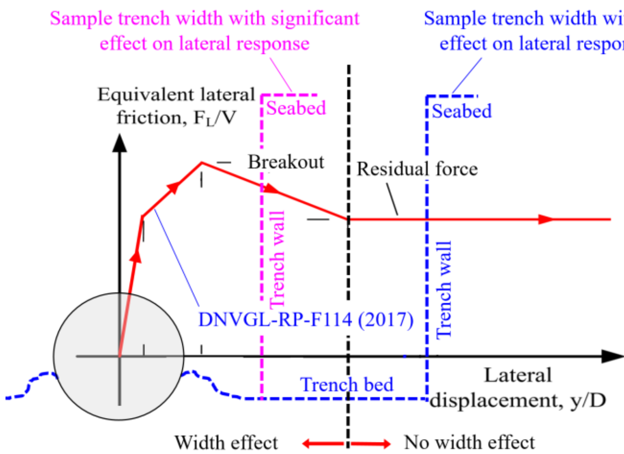

Figure 4.18. The typical lateral response of partially buried pipe and its relation to the trench width

Combining the observations of the current study with the response shown in Figure 4.18, it can be concluded that in a wide trench, the pipeline/trench-bed interaction can be fully developed with full lateral pipe break out. In such a case, when the pipe arrives in Zone II, it is in the steady-state zone of Figure 4.18, and there would be no soil berm in front of the pipe. This will eliminate Zone II and significantly affect the lateral response. However, in reality, the trench width is not wide enough to allow a complete lateral breakout to the pipe, leaving a soil berm in front of the pipe, and consequently, a non-existent Zone II. Therefore, it can be concluded that the trench bottom width can have a significant effect on the lateral response, except for extremely wide trenches. This area still needs further in-depth investigations, currently being undertaken by the authors.

In addition, in the real practice, a wide range of parameters including the pipe weight, construction process, backfill properties, longitudinal profile, etc. can affect the pipeline interaction with the trench bottom and consequently its impact on ultimate lateral soil resistance. These aspects are significant and need to be explored. A comprehensive research program has already been initiated by the authors to investigate these aspects as the result of the findings of the current study.

Observations in Zone II

Entering into Zone II, two important effects initiated in Zone I influenced the soil resistance. First, the developed soil berm squeezed into the trench corner pushed the pipeline upward and resulted in an oblique penetration into the trench wall. Second, the squeezed soil berm intervened and stopped the rotational backfill flow in front of the pipe due to its higher stiffness compared to the backfilling soil. This mechanism converted the pipe diameter to act like a virtual larger pipe section penetrating into the trench wall and affected the burial depth ratio and failure mechanism in the later stages of lateral pipe movement (see Figure 4.19). The diameter of the virtual pipe in a shallow trench (T4P1) was larger compared to the deep trench (T5P1) due to the lesser soil pressure applied to the squeezed soil berm. The larger and deeper the pipe-bed interaction, the larger the diameter of the virtual pipe section.

One may expect these two mechanisms to show inverse effects on lateral soil resistance in Zone III. The oblique penetration into the wall was expected to decrease the volume of mobilized soil wedge in the native ground and decrease the ultimate soil resistance. However, it was observed that the secondary effect of these two mechanism depends on the burial depth when both of them are almost identically initiated. As shown in Figure 4.19, in the case of the shallow trench (T4P1), the larger virtual pipe globally pushed the trench wall to move away horizontally. This, in turn, caused the first global spiral shear band to form way before the arrival of the real pipe to the initial location of the wall (1D displacement) and to completely develop towards the ground surface at the end of Zone II. Also, the relocation of the trench wall resulted in the backfilling surface and consequently, the burial depth ratio to quickly drop due to the widening of the trench. In deep trench (T5P1), the failure was found to be more localized in the form of an obliqued large virtual pipe penetrating to the trench wall. In this case, the trench wall was not globally relocated, and there was almost no shear band developed by the end of Zone II. In other words, despite the shallow trenches, the trench wall was not globally displaced in deep trenches, while the pipe was laterally traveling inside the Zone II. In the case of the deep trench, the energy cannot be dissipated within the global shear bands and stored via local compressions over the Zone II. When combined with a larger soil column on top of the pipe, it results in a higher lateral p-y response (see Figure 4.17).

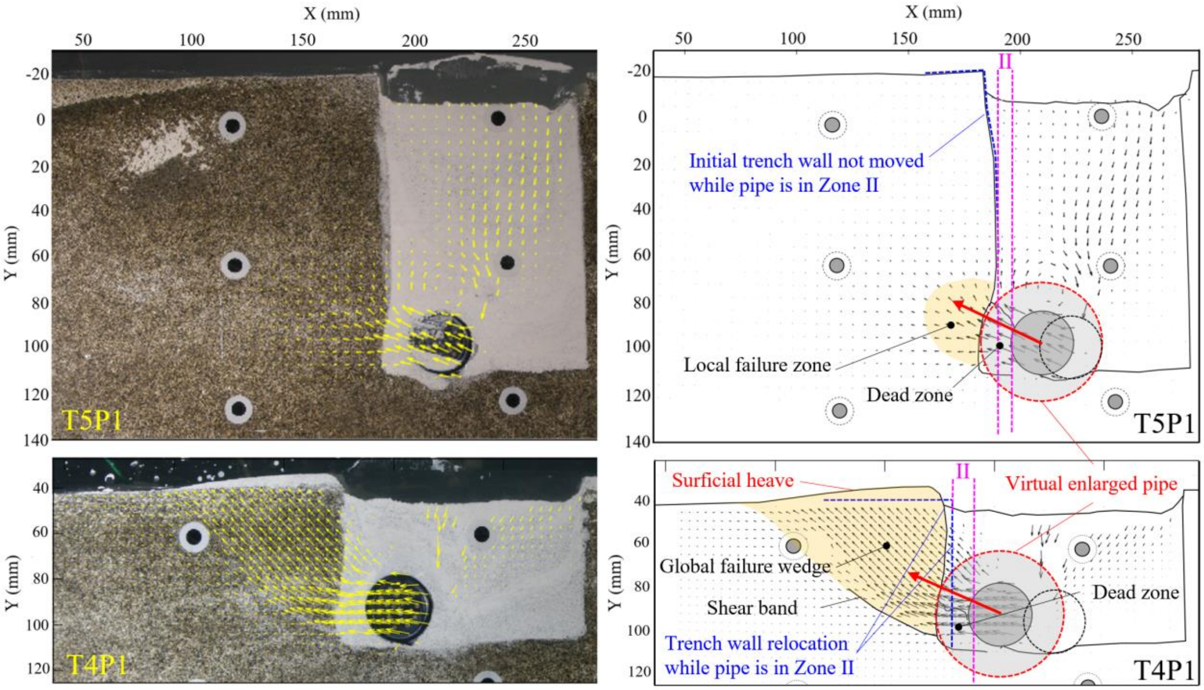

Figure 4.19. Different soil displacements in Zone II

As mentioned earlier, in practice, the probability of a pipeline falling into Zone II is likelier than it falling into Zone III, where pipelines undergo extreme relocations. There is still no plasticity solution or empirical equation in the literature to predict the lateral soil resistance against the moving pipe in Zone II. The existing models underestimate the lateral soil resistance in this zone (see Figure 4.10), and the area still needs to be further investigated. The authors have recently initiated a complementary research program to propose a new lateral pipe-soil interaction model within Zone II for the trenched and backfilled pipelines.

Similar to the influence of Zone I on Zone II, Zone II affects the failure mechanisms in Zone III, where the pipeline-backfill-trench interaction enters into a higher level of complexity.

Observations in Zone III

By approaching Zone III, where the pipe front arrives at the initial trench wall location, a small triangular wedge was created in front of the pipe, while the logarithmic spiral shear band was faster developed underneath the pipe (see Figure 4.20). The observed isosceles triangle, which is similar to Terzhaghi's active zone under a footing, had a different size and direction in shallow (T4P1) versus deep trenches (T5P1). It also followed a different progression scheme. As compared in Figure 4.20, the active wedge in a shallow trench, which was larger than the deep trench, was surrounded by a spiral shear band underneath the wedge. In the deep trench, the active wedge was completely separated from the spiral shear band and was smaller compared to the shallow trench.

As the pipe proceeded into Zone III, the upper side of the small active wedge truncated the virtual pipe and shrunk its diameter to get even smaller than the real pipe. As soon as the top-front edge of the real pipe touched the native trench wall, the first global instability of the trench wall appeared on top of the pipe, and a slice of the native soil slid down into the backfill. The base of the failed slice asymptotically reached the top of the pipe in the form of a spiral surface. This mechanism was fairly similar in shallow and deep trenches (see Figure 4.20).

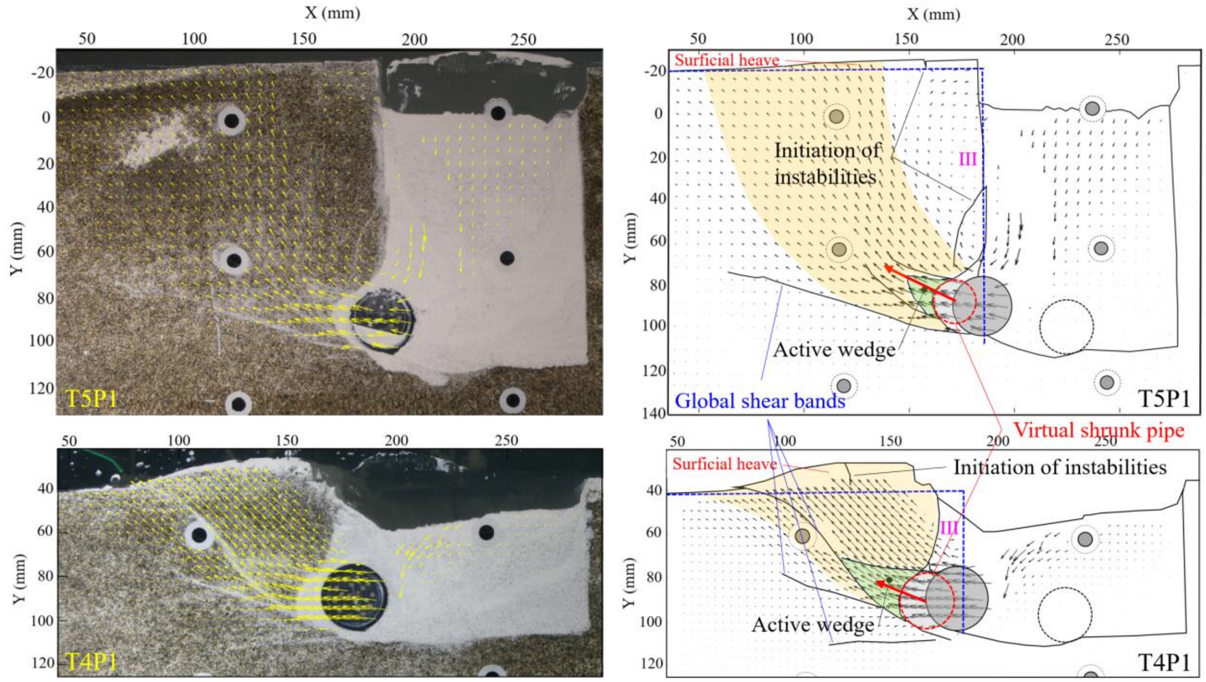

Figure 4.20. Trench deformations at the start of Zone III

Figure 4.20 shows as the pipe moved forward in a shallow trench, the global shear band under the pipe was entirely developed towards the ground surface, and a large passive block was created in the top-front area of the pipe. In deep trenches, this global shear band was progressively developed but never arrived at the ground surface, and this caused a significant increase in the lateral soil resistance. Also, the offset angle between the horizontal line and the cable was enlarged, causing the cable to apply a downward pulling force component on the pipe. This downward force affected the pipe trajectory and resulted in a new global spiral shear band to form under the pipe. In addition, the continuous changing of the pipe trajectory caused the progressive creation of a new triangle active wedges with an inclination angle that was gradually reduced following the pipe's trajectory. As the pipe was further displaced, new unstable slices of trench wall slid into the backfill.

A very important trench-backfill interaction mechanism was observed at this stage, which was significantly different in shallow (T4P1) and deep trenches (T5P1). As shown in Figure 4.21, in the case of a deep trench (T5P1), when the slices of the trench wall slid into the backfill, a passive pressure was applied by backfill against the failing soil slice. Obviously, the magnitude of this passive pressure depends on the stiffness of the backfilling material. This could affect the severity of the trench wall failure and control the shape and the volume of the failing wall slice. In other words, the stiffer backfilling material created a large passive pressure against the failing wall slices and mitigated the instability of the trench wall. This, in turn, resulted in a larger soil wedge mobilized in front of the moving pipe and a higher magnitude of soil resistance in comparison with softer backfill. In reality, the stiffness of the backfilling material may widely vary depending on a range of parameters, including, but not limited to, backfilling soil type, trenching methodology, construction procedure, etc. Therefore, all of these parameters can affect the magnitude of the passive pressure against the failing trench wall and consequently, the ultimate soil resistance against the moving pipe. This aspect is significant and requires further investigation.

In the case of shallow trenches, as discussed before, because of the horizontal trench wall relocation, widening the trench and reducing the backfilling soil height, the backfill elevation appeared much lower than the failing wall slices. This caused the backfill to have no efficient passive pressure against the wall sliding. In other words, in shallow trenches, the backfilling stiffness had no or only a minor effect on lateral soil resistance against the moving pipeline, which was inverse to the deep trenches. As the pipe section was further displaced laterally, the observed mechanisms in Zone III were progressively updated based on the pipe's trajectory, and large soil heaves were formed on the ground surface. Figure 4.21 shows the ultimate soil deformation in shallow and deep trenches for about 4D lateral displacement. It is still hard to say that the large failing trench wall applies an active load behind the pipe, pushing it forward. However, the PIV results suggested that the probability of such active pressure needs further investigation.

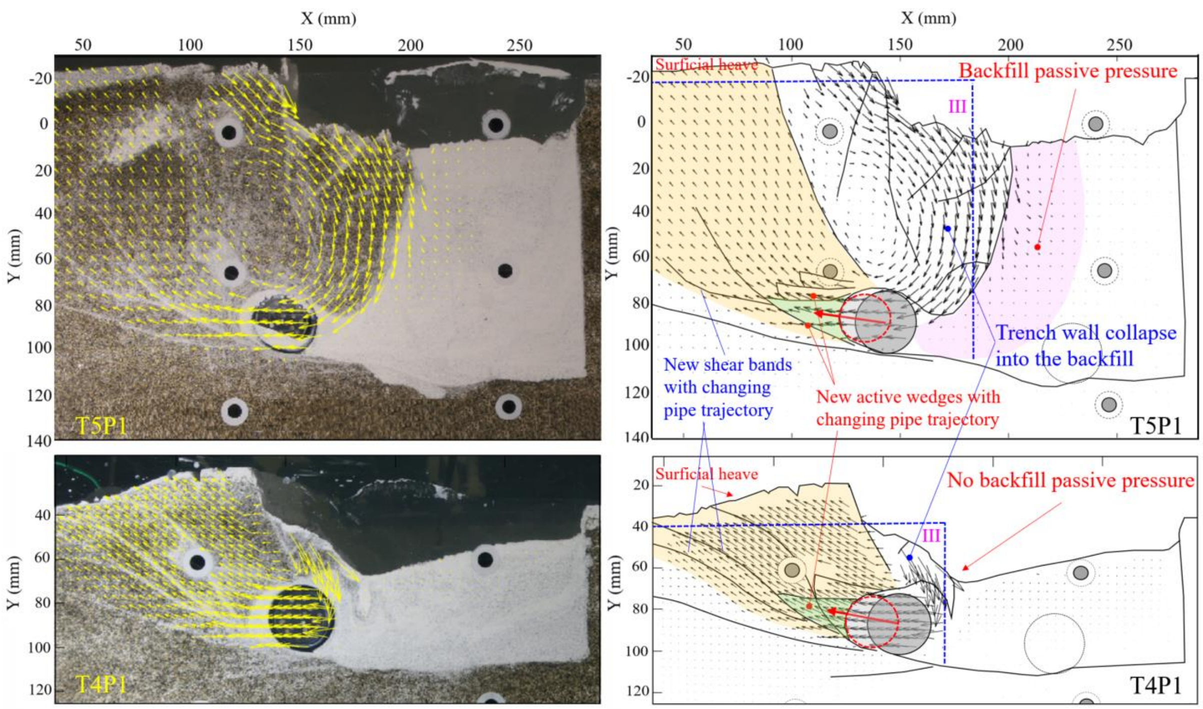

Figure 4.21. Trench deformations midway in Zone III

A series of maximum total shear strain variations throughout Zone I, II, and III, along with the observed deformations are presented in Figure 4.22 to show a better view of the mechanisms. The shear bands and failures have been obtained from captured images and coincided with PIV results. A good correlation was achieved between the PIV results and the actual deformations.

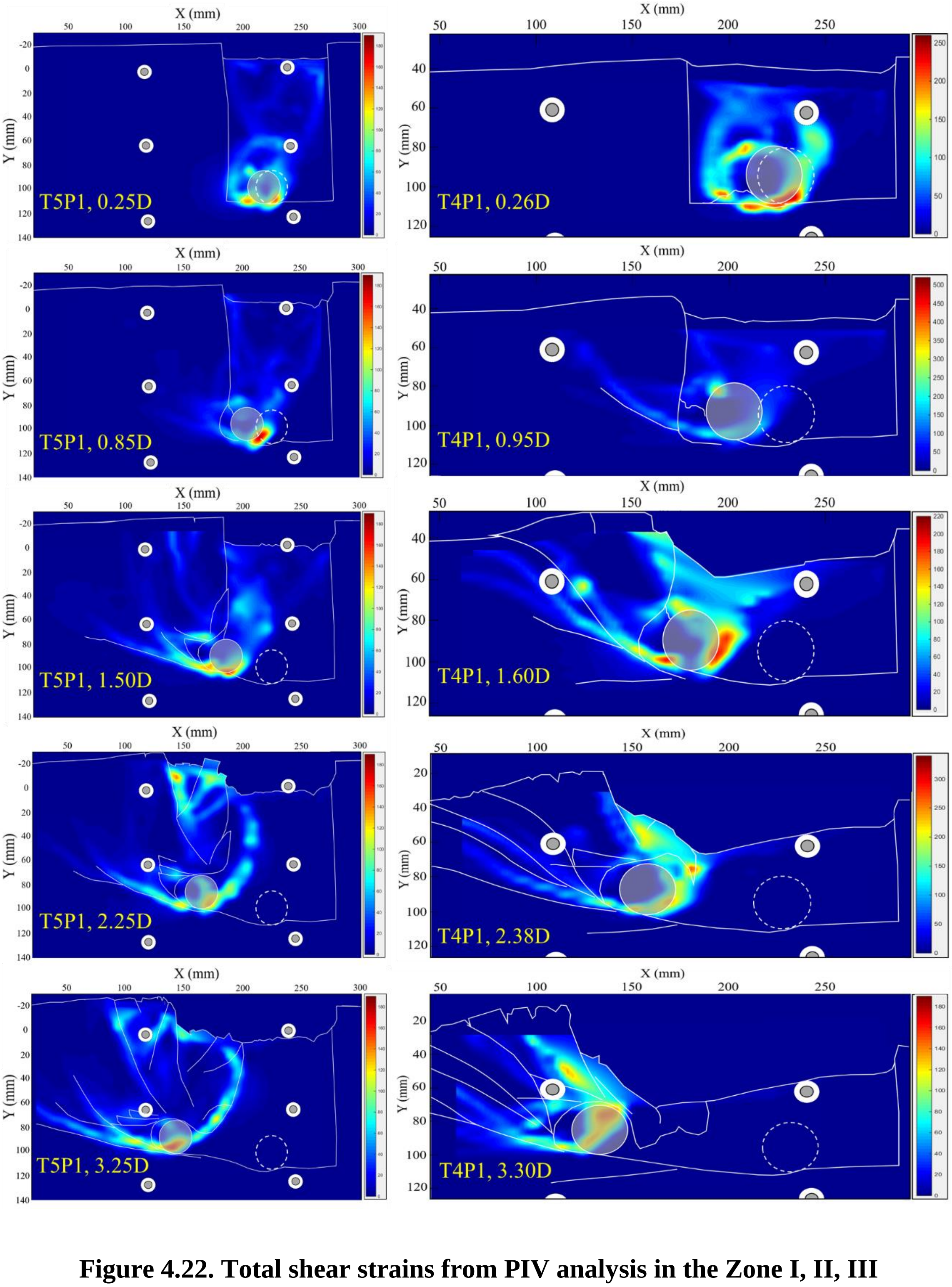

Figure 4.22. Total shear strains from PIV analysis in the Zone I, II, III

Analytical approximation of the mobilized resistance

There remains no plasticity solution for the large lateral displacement of either trenched/backfilled pipelines or even the pipe in a uniform cohesive soil under undrained, partial drained, and drained conditions. However, the results of the PIV analysis and the observed mechanisms combined with the recorded lateral loads in Zone I, II, and III can be used to approximate the mobilized soil resistance in a couple of instant spots in Zone I and III. For the purpose of this analysis, it was assumed that the backfill soil had no shear strength and the only mechanism acting in Zone I was the pipeline/trench-bed interaction (partially embedded pipe). In Zone III, the passive pressure applied by the backfill was neglected, and only the contribution of the active and passive wedges was accounted for. The influence of collapsed slips from the trench wall was also eliminated. In Zone II, the lateral resistance can be simply obtained by matching the values at the end of the Zone I and at the start of Zone III. Figure 4.23 schematically shows the assumed simplified mechanisms and the relationship between the mechanism in Zone I, II, and III.

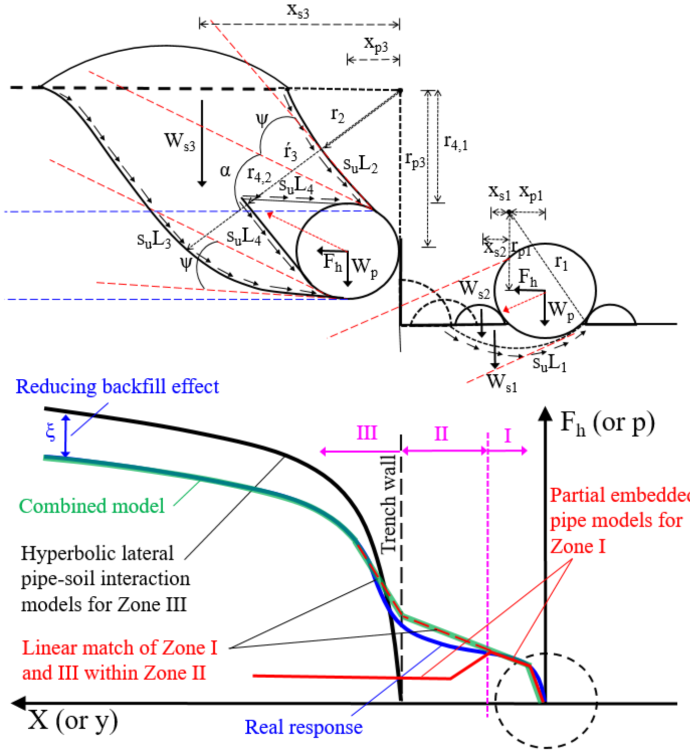

Figure 4.23. Schematic view of the simplified mechanisms and combining the models in Zone I and III

Using Figure 4.23, the horizontal force (F_h) required to displace the pipe laterally in Zone I and III can be given by following the basic equations:

$$F_{h\,(Zone\ I)} = \frac{S_uL_1r_1 - W_pX_{p1} + W_{s1}X_{s1} + W_{s2}X_{s2}}{r_{p1}} \tag{4.1}$$

$$F_{h\,(Zone\ III)} = \frac{S_u(L_2r_2 + L_3ŕ_3 + L_4(r_{4,1} + r_{4,2})) - W_pX_{p3} - W_{s3}X_{s3}}{r_{p3}} \tag{4.2}$$

Figure 4.24 shows the results of the analytical approximation of the soil resistance in a couple points in Zone I and III and its comparison with real lateral p-y responses obtained for tests T5P1 (deep trench) and T4P1 (shallow trench). The analytical results in Zone I were produced based on a partially embedded pipeline and its interaction with the trench bed is in better agreement with the test results compared with the analytical hyperbolic solutions for slurry backfill. This shows that the governing mechanism in Zone I is the pipeline/trench-bed interaction, not the pipe-backfill interaction. It is worth mentioning that these results are limited to very soft slurry backfills.

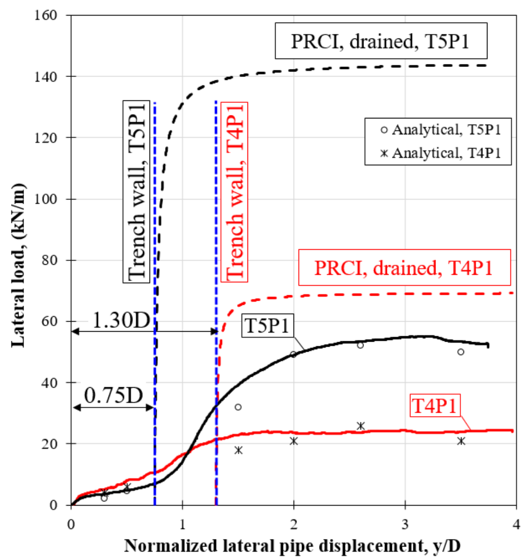

Figure 4.24. Comparison of analytical and PRCI predictions with test results

In Zone III, the analytical results are in closer agreement with test results when compared with existing hyperbolic solutions. However, taking into account that the analytical calculations have been conducted based on undrained conditions, if the parameters from partially drained conditions were incorporated, the results would be higher than those of drained conditions and closer to the test results in most places. Since the existing hyperbolic solutions overestimate the lateral p-y responses, the current study has shown that the stiffness of the backfill can play a significant role in the reduced resistance (ξ, Figure 4.23).

The effect of the passive pressure applied by the backfill against the collapsing trench wall is an area that still needs further investigation (currently ongoing by the authors).

Developing further sophisticated models that incorporate the effect of the collapsed trench wall, the passive backfill pressure, and most importantly, the failure mechanisms in Zone II can provide much more accurate predictions of the test results. More research is needed to investigate all of these details and improve the lateral response of trenched/backfilled pipelines to large displacements.

4.7 Conclusions

The effect of trenching/backfilling on the lateral soil resistance against the moving pipeline was investigated with different burial depth ratios. Internal soil deformations and failure mechanisms around the trenched/backfilled pipelines were investigated by using a transparent observation window and PIV analysis. To magnify the significance of pipeline-backfill-trench interaction, partially drained condition was adopted. Several important aspects were observed, and a couple of new research avenues were identified that are shortly summarized as below:

- Regardless of the backfill stiffness, the presence of the trench results in less ultimate lateral soil resistance (by ξ) against the pipe when approaching/penetrating the trench wall. This resistance reduction is due to the progressive collapse of the trench wall into the backfill. The magnitude of the reduction (ξ) depends on the stiffness of the backfill and the amount of passive lateral pressure and the buoyancy force that the backfill material mobilizes against the active trench collapse. Further investigation is needed to propose a model that incorporates the effect of backfill stiffness and buoyancy on ξ.

- The lateral soil resistance in the backfill is not purely governed by backfill material. The pipeline-trench bed interaction, including the magnitude of the initial pipe embedment into the trench bed and the lateral failure mode of the partially embedded pipe (immediate or slow breakout), makes a significant contribution to the lateral soil resistance. The backfill stiffness and its passive downward pressure against the developing soil berms in front of the pipe can have a significant impact on pipe-bed interaction and consequently on the ultimate lateral soil resistance. This important aspect has not been properly investigated in the past and needs further comprehensive investigations.
- The pipe-bed interaction and the squeezing of the trench bed material into the trench corner cause the pipe to move upwards, and approach/penetrate the trench wall in an oblique direction ($\alpha°$ from the horizon). The magnitude of α and consequently, the laterally mobilized soil wedge in the trench wall depends on several aspects of pipe-trench bed interaction. Different mechanisms were observed in shallow and deep trenches at the stage when the pipe approaches the trench wall. The complex pipe-backfill-trench interaction at this stage needs further examination to propose possible solutions.
- A spiral contracting mechanism was observed around the moving pipe inside the backfill that interacts with the trench wall and the bottom in a complex manner. This interaction turns the pipe into a virtual pipe moving inside the soil. The size of

this virtual pipe is larger than the real pipe when the pipe is approaching the trench wall and is less than the real pipe when the pipe penetrates into the trench wall. Proposing models to quantitatively predict these mechanisms warrants further study.

- The observations shortly referred above denote the potential influence of several parameters on lateral soil resistance against the largely displaced pipeline that have not previously been addressed or investigated. Some of these parameters include pipe weight, pipe type, backfill properties, backfill buoyancy, trenching and backfilling methodology, construction procedure, construction season, operational loads, thaw settlement and permafrost, longitudinal seabed profile, etc. A close examination into each of these parameters opens up new research avenues that considerably influence the ultimate lateral soil resistance against the large displacement of the pipelines.

4.8 Acknowledgments

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CHAPTER 5. The influence of pipeline–trenchbed interaction intensity on lateral soil resistance and failure mechanisms

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Abstract

Buried offshore pipelines may be subjected to large lateral displacements caused by ground movements, landslides, etc. A proper estimation of lateral soil resistance against the moving pipeline requires a deep understanding of pipeline–backfill–trench interaction mechanisms. Recent studies show that pipeline– trenchbed interaction can significantly affect the failure mechanisms and the resultant lateral soil resistance, an important aspect that is currently neglected by pipeline design codes. In this paper, the effect of pipeline–trenchbed interaction intensity on lateral soil resistance was investigated by performing centrifuge tests. The soil deformations and failure mechanisms were obtained by Particle Image Velocimetry (PIV) analysis. Two experiments with horizontal and downward inclined pulling directions were conducted to simulate different intensities of bed interaction. It was observed that increasing the intensity of pipeline–trenchbed interaction results in a faster and full development of shear bands in the trench wall and reduction of the mobilized lateral soil resistance.

Keywords: Lateral pipe-soil interaction; pipeline/trench-bed interaction, p-y response; large deformation; centrifuge testing

5.1 Introduction

Submarine pipelines continue to be trenched and buried in shallow waters to reduce the risk of damage from environmental, operational, and accidental threats. Buried pipelines may undergo large lateral displacements involving pipeline-backfill-trench interaction due to ground movements, landslides, ice gouging, and external impact of drag embedment anchors, etc. Pre-excavated material is commonly used as a cost-effective solution to backfill the pipeline laid inside the trench. These backfilling materials are usually remoulded to a high extent during the subsea trenching operation, resulting in a material which is much softer than the native ground. The previous studies have shown that the high difference between the stiffness of the backfill and native ground (i.e., trench wall and bed) combined with pipeline configuration inside the trench could significantly affect the soil failure mechanisms and consequently the lateral soil resistance against the relocated pipe (Paulin et al., 1998; Kianian et al., 2018). This significant effect is currently neglected by pipeline design codes (e.g., DNVGL-RP-F114, 2017; PRCI, 2009; ALA, 2005; and ASCE committee, 2014) assuming a uniform soil medium. The existing solutions are proposed based on various earlier studies on lateral pipe-soil interaction (Tschebotarioff, 1973; Wantland et al., 1979; Paulin, 1998) or anchor soil interactions (Hansen, 1948; Hansen and Christensen, 1961; Smith, 1962; Ovesen, 1964; Kosteyukov, 1967; Ovesen and Stromann, 1972; Neely et al., 1973; Das and Seeley, 1975; Rowe and Davis, 1982; Merrifield et al., 2001). In reality, the effect of pipeline-backfill-trench interaction compared to uniform soil condition is usually appeared as three different scenarios: a) the increased soil resistance against the limited pipeline displacements inside the backfill, b) the reduced soil resistance against the large pipeline displacements throughout the trench wall, and c) the increased soil resistance within the pipeline transition from backfill to trench wall (Paulin et al., 1998; Kianian et al., 2018). Figure 5.1 schematically shows these three scenarios.

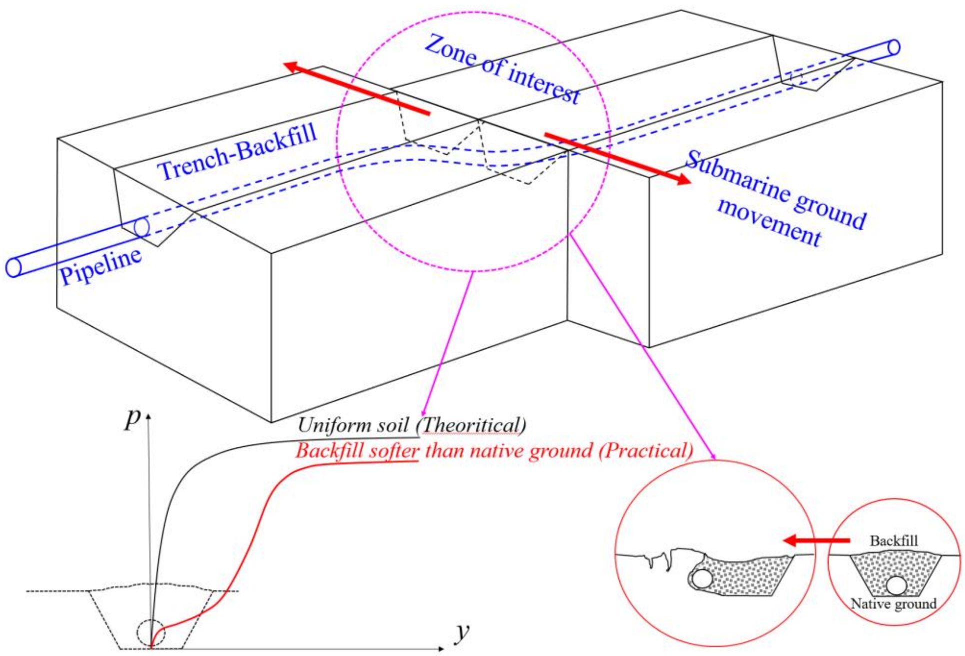

Figure 5.1. The lateral response of trenched pipeline to submarine ground movement

The pipeline laid inside the trench may achieve an initial embedment into the trench bed for different reasons such as pipe weight, construction method, environmental loads, etc. (Paulin et al., 1998; Cheuk et al., 2007). Also, the lateral displacement of the pipeline may slightly deviate from ideal horizontal direction, upward towards the backfill, or downward towards the trench bed due to the nature of external load, the longitudinal ground and pipeline profile, etc. The combination of initial embedment and the displacement orientation may result in different intensities of the pipeline-trenchbed interaction during the lateral displacement. Using the particle image velocimetry (PIV) analysis, Kianian et al. (2018) observed the potential impact of the pipeline-trenchbed interaction on lateral soil resistance against the moving pipe.

This important observation motivated the authors to conduct the current study and investigate the influence of pipeline-trenchbed interaction intensity on the lateral soil failure mechanism and the resultant soil resistance.

In this study, two centrifuge tests were conducted on pipelines shallowly trenched and buried in soft clay. An identical initial pipe embedment into the trenchbed was considered, and two cases of the lateral pipe displacement with horizontal and slightly downward inclination were considered to simulate the base case and higher intensity of pipeline-trenchbed interaction, respectively. The main objective of the study was qualitative assessment of the pipeline-trenchbed interaction intensity effect on the resultant failure mechanism and lateral soil resistance by using PIV analysis. Therefore, there was no need for a large number of tests that are usually conducted for quantitative studies. The partially drained condition was adopted by applying a low pipe moving velocity to magnify the pipeline-backfill-trench interaction effects.

It was observed that increasing the intensity of pipeline-trenchbed interaction by downward inclined pipe displacements resulted in faster and full development of shear bands in the trench wall and reduction of the mobilized lateral soil resistance. Inversely, the lowered bed interaction intensity by horizontal pipe displacement resulted in formation of a series of premature shear bands and mobilization of huge amount of lateral soil resistance. The study showed that the design practice could take advantage of pipeline-trenchbed interaction effect to improve the safety and cost-effectiveness of the buried pipelines design against the large lateral displacements. Further numerical and experimental studies are required for proposing quantitative design equations incorporating the pipeline-backfill-trench interaction effects on lateral soil resistance.

5.2 Modeling considerations and test set up

The testing program was conducted at C-CORE centrifuge facilities located at the St. John's campus of the Memorial University of Newfoundland. The main objective of the study was a qualitative assessment of the effect of pipeline-trenchbed interaction intensity on the failure mechanisms and soil deformations around the laterally displaced pipeline. The failure mechanisms and soil deformation were directly monitored through an acrylic transparent sidewall and analyzed by applying particle image velocimetry (PIV). The test apparatus was designed to conduct two tests (i.e., T3P1 (Horizontally pulled) and T4P1 (downward inclined pulled)) at the same time with two pipe sections pulled in opposite directions. Since the current study did not intend to quantify the effect of bed interaction on lateral soil response, performing only two tests was sufficient for a comparative and qualitative assessment. A quantitative study in the future would need performing more tests to consider the effect of different soil embedment, inclination scenarios, trench geometry, etc. Figure 5.2 shows the schematic configuration of the tests setup and the post-test side-view observed through transparent window.

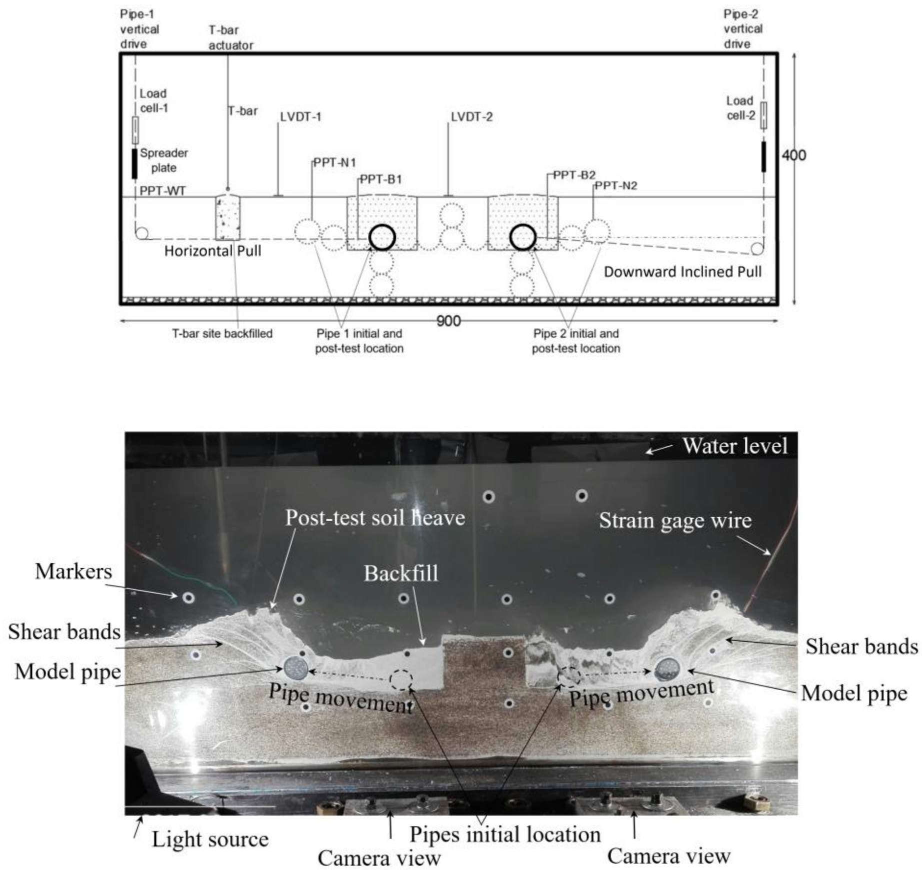

Figure 5.2. Schematic and real views of pre-test and post-test conditions (dimensions are in mm)

An average burial depth ratio of H/D = 1.5 was considered to model the shallow burial condition (H/D was defined as the initial ratio of the pipe springline depth to the pipe diameter). Very soft slurry backfill with rectangle trenches was used via partially drained condition (Normalized velocity, $vD/c_v = 0.3$, based on Phillips et al., 2004) to consolidate the surrounding soil, eliminate the effect of excess pore pressure and magnify the effect of pipeline-backfill-trench interaction. The test setup was developed in a similar approach undertaken by Paulin (1998) and Popescu et al. (1999 and 2002) to facilitate comparing the results with earlier studies. Also, the real pipe size was selected as 24” with an external diameter of 610 mm as a technical requirement by the project's industry sponsor to keep the continuation of their earlier full-scale studies in sand conducted by Burnett (2015) at Queens University. The stainless steel model pipe with a diameter of 31.75 mm was selected to enable accommodating the internal pressure transducer without folding. The pipe was instrumented with two sets of strain gauges, one internal pore pressure transducer (facing the rear of pipe), two strings of pulling cables, two rubber end caps (both lubricated, on patterned in window side). These sizing requirements resulted in setting a spinning acceleration of 19.1g. It is worth noting that centrifugal acceleration simulates gravity and allows for correspondence of stress fields between model and full-scale resulting in enhancement of modelling accuracy. However, to verify the scale effects, modelling of model tests at different "g" levels can also be conducted. Paulin (1998) conducted modelling of model tests and attempted to model the same prototype condition at 1:25, l:50 and 1:100 scales for the lateral response of trenched/backfilled pipelines in clay. The study yielded acceptable results with most of the interaction curves within a bandwidth of 0.5-1 normalized lateral loads. These results provided sufficient relaxation in the current study to avoid repeating the scaling studies, having a test set up almost identical to Paulin (1998).

Speswhite kaolin clay and Sil-Co-Sil silt were mixed by 50%-50% in weight with a sufficient amount of water to form a slurry with a nominal moisture content of about 70% to prepare the native ground. The native soil bed was incrementally consolidated up to the effective stress of 400 kPa and then was unloaded to 100 kPa with an open drainage valve. While removing the load, the water flows into the sample was restricted by closing the base drain and removing the excess water on top of the soil surface. This level of consolidation yielded a clay with an undrained shear strength of 15 to 25 kPa. Trenches 3.6D wide were excavated (1.3D clearance from trench wall in each side of the pipe), and a 2D clearance was considered between the trench bottom and the lower drainage layer at the bottom of the test box to ensure that there would be no boundary effects.

A T-bar penetrometer with a head bearing area of 30×7.4 mm^2 (Stewart and Randolph, 1994) was used to assess the strength profile of the backfill and native ground in-flight. Due to limited space on the test box and having access to only one T-bar actuator, the T-bar test site had to be designed in a way to allow assessing the backfill and native ground strengths in a single run. A T-bar site was excavated in the native ground and filled with slurry backfill material. The dimensions of the T-bar site was selected in a way to ensure the prevention of the boundary effects in different directions. The T-bar was first penetrating to the backfill material and then continued to penetrate into the native ground. A T-bar test with no excavation in native soil was also conducted to capture the pure shear strength profile of the native soil. A T-bar factor of $N_{kt} = 10.5$ was used (recommended by Stewart and Randolph, 1994, based on the plasticity solution earlier proposed by Randolph and Houlsby, 1984) to convert the measured unit bearing resistance, q, to the local undrained strength, s_u. For shallow depths, a reduced bearing factor considering the effect of soil buoyancy and a shallow failure mechanism mobilized prior to the full flow of soil around the bar was used to transform the measured bearing resistance to the undrained shear strength (White et al., 2010). The reduced bearing factor increases the shear strength profile that was obtained by the factor 10.5. This profile is considered for shallow penetrations. Approximate linear s_u profiles were fitted for both the backfill and native soils to facilitate the back-analysis of the test data. Figure 5.3 shows the schematic configuration of the T-bar site, and the measured undrained shear strength profiles with linear curve fits in T3P1 and T4P1 tests.

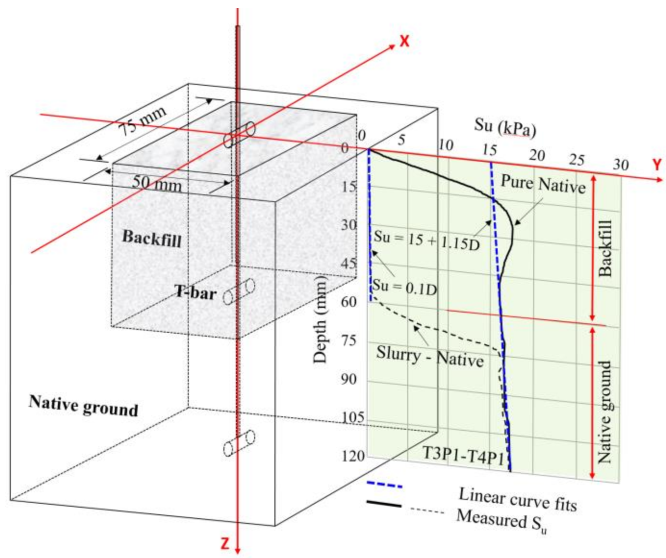

Figure 5.3 Schematic configuration of the T-bar site and the measured undrained shear strength profiles with linear curve fits

Table 5.1 shows a summary of the backfill and native soil prepared and tested in this study.

Table 5.1. Soil properties

Test ID	Trench backfill type	Native c_u at pipe SL (kPa)	Native water content before and after the test (%)	Υ_{sat} (kN/m3)	Native soil mudline strength, S_{um} (kPa)	Native soil Strength gradient, k_{su} (kPa/m)	Backfill mudline strength, S_{um} (kPa)	Backfill strength gradient, k_{su} (kPa/m)
T3P1	Slurry	16.8	32.04 - 32.97	18.51	15	1.15	0	0.1
T4P1	Slurry	16.7	30.12 - 32.13	18.45	15	1.15	0	0.1

Figure 5.3 shows that the undrained shear strength in slurry backfills is almost negligible. The native soil located underneath the backfill material showed a slightly softer response in the initial stages of penetration. This is due to the slight water dissipation from the

backfill to the native soil. By increasing the penetration, the plots of overlaid native soil strengths are gradually matching the profile of pure native soil.

Before running the tests, a range of instruments were used along with 3 parallel data acquisition systems to monitor the testing program. The instruments included pore pressure transducers (PPTs), strain gauges, load cells, conventional and laser linear variable differential transformers (LVDTs), T-bar, vertical drive motion controller, digital cameras, markers, and artificial textures. The instrumentation allowed the full capturing of progressive failure mechanisms and the development of shear bands in backfill and native soil, the lateral force-displacement response of the pipeline, the suction force variation behind the moving pipe, and the pore pressure variation both in the backfill and native ground. The internal walls of the strongbox were lubricated to eliminate the friction between the end caps and the box walls as much as possible. A trial pipe pulling was also conducted without soil bed, and the pulling load was measured to ensure that the friction has been properly suppressed. In order to eliminate the friction between the cable and the soil, two strain gauges were installed on each pipe to purely capture the lateral soil resistance right in front of the pipe. In addition, load cells were installed on pulling cables to double-check the results obtained from the strain gauges. The load cells captured the friction forces as well, including the friction between the end caps and the box walls, the friction between the cable and pulley, and the friction between the cable and soil. The results presented throughout the paper were obtained from strain gauges.

The pipes were laterally pulled in opposite directions using pulling cables passed through the fixed pulleys. The bottom elevation of the pulley was aligned with the pipe springline to ensure a horizontal pulley in test T3P1. The pulley was lowered by 3.0 mm in test T4P1 compared to the elevation of pipe springline to apply a downward inclined pulling orientation and intensify the pipeline-trenchbed interaction relative to T3P1. In reality, non-horizontal pipe displacement may happen due to convex or concave seabed profile, the direction of applied load such as drag anchors, the inherent tendency of deflected pipe to return to the longitudinal axis, etc. Figure 5.4 schematically shows the different intensity of pipeline-trenchbed interaction under horizontal pulling (T3P1) and downward inclined pulling (T4P1).

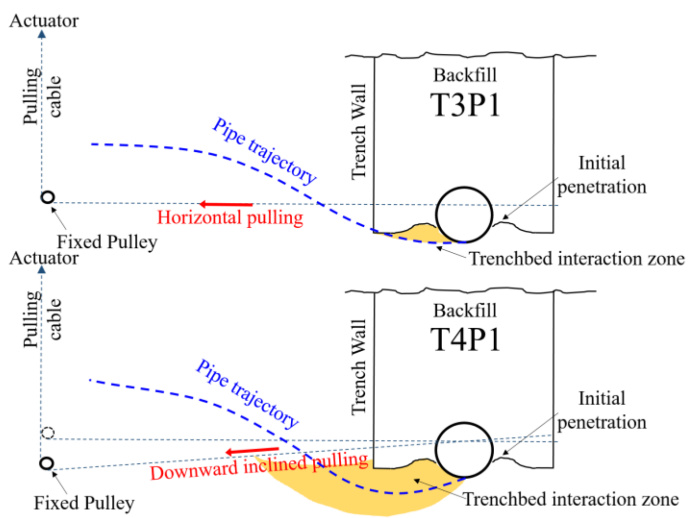

Figure 5.4. Pipeline-trenchbed interaction intensity in horizontal and downward pulling

In this study, the pipes were pulled under displacement control conditions but were free to displace vertically. For a precise evaluation of the purely lateral soil resistance, the measured lateral forces are better to be calibrated against the instantaneous pipe trajectory.

This would slightly improve the accuracy of the current results (by about $\pm$ 0.4% in the proposed configuration for T3P1 and T4P1).

5.3 Experimental study results

In this section, the internal soil deformations and the failure mechanisms observed during the tests are investigated alongside the recorded lateral soil resistance. PIV analysis was conducted by using the high-quality images captured by digital cameras and the GeoPIV-RG software (White et al., 2003; Stanier et al., 2016) to obtain the soil displacements and strain levels. Several interactive soil deformation and failure mechanisms were observed in the various stages of pipeline lateral displacements that are significantly affected by the pulling load direction or the intensity of pipeline-trenchbed interaction. These mechanisms can be investigated in three different zones (zone I to III) of lateral pipeline displacements (see Figure 5.5). Zones I ,II , and III refer to i) the pipeline moving inside the backfill, ii) the pipeline in transit from backfill to trench wall, and iii) the pipeline fully penetrated into trench wall, respectively.

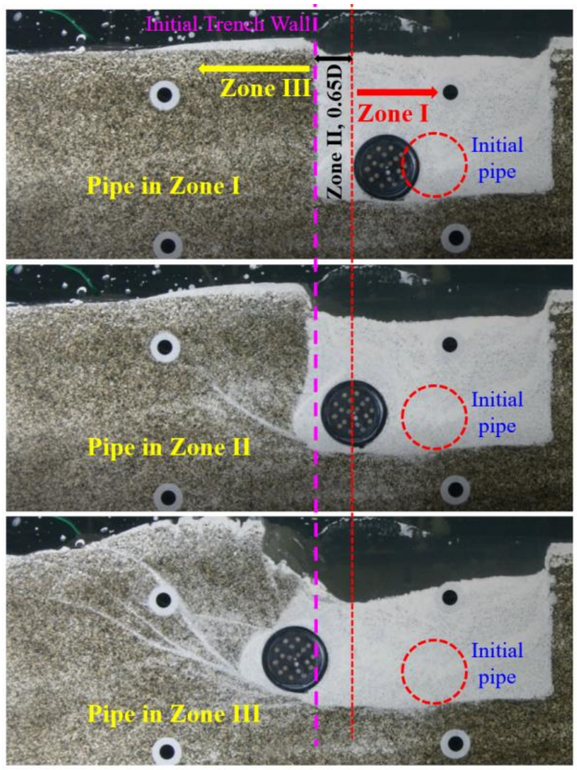

Figure 5.5. Identified zones of pipeline displacements with different failure mechanisms

Zone I, pipeline small displacement inside the trench

Starting the centrifuge spinning, the pipeline was slightly penetrated into the trench bed (0.1D). This was accounted for adjustment of the pulley elevation in both tests to achieve the target inclination of the pulling cable. While the pipe was moving in the zone I, the pipeline interacted with backfill and trenchbed, both of which could contribute to the mobilized lateral soil resistance. The contribution of pipeline-backfill interaction was minimized by using a very soft slurry backfill with almost nil shear strength resistance. This magnified the contribution of the pipeline-trenchbed interaction and enabled the study of its intensity on lateral soil resistance, which was the main objective of the current study.

Figure 5.6 compares the lateral soil resistance and soil deformations in zone I for tests T3P1 (horizontal pulling) and T4P1 (downward inclined pulling). It is usually appropriate to normalize the load with pipe diameter and undrained shear strength to obtain the interaction factor. In this case, normalizing the load with highly different magnitude of undrained shear strength in backfill and native will result in discontinuities in p-y response, particularly with the complex form of interaction in the transition from backfill to native soil. Therefore, the load response was not normalized in Figure 5.6 for a proper presentation of p-y response.

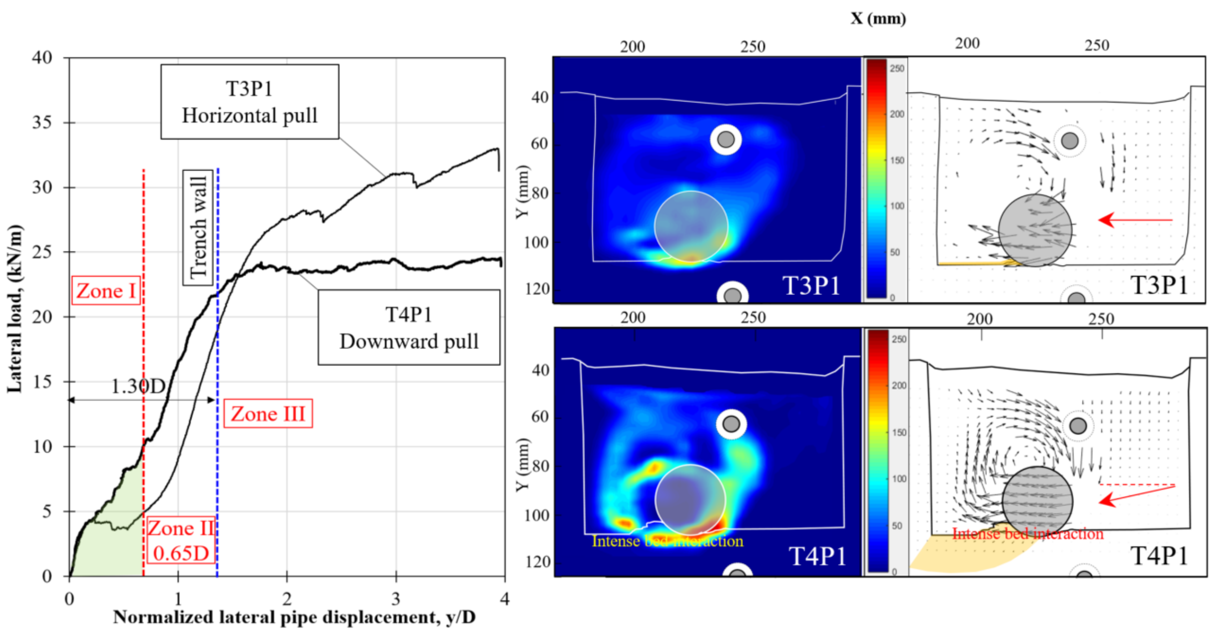

Figure 5.6. p-y response and a sample of total shear strain contours and soil displacement field in zone I.

As shown in Figure 5.6, the p-y response started with a very close lateral force in both of the tests (thick and thin solid lines) until about 0.2D displacement. This represents the pipeline interaction with the slight side berm created under initial pipe penetration into the trenchbed. The PIV results show the higher intensity of bed interaction in T4P1 with downward inclined pulling direction. With further pulling, a basal shear happened under the side berm in T3P1 with horizontal pulling, and the lateral force turned to become constant in zone I (0.65D)(the thin solid line in p-y curve, Figure 5.6). In T4P1 with downward inclined pulling direction, the lateral force kept increasing (the thick solid line in p-y curve, Figure 5.6). Therefore, as expected, a higher lateral resistance was generated in test T4P1 due to higher bed interaction intensity.

Although the contribution of pipeline-backfill interaction minimized by using a very soft slurry backfill, but a spiral soil flow was observed around the pipe section moving inside the trench. This soil deformation mechanism that comprises loops of eccentric spiral soil flows with rotational circles around the moving pipe could be significantly important, if the backfill had a remarkable stiffness and considerably contributed to the lateral soil resistance. In these tests with negligible backfill stiffness, it was observed by PIV analysis that the spiral flow surfaces emanate from a point above the pipe and move horizontally with the pipe until the failure surface touches the trench wall. From this stage, with a further displacement of the pipe towards the trench wall, the spiral flow starts to contract with a varying ratio that depends on its distance to the wall; the closer to the wall, the smaller the failure circle will be. Figure 5.7schematically shows the spiral flow mechanism observed in the conducted tests.

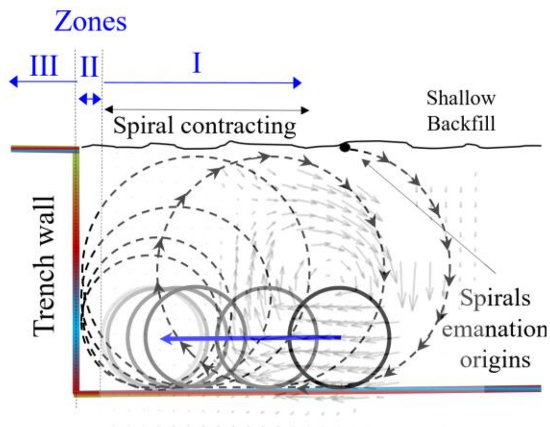

Figure 5.7. Contracting spiral flow mechanism of slurry backfill

Looking at progressively contracting spiral flows, the contribution of pipeline-backfill interaction to the lateral load is dissipated due to the reduction of the magnitude of mobilized rotational shear stress in the backfill. However, the obtained p-y responses show a continuous increase in lateral load due to the aforementioned pipeline-trenchbed interaction process (see Figure 5.6). This important soil deformation mechanism still needs to be further investigated by using different range of backfill stiffness and trench configuration in the future studies for an accurate assessment of lateral soil resistance against the pipeline moving inside the trench.

Zone II, pipeline approaching the trench wall

Zone II was started as soon as the pipeline displacement inside the trench causes the trench wall to be displaced as well. This zone was ended by arriving at the pipe front at the initial position of the trench wall (see Figure 5.5). The total length of zone II in this study was about 0.65D that might change for different trench configurations and test setups. During the pipeline displacement in zone II, the soil deformations in the trenchbed, backfill, and trench wall were interactively developed. The PIV analysis results showed that the soil berm in front of the pipe is displaced/enlarged in T3P1/T4P1 and squeezed into the bottom corner of trench creating a barrier between the pipe and trench wall. However, due to the basal shear in T3P1 (horizontal pulling condition), the volume of soil berm was not increased and only translated towards the trench corner. Despite T3P1, the soil berm in T4P1 was gradually enlarged because of more intense bed interaction caused by downward inclined pulling orientation. The larger soil barrier squeezed between the pipe and trench wall in T4P1 caused the pipe to convert to a virtual pipe with a diameter larger than the real pipe. This virtual pipe pushed into the trench wall in T4P1 resulted in the formation of the first failure shear band in the trench wall much earlier than T3P1 and before arriving the pipe to the end of zone II or the initial trench wall location. The formation mechanism of the larger virtual pipe in T4P1 and its influence on the lateral p-y response is shown in Figure 5.8. Also, the comparison of strain levels shows that the larger soil berm formation in T4P1 blocks the spiral backfill flow inside the trench.

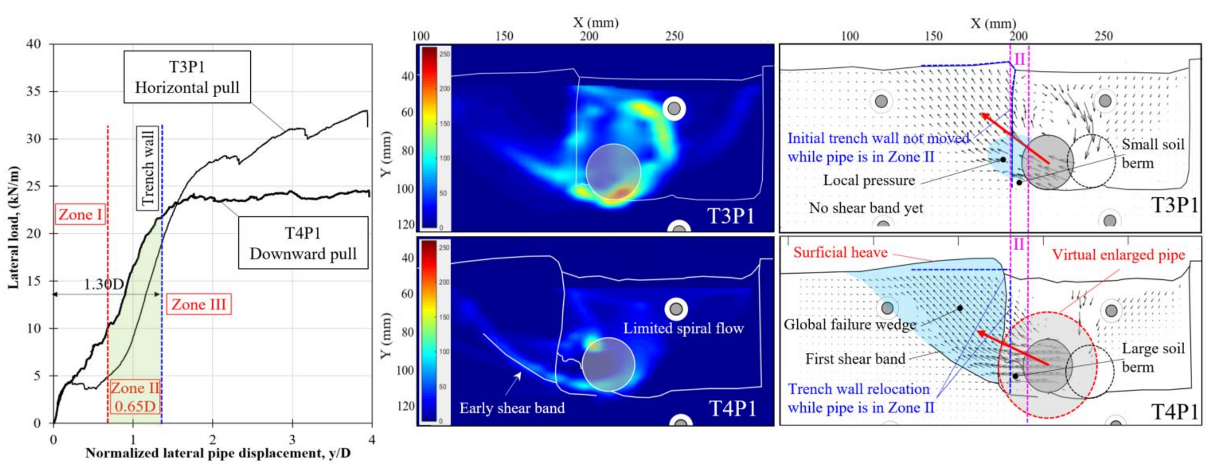

Figure 5.8. p-y response and sample of strain levels and soil displacement in zone II.

The virtual pipe in T4P1 with a larger imaginary diameter has mobilized a larger amount of native ground and resulted in a higher lateral soil resistance in comparison with T3P1 as highlighted by green colour in Figure 5.8.

It is worth having a closer look at the pipes trajectories both in T3P1 and T4P1 to identify its relationship with the trends observed in p-y responses. Figure 5.9 shows the pipe trajectory against the p-y responses. Both in T3P1 and T4P1 with horizontal and downward pulling direction, having no vertical restriction, the pipes slightly penetrated into the trench bed in zone I, due to pipe weight and then naturally followed the route with least work, i.e., upward towards the backfill with softer material (curves 1 and 2). This showed that the pipelines are both penetrated into the trench wall in an oblique direction. The vertical displacements were continued in zone II as well (see Figure 5.9).

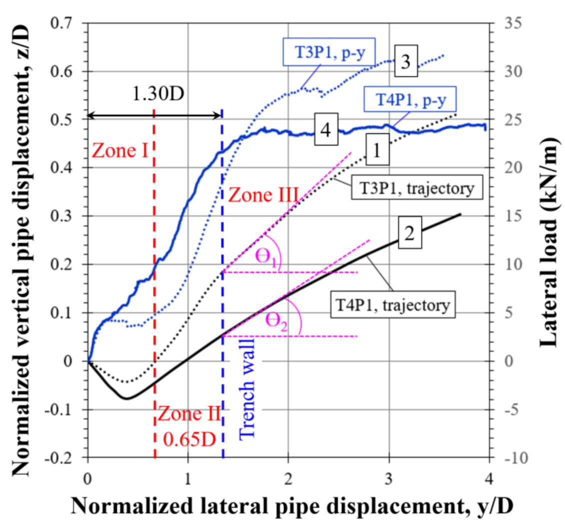

Figure 5.9. Pipeline trajectory against the p-y response

The most interesting trend was observed in the pipe trajectories beyond the trench wall and its relationship with the p-y curves (curves 3 and 4). Comparing the plots No. 1 and 2, the pipe in T3P1 entered into the zone III with a steeper slope relative to T4P1 ($\Theta_1 > \Theta_2$, Figure 5.9). The steeper angle in T3P1 was expected to result in a smaller lateral soil mobilization in native ground and a less lateral soil resistance, consequently. However, the observed results were completely inverse, where the lateral resistance in T3P1 in zone II was surprisingly larger than T4P1 that had a lower angle of penetration to the trench wall. To identify the reason behind this interesting trend the failure mechanisms in zone III need to be investigated. Figure 5.10 schematically shows the subtle difference between the intensity of the bed interaction in T3P1 and T4P1 and its consequences on pipeline-trench wall interaction resulting in the ultimate soil resistance.

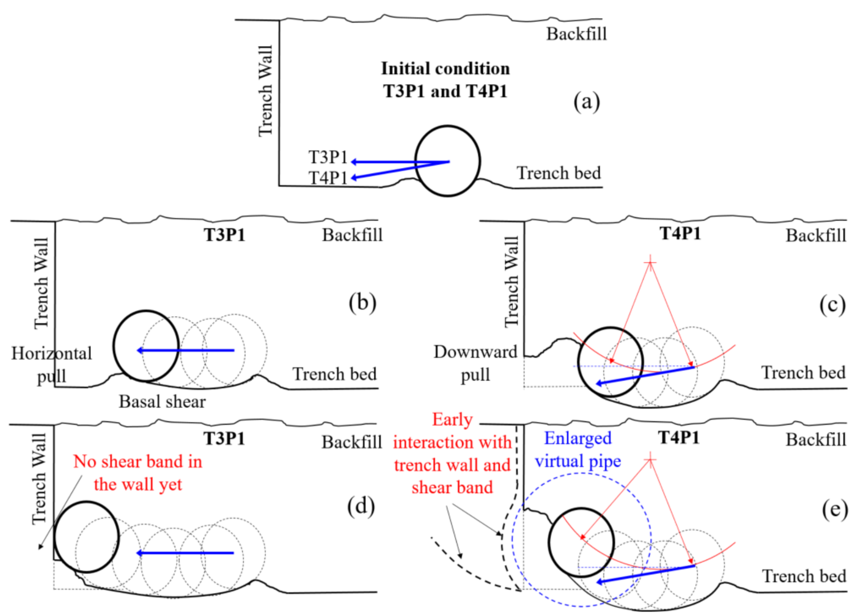

Figure 5.10. Schematic illustration of bed interaction process in horizontal and downward pulls

It will be shown in the next section (zone III) that how the earlier interaction of the pipeline and trench wall in T4P1, and the faster formation of the first shear band in the wall reverse the trends observed in p-y response and causes the soil resistance to drop down below the T3P1 (thick solid line in Figure 5.8).

Zone III, pipeline penetrating into the trench wall

Zone III starts by arriving at the pipe front at the initial trench wall location. At this stage, a small triangular wedge was created in front of the pipe, while the logarithmic spiral shear band in the trench wall was faster developed in T4P1. It was observed that the orientation of this shear band with the horizontal line depends on the orientation of the pipeline displacement and almost follows the same orientation as the pipeline. As the pipe moved forward in T4P1 (downward pulling), the global shear band in the trench wall was entirely developed towards the ground surface, and a large passive block was created in the top-front area of the pipe (see Figure 5.11). Arriving the shear band to the ground surface releases the stress in soil to some extent and does not allow for successive accumulation of the soil resistance against the pipe.

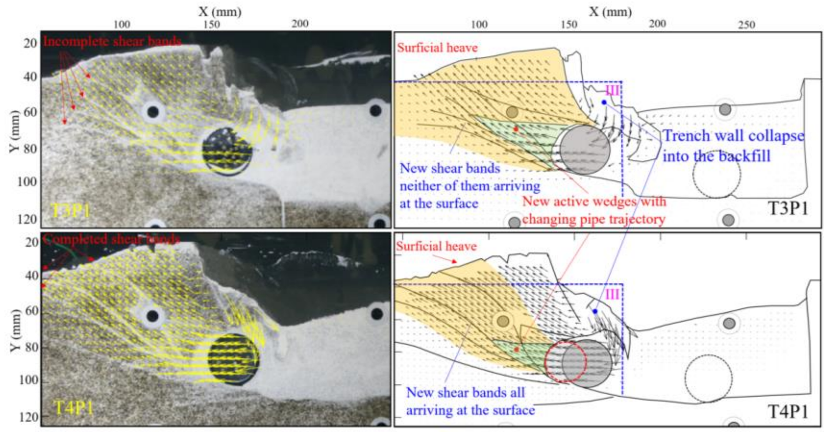

Figure 5.11. Sample of PIV results and soil displacement in zone III.

As shown in Figure 5.11, as pipe moved forward, the cable angle offset from the horizontal line was increased, and this affected the pipe trajectory by pulling it down. By changing the pipe trajectory, a new global shear band formed underneath the pipe with a new orientation. The first global instability of the trench wall appeared on top of the pipe, as soon as the top-front edge of the real pipe touched the native trench wall. This caused a slice of the native soil to slip down into the backfill. The base of the failed slice asymptotically reached the top of the pipe in the form of a spiral surface. This mechanism was fairly similar in T3P1 and T4P1.

A key difference was observed between the T3P1 and T4P1 in the progressive formation of the global shear bands that revealed the reason behind reversing the trend of p-y responses in zone III. It was observed that in T3P1 (horizontal pulling), these global shear bands were progressively developed same as T4P1. However, the new shear bands forming due to progressive changing of the pipe trajectory was started before completion of the earlier shear band and its arrival at the ground surface. Therefore, either of shear bands never arrived at the ground surface, and this happened due to a delayed start at the beginning. The incompletion of failure shear bands caused a significant accumulation of the lateral soil resistance and a large soil heave formation at the surface. This, in turn, resulted in reversing the p-y variation trend and dramatic increasing of the lateral soil resistance in T3P1 within the zone III (see Figure 5.11).

It is also worth mentioning that the continuous changing of the pipe trajectory caused the progressive creation of a new triangle active wedges with an inclination angle that was gradually reduced following the pipe's trajectory. The observed isosceles triangle, which is similar to Terzhaghi's active zone under a footing, had a different size and direction in T3P1 and T4P1 due to different trajectories and interactions with the trench wall. As the pipe proceeded into Zone III, the upper side of the small active wedge truncated the virtual pipe and shrunk its diameter to get even smaller than the real pipe (see Figure 5.11). The shear stress around this dead wedge needs to be considered in back analysis of the test results that will be presented in later sections.

Pore pressure measurement

Pore pressure transducers (PPTs) were placed inside the pipe and in several locations (see Figure 5.2) inside the backfill and native ground to capture the pore pressure variation in the surrounding soil, particularly the suction force mobilization behind the pipe. A surface PPT was also used to monitor the water table in the testbed. Figure 5.12 shows the variation of pore pressure against the pipe displacement in the rear of the pipe (Internal PPT), native ground (NG), and in backfills (BF).

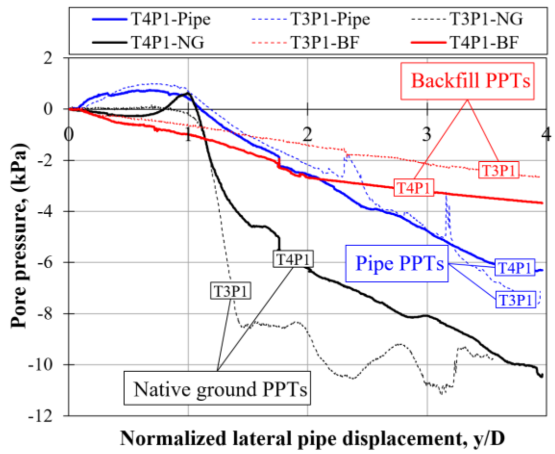

Figure 5.12. Pore pressure variation in backfill (BF), native ground (NG), and rear of the pipe

The internal PPTs (Blue plots in Figure 5.12) showed an initial increase of the pore pressure in the zone I followed by the dissipation of the excess pore pressure and developing a slight suction force behind the pipe in zones II and III. The magnitude of this suction force is not significant due to the low displacement rate of the pipe in a partially drained test condition. The PPTs inside the backfill (Red plots in Figure 5.12) showed almost a similar trend but with a lower magnitude of suction force mobilization due to the PPT offset from the pipe. The PPTs inside the native ground (Black plots in Figure 5.12) showed a slight negative pressure at the beginning that was the sign of pipeline-trenchbed interaction resulting in shear surfaces at the trenchbed. The pore pressure was then followed by an excess pore pressure at the end of the zone I due to starting the compression of native ground. Then the pore pressure was dissipated and turned to suction force mobilization in zone II and III due to the soil failure through the shear bands. This pore pressure variation trend was almost similar to the expansion of an over-consolidated clay under the drained triaxial test. The larger magnitude of suction force in zone III of the T3P1 and fluctuation of the pore pressure was in agreement with the soil failure mechanisms and progressive development of incomplete shear bands in the native ground due to delayed interaction of the pipe section and the trench wall, and consequently the p-y responses showed in Figure 5.11.

5.4 Existing solutions for the lateral p-y response of buried pipelines

The p-y responses in T3P1 and T4P1 were compared with the solutions recommended by some of the most popular pipeline design codes (i.e., PRCI, 2009; ALA, 2005; and ASCE committee, 2014). The comparison was independently made for backfill and native ground assuming a uniform soil in zone I and III since there is still no systematic solution or recommendation by design codes to consider the effect of trenching and backfilling on lateral soil resistance. Therefore, no comparison was made for the p-y responses in zone II. Figure 5.13 shows the p-y comparison both in drained and undrained conditions depending on the applicability. The soil strength parameters for the undrained condition were extracted from Figure 5.3, and the drained parameters were adopted from the tri-axial tests (Paulin, 1998). The PRCI (2009) and ALA (2005) guidelines are considering both drained and undrained conditions by superposing the contribution of effective friction angle and effective cohesion for drained condition and considering the undrained shear strength for the undrained condition. The methodology proposed by the ASCE Committee on thrust restraint design of buried pipelines (ASCE committee, 2014) provides separate equations for sand and clay using the friction angle and undrained shear strength, respectively. To use the recommended equations for clay under drained conditions, the effective cohesion was disregarded. For sand, the effective friction angle was used in proposed equations. Unlike ASCE committee (2014), the formulations recommended by PRCI (2009) and ALA (2005) do not cover the low range of friction angles preventing them from being used for very soft slurry backfill used in this study.

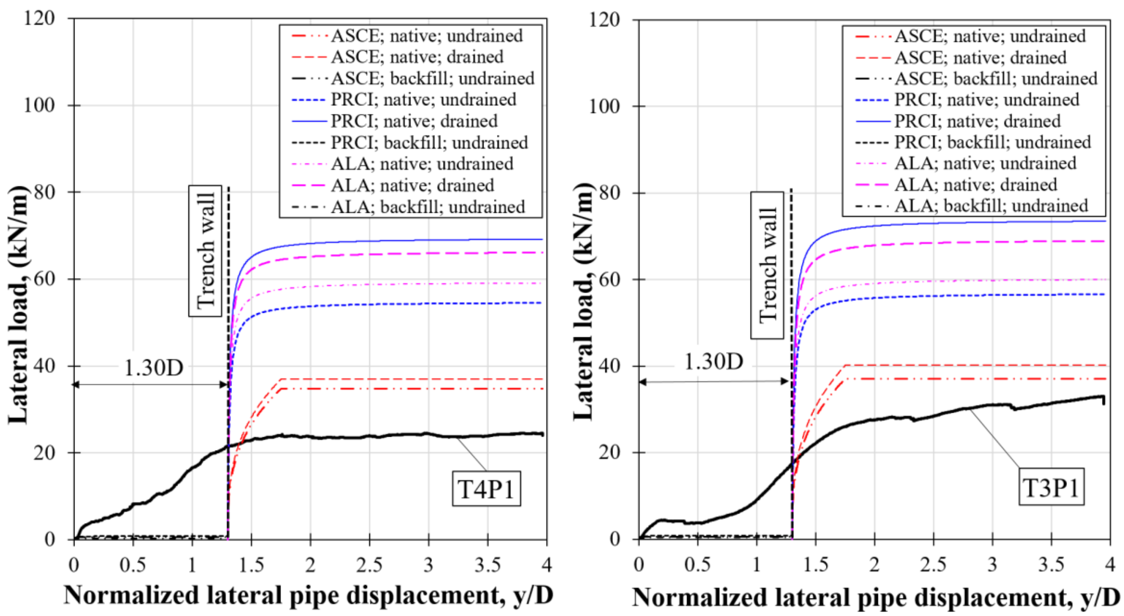

Figure 5.13. The comparison of the p-y responses between the test results and design codes

Figure 5.13 shows that neglecting the trenching effect could result in a significant underestimation of the lateral soil resistance in the zone I and overestimation in zone III. The PIV results and the observed failure mechanism reveals two different analysis for these differences. The underestimation of the soil resistance inside the backfill (zone I) is due to neglecting the pipeline-trenchbed interaction. Indeed, a lateral resistance of about 5 kN/m inside the backfill is not provided by backfill shear strength (very soft clay with nil shear strength). This lateral soil resistance is mobilized by pipeline-trenchbed interaction. The overestimation of ultimate soil resistance in native ground (zone III) is because of the trench effect, where the passive pressure against the collapsing trench wall is largely released by a backfill softer than native ground. Both of these effects are currently neglected by design codes (e.g., DNVGL-RP-F114, 2017; PRCI, 2009; ALA, 2005; and ASCE committee, 2014) due to less explored soil deformation mechanisms. Also, Figure 5.13 shows that the design codes underestimate the lateral soil resistance in zone II or transition from backfill to native ground. The underestimation of lateral soil resistance in zone I and II by design codes need particular attention since the majority of pipeline displacements occur in these ranges. A proper estimation of the soil resistance in these domains could result in more cost-effective design. Figure 5.14 schematically shows the comparison of the observed soil resistance trends.

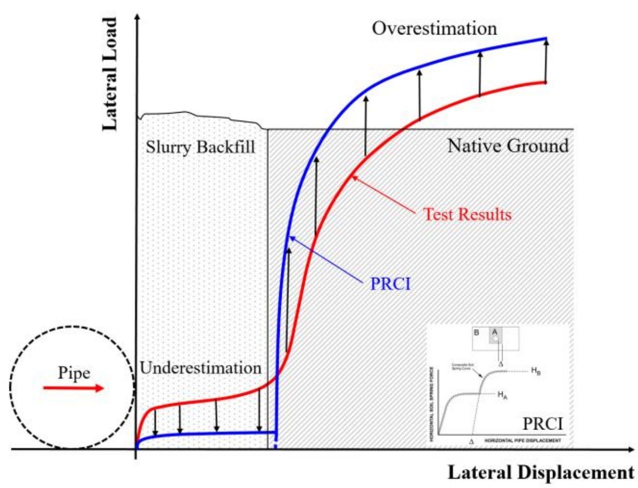

Figure 5.14. Schematic illustration of the difference between the p-y curves predicted by design code and experimental observations

To further assess the obtained test results, the interaction factors or the normalized lateral loads of the conducted tests against the burial depth ratio were roughly compared with some of the key published studies in Figure 5.15. Besides the burial depth, the interaction factors are affected by some other parameters. For instance, the interaction factor may be regarded as a function of dimensionless overburden pressure (Rowe and Davis, 1982; and Merifield et al., 2001). The non-uniqueness of the N_c (interaction factor) versus H/D relation may also be induced by combining the effect of pipe size, model scale, burial depth H, or a more general stress level (Guo and Stolle, 2005). A better comparison needs considering the effect of displacement rate, depth, trench slope, strength parameters of both backfill and native, etc.

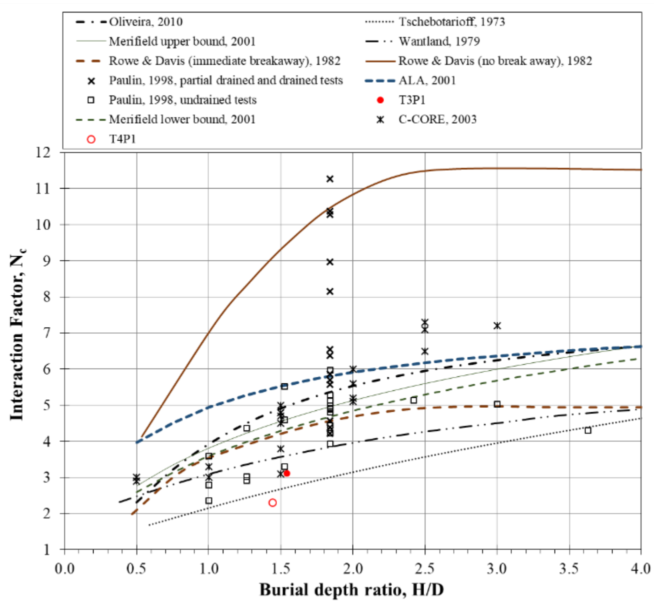

Figure 5.15. Interaction factor against the burial depth ratio

Figure 5.15 shows an agreement between the current study and the work conducted by Paulin (1998). The predictions made by Wantland et al. (1979) and Tschebotarioff (1973) were also fairly close to the conducted test results. The plasticity solution proposed by Merifield et al. (2001) for anchor and the equations recommended by ALA (2005) overestimated the lateral soil resistance. The overestimation is due to ignoring the pipeline-backfill-trench interaction effect on failure mechanisms and consequently, on the ultimate soil resistance, as shown in Figure 5.13 and Figure 5.14. A similar overestimation trend was observed in the predictions made by Oliveira et al. (2010).

5.5 Analytical approximation of the soil resistance

There is still no plasticity solution for the large lateral displacement of either trenched/backfilled pipelines or even the pipe in a uniform cohesive soil. However, the results of the PIV analysis and the observed mechanisms combined with the recorded lateral loads in Zone I, II, and III were used to approximate the mobilized soil resistance in a couple of instant spots in Zone I and III.

For the pipe inside the trench, the backfill was eliminated because of having no shear strength. The lateral and breakout soil resistance in partially embedded pipeline has been widely investigated in the past using numerical (Merifield et al., 2008; Wang et al., 2010; Chatterjee et al., 2012), experimental (Cheuk et al., 2007; White and Randolph, 2007; Dingle et al., 2008), and analytical (Randolph and White, 2008) investigations. In all of these studies, the pipeline is partially exposed to the water or air on its top, as assumed in the current study. However, in the case of the buried pipelines, the presence of backfill and its passive pressure against the side berm development would affect the lateral soil resistance. There is still no study in the literature to consider these effects. Therefore, it was assumed that the pipeline/trenchbed interaction (partially embedded pipe) was the only mechanism acting in zone I.

In Zone III, the passive pressure applied by the backfill was also neglected, and only the contribution of the active wedges and shear bands in the native ground was accounted for. In Zone II, the lateral resistance was simply obtained by matching the values at the end of the Zone I and at the beginning of Zone III. Figure 5.16 schematically shows samples of the assumed simplified mechanisms and the relationship between the mechanism in Zone I, II, and III for test T3P1 (horizontal pulling) and T4P1 (downward inclined pulling).

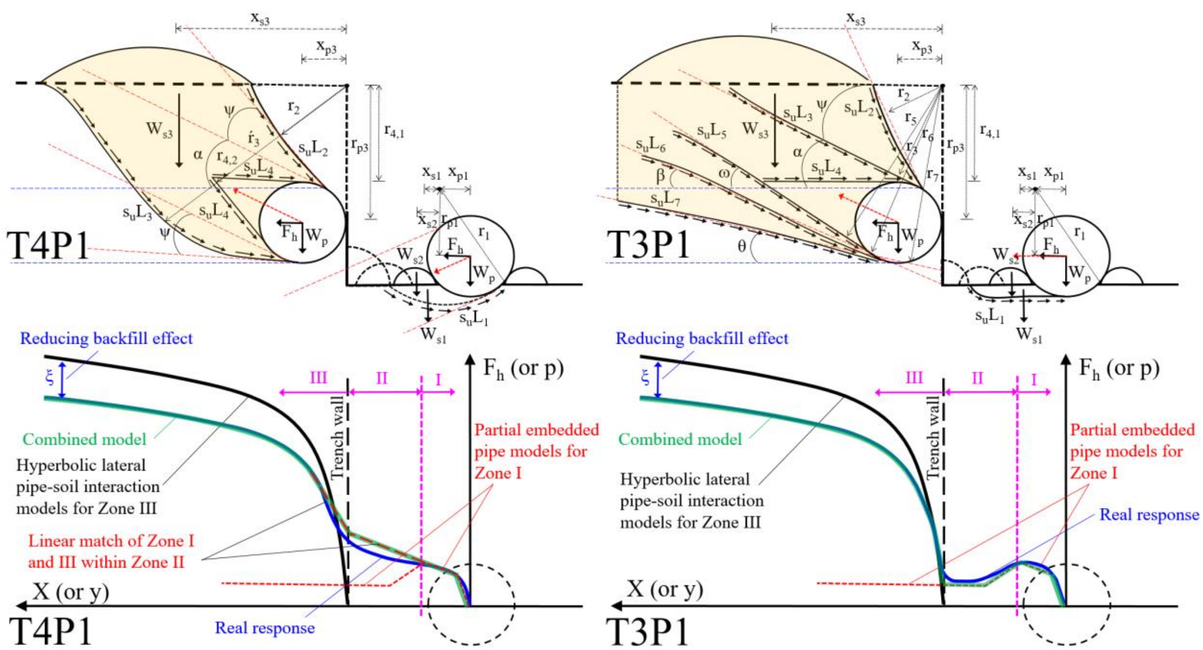

Figure 5.16. Schematic view of the simplified mechanisms in T3P1 and T4P1

Using Figure 5.16, the horizontal force (F_h) required for lateral pipe displacement in Zone I and III can be given by following the basic equations:

$$F_{h-T3P1\,(Zone\ I)} = \frac{S_u L_1 r_1 - W_p X_{p1} + W_{s1} X_{s1} + W_{s2} X_{s2}}{r_{p1}} \tag{5.1}$$

$$F_{h-T3p1\,(Zone\ III)} = \frac{S_u (L_2 r_2 + L_3 r_3 + L_4 r_{4,1} + L_5 r_5 + L_6 r_6 + L_7 r_7) - W_p X_{p3} - W_{s3} X_{s3}}{r_{p3}} \tag{5.2}$$

$$F_{\mathrm{h}-T4P1\,(Zone\ I)} = \frac{S_u L_1 r_1 - W_p X_{p1} + W_{s1} X_{s1} + W_{s2} X_{s2}}{r_{p1}} \tag{5.3}$$

$$F_{h-T4p1\,(Zone\ III)} = \frac{S_u (L_2 r_2 + L_3 \acute{r}_3 + L_4 (r_{4,1} + r_{4,2})) - W_p X_{p3} - W_{s3} X_{s3}}{r_{p3}} \tag{5.4}$$

Figure 5.17 compares the analytical approximation of the soil resistance in a few points in Zone I and III with test results and ASCE predictions. The good correlation of the analytical approximation with the test results shows that the governing mechanism in Zone I is the pipeline-trenchbed interaction, not the pipe-backfill interaction (assuming a very soft slurry backfill).

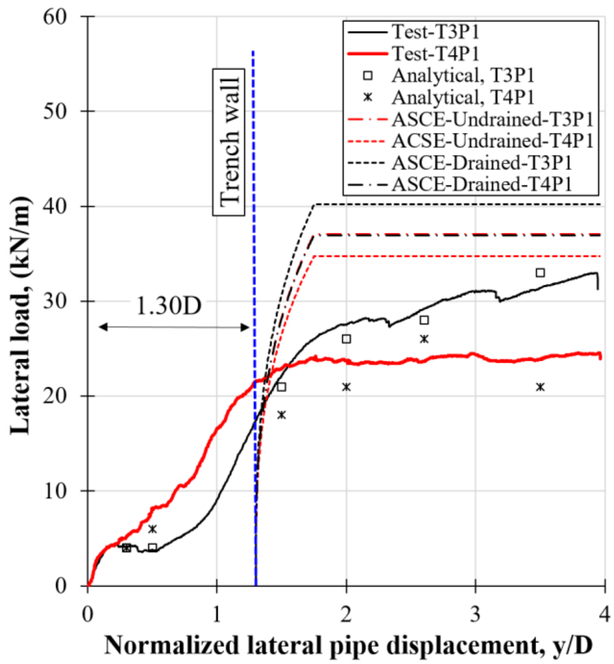

Figure 5.17. Comparison of analytical and ASCE predictions with test results

In Zone III, the analytical results are also in a close agreement with test results compared with existing hyperbolic solutions. The comparison shows that the analytical approximation based on the observed mechanisms could closely produce the observed test results. However, taking into account that the analytical calculations have been conducted based on undrained conditions, if the parameters from partially drained conditions were incorporated, the results would be higher than those of drained conditions and would be closer to the test results in most places. Since the existing hyperbolic solutions overestimate the lateral p-y responses, the current study showed that the stiffness of the backfill could significantly affect the reduction of lateral soil resistance (ξ, Figure 5.17). The effect of the passive pressure applied by the backfill against the collapsing trench wall is an area that still needs further investigation.

Developing further sophisticated models that incorporate the effect of the collapsed trench wall, the passive backfill pressure, and most importantly, the failure mechanisms in Zone II can provide much more accurate predictions of the test results. More research works are required to investigate all of these details and improve the prediction of the lateral response of trenched/backfilled pipelines to the large displacements.

5.6 Conclusions

The effect of pipeline-trenchbed interaction intensity on the lateral soil resistance against the moving pipeline was investigated by performing two small-scale centrifuge tests. Horizontal and downward inclined pulling directions were adopted to produce difference bed interaction intensities. Internal soil deformations and failure mechanisms in the soil surrounding the pipe were investigated by using a transparent observation window and PIV analysis. The partially drained condition was adopted to magnify the significance of pipeline-backfill-trench interaction. Several important aspects were observed that are shortly summarized as below:

- Pipeline-trenchbed interaction significantly contributes to the lateral soil resistance against the pipe moving inside the trench, and also the ultimate soil resistance against the pipe penetrating into the trench wall.
- More severe pipeline-trenchbed interaction results in an earlier shear band formation in the trench wall, arriving the failure surfaces to the ground surface, and consequently less magnitude of ultimate soil resistance. The shear band initiation in the trench wall is delayed by reducing the intensity of pipeline-trenchbed

interaction. This, in turn, results in a series of premature failure surfaces that never arrive at the ground surface and accumulate a larger amount of lateral soil resistance against the pipe penetrating into the trench wall.

- The intensity of the pipeline-trenchbed interaction is governed by the pipeline moving direction and the initial embedment into the trenchbed. These two parameters can be affected by several aspects such as pipe weight, pipe type, pipe laying method, construction season, construction procedure, trenching and backfilling methodologies, longitudinal ground profile, seabed soil properties, environmental loads, operational loads, etc. Therefore, an accurate prediction of the lateral soil resistance needs to account for the project-specific conditions.
- Regardless of the backfill stiffness, the presence of the trench results in less ultimate lateral soil resistance against the pipe when approaching/penetrating the trench wall. This resistance reduction is due to the progressive collapse of the trench wall into the backfill. The magnitude of the reduction depends on the stiffness of the backfill and the amount of passive lateral pressure and the buoyancy force that the backfill material mobilizes against the active trench collapse. Further investigation is needed to propose a model that incorporates the effect of backfill stiffness and buoyancy on the resistance reduction.

Overall, the study showed a significant effect of trenching/backfilling on the lateral soil resistance against the large pipeline displacements. These effects depend on a wide range of parameters including but not limited to the trench configuration, seabed soil properties, construction methodologies, nature of applied loads, site-specific condition, etc. The study suggests that using a general mathematical response for lateral soil resistance against a pipeline buried in the uniform soil can be a gross simplification and result in remarkable level of inaccuarcies in some occations.

5.7 Acknowledgments

The authors gratefully acknowledge the financial support of “Wood Group,” that established a Research Chair program in Arctic and Harsh Environment Engineering at Memorial University of Newfoundland, the “Natural Science and Engineering Research Council of Canada (NSERC)”, and the “Newfoundland Research and Development Corporation (RDC) (now InnovateNL)” through “Collaborative Research and Developments Grants (CRD)”. Special thanks are extended to Memorial University for providing excellent resources to conduct this research and also the technicians at C-CORE’s centrifuge lab for their kind technical support.

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CHAPTER 6. The effect of backfilling shear strength on the lateral p-y response of the deeply trenched-backfilled pipelines: an experimental study

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Abstract

Subsea pipelines passing through the shallow area are physically protected against the environmental, accidental, and operational loads by trenching and backfilling. The backfill can be selected from purchased or pre-excavated material depending on the project-specific requirement; although the latter one is usually a cost-effective option in most of the projects. Depending on construction methodology, environmental loads, and seabed soil properties, the stiffness of backfilling material may become largely different from the native ground (softer than native ground in most of the cases). The different stiffness between the backfill and native ground affects the soil failure mechanisms and lateral soil resistance against large pipeline displacements that may happen due to ground movement, landslides, ice gouging, and drag embedment anchors. This important aspect is not considered by current design codes. In this paper, the effect of trench-backfill stiffness difference on lateral pipeline-backfill-trench interaction was investigated by performing centrifuge tests. The soil deformations and failure mechanisms were obtained by Particle Image Velocimetry (PIV) analysis. Three experiments were conducted by using three different backfills including loose sand, slurry, and chunky clay that represent the purchased, natural in-fill, and pre-excavated materials, respectively. It was observed that the backfill stiffness has a significant impact on the failure mechanism. Also, the study showed that the current design codes underestimate the lateral soil resistance for small to moderate pipe displacements inside the trench, and overestimate it for large lateral displacement, where the pipeline is penetrating into the trench wall.

Keywords: Lateral pipe-soil interaction; pipeline-trenchbed interaction, p-y response; large deformation; centrifuge testing.

6.1 Introduction

Submarine pipelines are usually trenched and buried in shallow waters to mitigate the risk of damage due to environmental, operational, and accidental loads. Buried pipelines may be subjected to large lateral displacements due to ground movements, landslides, ice gouging, and drag anchors. Pre-dredged material is commonly used as a cost-effective backfilling option. Purchased granular material may be also used on some occasions to achieve the pipeline target stability, or suppress the wave/current-induced liquefaction or scour. The fine backfilling material is usually remoulded to a high extent depending on the trenching techniques, construction procedure, and environmental loads. The granular backfilling materials may be also densified under the action of environmental loads. These processes result in a backfilling material with a stiffness that is significantly different from the seabed native ground (see Figure 6.1). The previous studies have shown that the high difference between the stiffness of the backfill and native ground (i.e., trench wall and bed) could have a significant influence on lateral soil resistance against the largely displaced pipe (Paulin, 1998; Kianian et al., 2018). However, the trenching/backfilling effect is currently not considered by pipeline design codes (e.g., DNVGL-RP-F114, 2017; PRCI, 2009; ALA, 2005); and ASCE committee, 2014) due to less-explored pipeline-backfill-trench interaction and its impact on the soil failure mechanism. Therefore, the existing solutions for prediction of lateral soil resistance are usually based on lateral pipe-soil interaction in uniform soil (Tschebotarioff, 1973; Wantland et al., 1979; Paulin, 1998) or anchor soil interactions in uniform soil (Hansen, 1948; Hansen and Christensen, 1961; Das and Seeley, 1975; Rowe and Davis, 1982; Merifield et al. 2001).

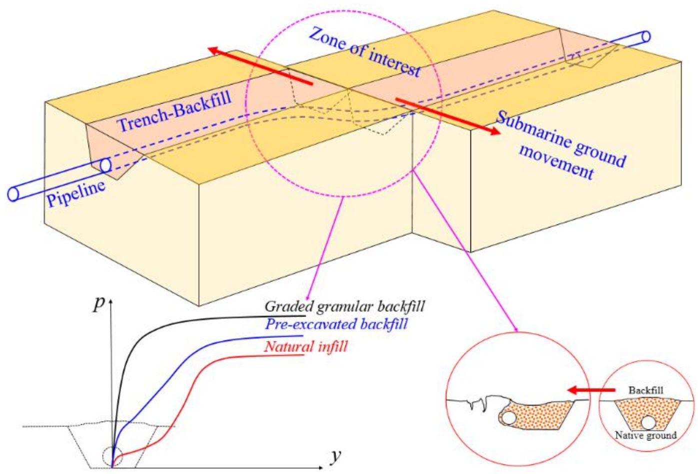


Figure 6.1 The lateral response of trenched pipeline to submarine ground movement.

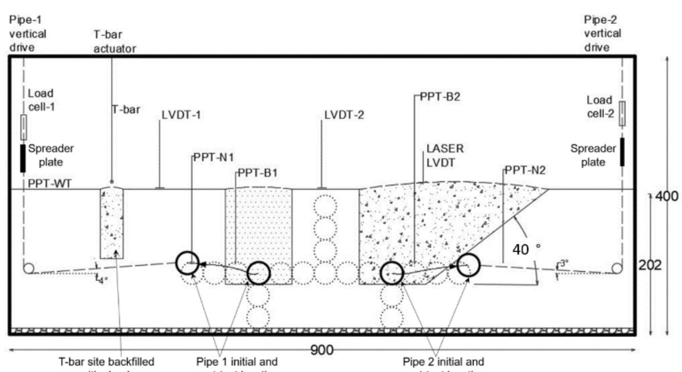
The previous studies have shown that the high difference between the stiffness of the backfill and native ground (i.e., trench wall and bed) could have a significant influence on lateral soil resistance against the largely displaced pipe (Paulin, 1998; Kianian et al., 2018). However, the trenching/backfilling effect is currently not considered by pipeline design codes (e.g., DNVGL-RP-F114, 2017; PRCI, 2009; ALA, 2005); and ASCE committee, 2014) due to less-explored pipeline-backfill-trench interaction and its impact on the soil failure mechanism. Therefore, the existing solutions for prediction of lateral soil resistance are usually based on lateral pipe-soil interaction in uniform soil (Tschebotarioff, 1973; Wantland et al., 1979; Paulin, 1998) or anchor soil interactions in uniform soil (Hansen, 1948; Hansen and Christensen, 1961; Das and Seeley, 1975; Rowe and Davis, 1982; Merifield et al. 2001).

In this study, centrifuge tests were conducted on trenched pipelines deeply buried in three different kinds of backfills including loose sand, slurry, and chunky clay to qualitatively investigate the internal soil deformations and failure mechanisms affected by different backfilling strength. These three backfilling material represented the purchased (granular), natural infill (slurry), and pre-excavated material (remoulded soft clay) that are the most probable scenarios. Particle Image Velocimetry (PIV) analysis was conducted by using a transparent observation window and digital cameras. Paulin (1998) observed that pipeline-backfill-trench interaction is not fully developed in undrained conditions. Therefore, the partially drained condition was adopted in this study by applying a low pipe moving velocity to magnify the pipeline-backfill-trench interaction effects. This enabled a clear capturing of the shear bands and failure mechanism affected by trench configuration and seabed soil properties.

The study showed that the current design codes underestimate the lateral soil resistance for small to moderate pipe displacements inside the trench, and overestimate it for large lateral displacement, where the pipeline is penetrating into the trench wall. The design practice can take advantage of pipeline-backfill-trench interaction effects to improve the safety and cost-effectiveness of the design of the buried pipeline against the large lateral displacements. Further numerical and experimental studies are required for proposing quantitative design equations incorporating the pipeline-backfill-trench interaction effects on lateral soil resistance.

6.2 Test set up and modeling remarks

The C-CORE centrifuge facilities located at St. John's campus of the Memorial University of Newfoundland were used to conduct a series of tests investigating different aspects of lateral pipeline-backfill-trench interaction. Since the main objective of the study was a qualitative assessment of the effect of different backfilling stiffness on the failure mechanisms, there was no need for a high number of tests that are usually required for quantitative studies. Therefore, three tests (T1P2, T2P1, and T5P2) with a different type of backfilling material were selected to study the most potential backfilling scenarios. The PIV analysis was conducted to obtain the internal soil deformation and failure mechanisms by monitoring through a transparent observation sidewall. Similar test setup to Paulin (1998) was adopted to facilitate comparing the results with earlier studies. The strongbox accommodated two pipe sections in each run that were horizontally pulled in opposite directions. Figure 6.2 shows the test setup and post-test condition.



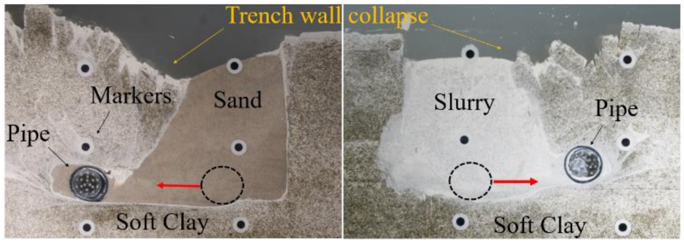


Figure 6.2. Test setup and post-test conditions (dimensions in mm).

A 24” pipe with an external diameter of 610 mm was selected as a technical requirement by the project’s industry sponsor in continuation of their earlier full-scale studies in the sand (Burnett, 2015). Therefore, a stainless still pipe section was used for model pipe with a diameter of 31.75 mm to enable accommodating the internal pressure transducer without folding the data transfer cable. Two sets of strain gauges, one internal pore pressure transducer (facing the rear of pipe), two pulling cables, and two rubber end caps (both lubricated, on patterned in window side) were used as model pipe instrumentation. These sizing requirements resulted in a spinning acceleration of 19.1g. It is worth noting that centrifugal acceleration simulates gravity and allows for correspondence of stress fields between model and full-scale resulting in enhancement of modelling accuracy. Paulin (1998) conducted modelling of model tests and attempted to model the same prototype condition at 1:25, l:50 and 1:100 scales for the lateral response of trenched/backfilled pipelines in clay to verify the scale effects. The study showed acceptable results with most of the interaction curves within a bandwidth of 0.5-1 normalized lateral loads. Considering these earlier results, the scaling studies were not repeated in the current study, having a test set up almost identical to Paulin (1998).

A 50%-50% (equal weight) mixture of Speswhite kaolin clay and Sil-Co-Sil silt were used with sufficient amount of water to form a slurry with a nominal moisture content of about 70%. The prepared slurry was incrementally consolidated up to the effective stress of 400 kPa and then was unloaded to 100 kPa with an open drainage valve. The base drain was closed to stop the water flow into the sample during the unloading process. The consolidation resulted in a soft clay bed with an undrained shear strength of 15 to 25 kPa. A blade mounted on a supporting beam was used to excavate rectangular trenches about 3.6D wide and 4.0D deep (T1P2 and T2P1). Chaloulos et al. (2015) showed that the trench wall angle has no influence on lateral soil resistance of deeply buried pipelines. Having limitations in access to the centrifuge facilities and the total number of tests, this advantage reported by Chaloulos et al. (2015) was used and a trench wall angle of fourty degrees was configured in the test T5P2. This enabled using the results of this test in multiple comparative studies in the main testing program without affecting the objectives of the current study. An average burial depth ratio of H/D = 3.5 were considered to model the deep burial condition (H/D was defined as the initial ratio of the pipe springline depth to the pipe diameter). The boundary effects were suppressed by assuming a 2D clearance between the trench bottom and the lower drainage layer at the bottom of the test box.

The testing operation was monitored by using three parallel data acquisition systems and a range of instruments including pore pressure transducers (PPTs), strain gauges, load cells, conventional and laser linear variable differential transformers (LVDTs), T-bar, vertical drive motion controller, digital cameras, markers, and artificial textures. The state-of-the-art instrumentation enabled capturing the development of shear bands in backfill and native soil, progressive failure mechanisms, lateral soil resistance of the pipeline, the suction force in the rear of pipe, and the pore pressure variation in the surrounding soil. The lubricant was applied to the internal walls of the strongbox to suppress the friction between the box walls and the pipe end caps. The suppression of the friction was verified by performing a trial pipe pulling with no soil bed and measuring the pulling load. In addition, to by-pass, the friction between the cable and the soil, and the cable and pulleys, two strain gauges were installed on each pipe to purely measure the lateral soil resistance right in front of the pipe. Load cells were also used on pulling cables to crosscheck the measurements of the strain gauges. The results presented in this paper are those obtained from the strain gauges.

The pipes located at the center of the trench were laterally pulled in opposite directions and under displacement-controlled condition, while there was no restriction against vertical displacement. The bottom elevation of the pulleys was aligned with the pipe springline to ensure a horizontal pull. Releasing the vertical restraints allowed the pipes to slightly move upward at the beginning of the test following the least resistance path. This enabled the effect of the pipeline/trenchbed interaction on lateral soil resistance to develop. With larger pipe displacements, the pipes tended to move vertically downward as the result of the pipe weight, bearing stress, and pulling cable. The measured lateral forces are better to be calibrated against the instantaneous pipe trajectory for more accurate lateral soil resistance. However, this would only improve the current results by about $\pm$ 0.4%. Figure 6.3 shows the flowchart of the test set up preparation and testing procedure.

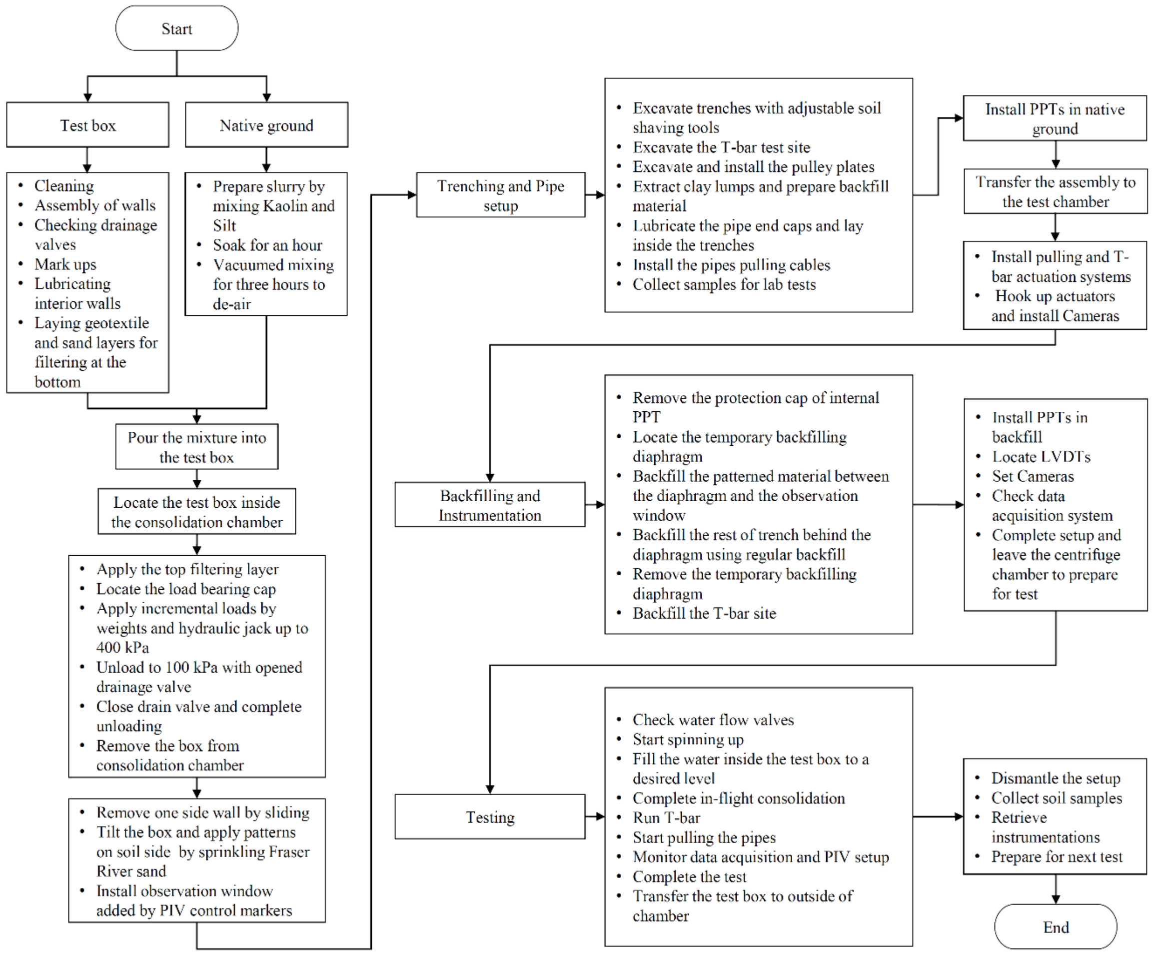


Figure 6.3. Flowchart of test preparation and testing procedure

Partially drained condition was adopted (average normalized velocity, $vD/c_v = 0.3$, based on Phillips et al., 2004) to consolidate the surrounding soil, eliminate the effect of excess pore pressure and magnify the effect of pipeline-backfill-trench interaction. Besides these advantages of partially drained condition, silt fractions are found in natural offshore soft clays in many geographical locations (e.g., the Gulf of Mexico, Schiffman, 1982) that tend the consolidation characteristics of clay towards partially drained and even fully drained conditions. Other compositional and depositional fractions may also show a similar effect.

The strength profile of the backfill and the native ground was measured in-flight by using a T-bar penetrometer with a head bearing area of 30 × 7.4 mm^2 (Stewart and Randolph, 1994). The T-bar test site was configured to allow for assessing the backfill and native ground strengths in a single run (see Figure 6.2).This was due to limited space on the test box and having access to only one T-bar actuator. The selected dimensions of the T-bar site allowed the suppression of the boundary effects in different directions. The T-bar was first penetrating to the backfill material and then continued to penetrate into the native ground. A base-case T-bar test was also conducted on the pure native ground to capture the net shear strength profile of the native soil. Figure 6.4 shows the schematic configuration of the T-bar site, and the measured undrained shear strength profiles with linear curve fit in T1P2 and T5P2 tests.

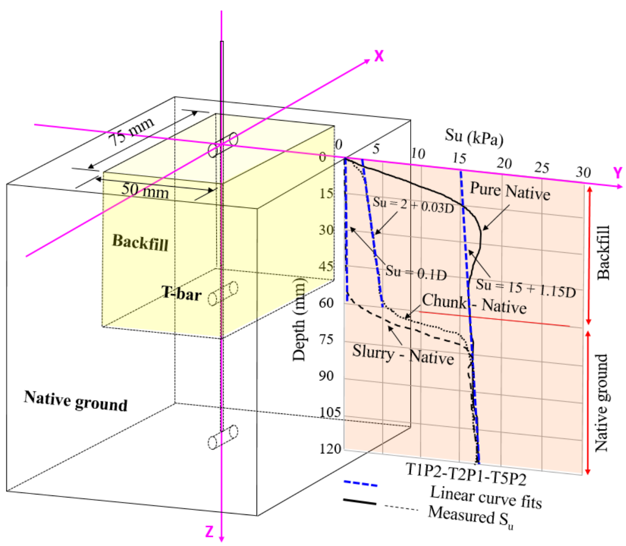


Figure 6.4. **Schematic configuration of the T-bar site and the measured undrained shear strength profiles with linear curve fits.**

Figure 6.4 shows almost a negligible undrained shear strength for slurry. Due to the slight water penetration from the backfill to the native soil, a slightly softer response was observed in the initial stages of penetration right underneath the backfill material. This is. By increasing the penetration, the plots of overlaid native soil strengths are gradually matching the profile of pure native soil. Table 6.1 shows a summary of the backfilling and native material prepared and tested in this study.

Table 6.1. Soil properties.

Test ID	Trench backfill type	Native c_u at pipe invert (kPa)	Native water content before and after the test (%)	Υ_{sat} (kN/m3)	Native soil mudline strength, S_{um} (kPa)	Native soil Strength gradient, k_{su} (kPa/m)	Backfill mudline strength, S_{um} (kPa)	Backfill strength gradient, k_{su} (kPa/m)
T1P2	Slurry	17.6	32.04 - 32.97	18.33	15	1.15	0	0.10
T5P2	Chunk	18	30.12 - 32.13	18.43	15	1.15	2	1.60
T2P1	Sand	17.9	31.86 – 32.56	18.56	15	1.15	N/A	N/A

In order to convert the measured unit bearing resistance, q, to the local undrained strength, S_u, a T-bar factor of N_{kt} = 10.5 was used as recommended by Stewart and Randolph (1994) (based on the plasticity solution, Randolph and Houlsby, 1984). A reduced bearing factor was used for shallower depth by considering the effect of soil buoyancy and a shallow failure mechanism mobilized prior to the full flow of soil around the bar (White et al., 2010). Linear curve fits were used to approximate the S_u profiles both in the backfill and native soils to facilitate the back-analysis of the test data. The sand backfill properties were adapted from Paulin (1998) due to using an identical sand material and test configuration.

6.3 Results of testing program

The effect of different backfills on internal soil deformations and the failure mechanisms were investigated by performing PIV analysis using GeoPIV-RG software (White et al., 2003; Stanier et al., 2016). The soil displacements and strain levels were obtained and its relationship with lateral soil resistance or force-displacement curves (p-y curve) were studied through defining three different zones of pipe displacements (i.e., zone I, II, and III). Several interactive soil deformation mechanisms were observed in the various stages of pipeline lateral displacements that are significantly affected by the stiffness of the backfilling material. The identified displacement zones (zone I, II, and III) facilitated a detailed investigation of the mobilization of lateral soil resistance against the moving pipe (see Figure 6.5).

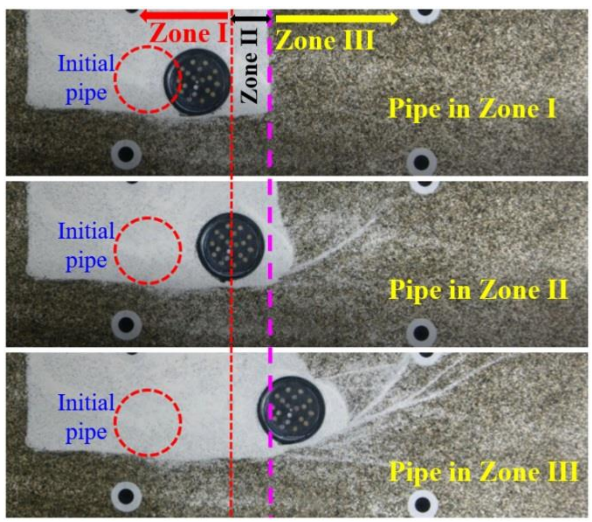


Figure 6.5. **Identified zones of pipeline displacements with different failure mechanisms.**

Zones I ,II , and III refer to i) the pipeline moving inside the backfill with almost no interaction with the trench wall, ii) the pipeline approaching and interacting with the trench wall, and iii) the pipeline fully penetrated into trench wall, respectively. It will be shown in the coming sections that in case of sand backfill, the pipeline interacts with the trench wall as soon as the pulling starts. Therefore, the zone I is not applicable to test T2P1 (see Figure 6.5). The p-y response for these three tests is shown in Figure 6.6. The responses in the zone I, II, and III have been highlighted by orange, green, and purple highlights for further references in the coming discussions. In these type of plots, the load is usually normalized by pipe diameter and undrained shear strength to obtain the interaction factor. However, in the current study, normalizing the load could result in discontinuities in p-y response due to different magnitude of undrained shear strength in backfill and native. This could be more complicated in the transition from backfill to native soil. Therefore, the load response was not normalized in Figure 6.6 for a proper presentation of p-y response.

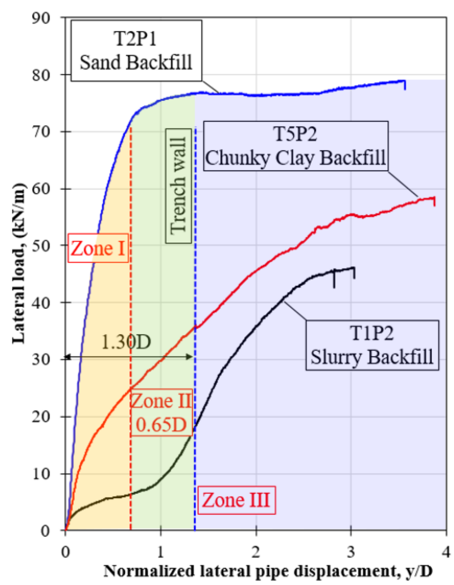


Figure 6.6. The p-y responses of tests with slurry, chunk, and sand backfills

It was observed that the ultimate lateral soil resistance is significantly reduced by increasing the stiffness of backfilling material. The slurry backfill qualitatively representing the natural infill of the seabed trench resulted in the lowest ultimate soil resistance. The sand backfill produced the highest soil resistance, even being in a loose condition. The response obtained from the chunky clay backfill that represents the pre-dredged material ended in between the sand and slurry backfilling cases. In addition, the soil resistance followed different but meaningful paths to achieve the ultimate resistance. A proper understanding of the differences between the p-y curves needs a comparative investigation of the soil deformation mechanisms and the pipe-backfill-trench interaction in different zones of the pipe displacements that will be discussed in the coming sections.

Zone I, pipeline moving inside the trench

As soon as the centrifuge starts spinning, the pipeline slightly penetrates into the trench bed (by about 0.1D) due to its weight, before starting the pulling operation. This causes the pipe to interact with the backfill and trenchbed, both of which could contribute to the mobilized lateral soil resistance. Figure 6.7 shows samples of the PIV analysis in the conducted tests, while the pipe is moving in zone 1. The results have been presented based on an identical pulling direction to facilitate comparison.

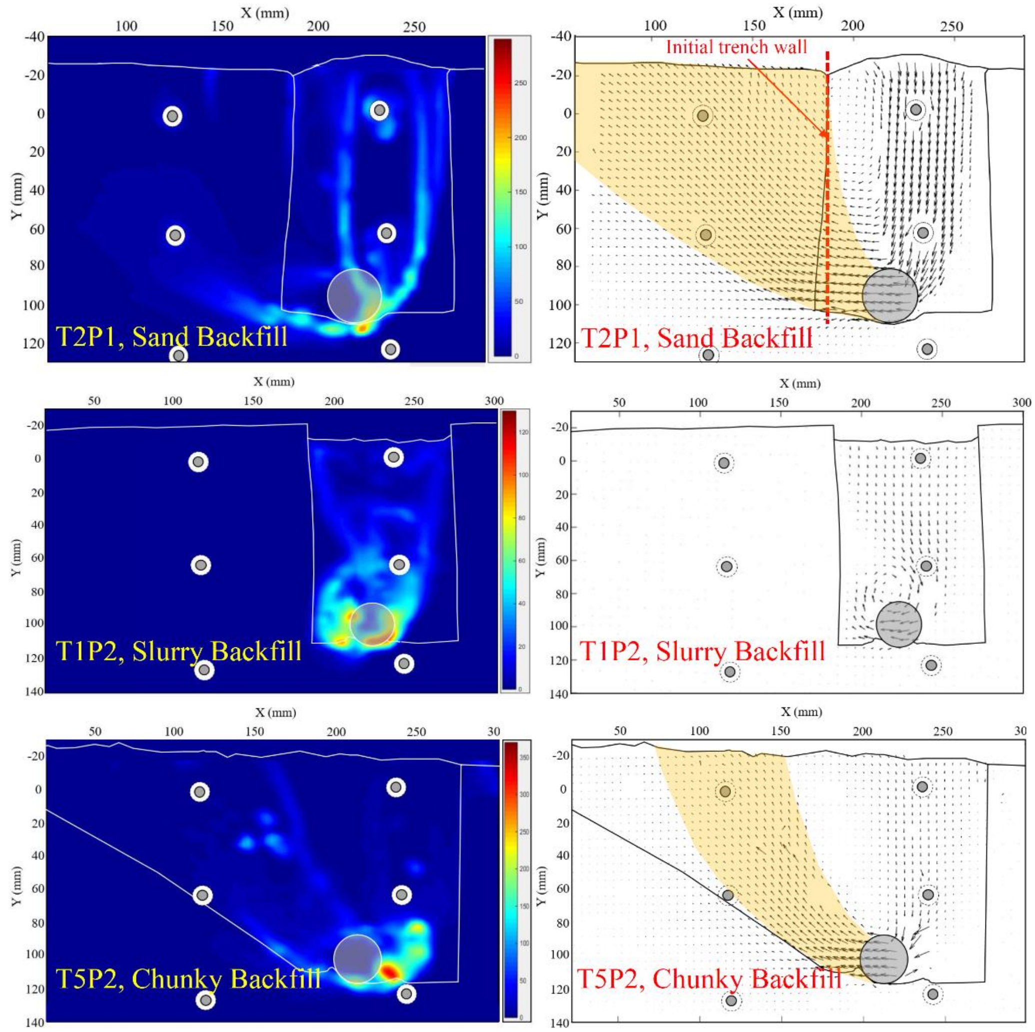


Figure 6.7. Sample of PIV analysis results in zone I.

Figure 6.7 shows several interesting mechanisms that describe the different p-y responses observed in Figure 6.6. In tests P1T2 and P5T2 (slurry and chunky backfilled), the pipe and backfill displacements showed almost no influence on the trench wall. Inversely, the trench wall in test T2P1 (sand backfilled) started to deform immediately after starting the pipe pulling. Therefore, as mentioned earlier, there is no zone I in this particular test. As shown in Figure 6.7 by highlighted orange areas, the soil volume mobilized against the pipe movement in T2P1 (sand backfilled) is contributed by both native ground and sand backfill. Therefore the p-y curve of T2P1 in Figure 6.6 is much larger than T1P2 (slurry backfilled) and T5P2 (chunky backfilled), where the native ground showed no contribution and the backfilling shear strength was much lower. Considering a very low magnitude of the shear strength in the slurry backfill (T1P2), the lateral soil resistance was expected to be very low in zone I. However, the p-y response of T1P2 in Figure 6.6 still shows a remarkable amount of lateral soil resistance. The PIV analysis revealed that this mobilized force in zone I is the result of the interaction between the pipeline and the trenchbed. The initial embedment of the pipe into the trenchbed during the inflight consolidation creates small soil berms on both sides of the pipe. These small soil berms, their enlargement during the lateral pipe movement, squeezing the soil berm to the trench corner, and overall the pipeline-trenchbed interaction and its intensity effect on lateral soil resistance have been entirely investigated and discussed by Kianian and Shiri (2019).

In addition, a second mechanism, i.e., the pipeline-backfill interaction was also observed in this zone. Although the contribution of this mechanism to p-y response in T1P2 is negligible due to the very low shear strength of the slurry backfill. However, it is still worth reviewing the features of this interesting mechanism. As schematically shown in Figure 6.8, the mechanism comprises loops of eccentric spiral soil flows with rotational circles around the moving pipe. These spiral flow surfaces emanate from a point above the pipe and move horizontally with the pipe until the failure surface touches the trench wall. From this stage, with a further displacement of the pipe towards the trench wall, the spiral flow starts to contract with a varying ratio that depends on its distance to the wall. The closer to the wall, the smaller the failure circle will be. An additional upside-down wedge failure was observed on top-right of the spiral flow with side legs remaining asymptotic to the spiral surface at the rear of the moving pipe. The far failure surface (in the rear of pipe) is still a kind of logarithmic spiral, but there is an inflection point in the near failure surface (above the pipe) of the deep backfill.

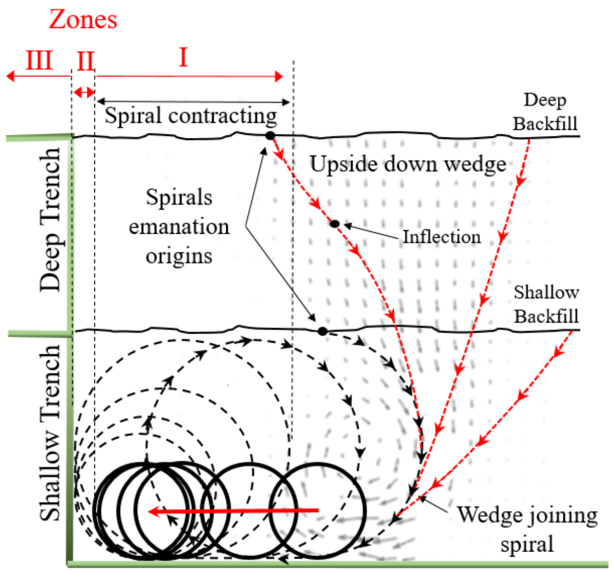


Figure 6.8. Contracting spiral flow mechanism of slurry backfill.

Looking at progressively contracting spiral flows, the contribution of pipeline-backfill interaction to the lateral load is dissipated due to the reduction of the magnitude of mobilized rotational shear stress in the backfill. However, the obtained p-y responses show a continuous increase in lateral load due to the aforementioned pipeline-trenchbed interaction process (see Figure 6.6).

Zone II, pipeline interacting with the trench wall

Zone II was started as soon as the pipeline interacts with the trench wall and the pipe displacement inside the trench causes the trench wall to be displaced as well. In test T2P1 (sand backfill), zone II started immediately as the pipeline was laterally pulled. This zone ends by arriving at the pipe front at the initial position of the trench wall (see Figure 6.5). The total length of zone II in this study was about 0.65D for T1P2 (slurry backfilled), 0.80D for T5P2 (chunky backfilled), and 1.30D for T2P1 (sand backfilled). This shows that the zone II becomes larger as the backfilling strength is increased. During the pipeline displacement in zone II, the soil deformations in the trenchbed, backfill, and trench wall were interactively developed. Moving inside the zone II, the backfilling material in T2P1 and the trench bed making a soil berm in front of the pipe in T1P2 were compressed between the pipe and trench wall and caused the pipeline to virtually act as a larger pipe. These virtual pipes were pushed into the trench wall and resulted in a local failure in T1P2 and formation of the first failure shear band in the trench wall of T2P1. The latter one was formed before arriving at the pipe to the end of zone II or the initial trench wall location. Figure 6.9 shows a sample of the failure mechanism and the PIV analysis results for T1P2 (slurry backfilled) and T2P1 (sand backfilled) tests in zone II. The larger virtual pipe in T2P1 with a larger imaginary diameter has mobilized a larger amount of native ground and resulted in a higher lateral soil resistance in comparison with T1P2.

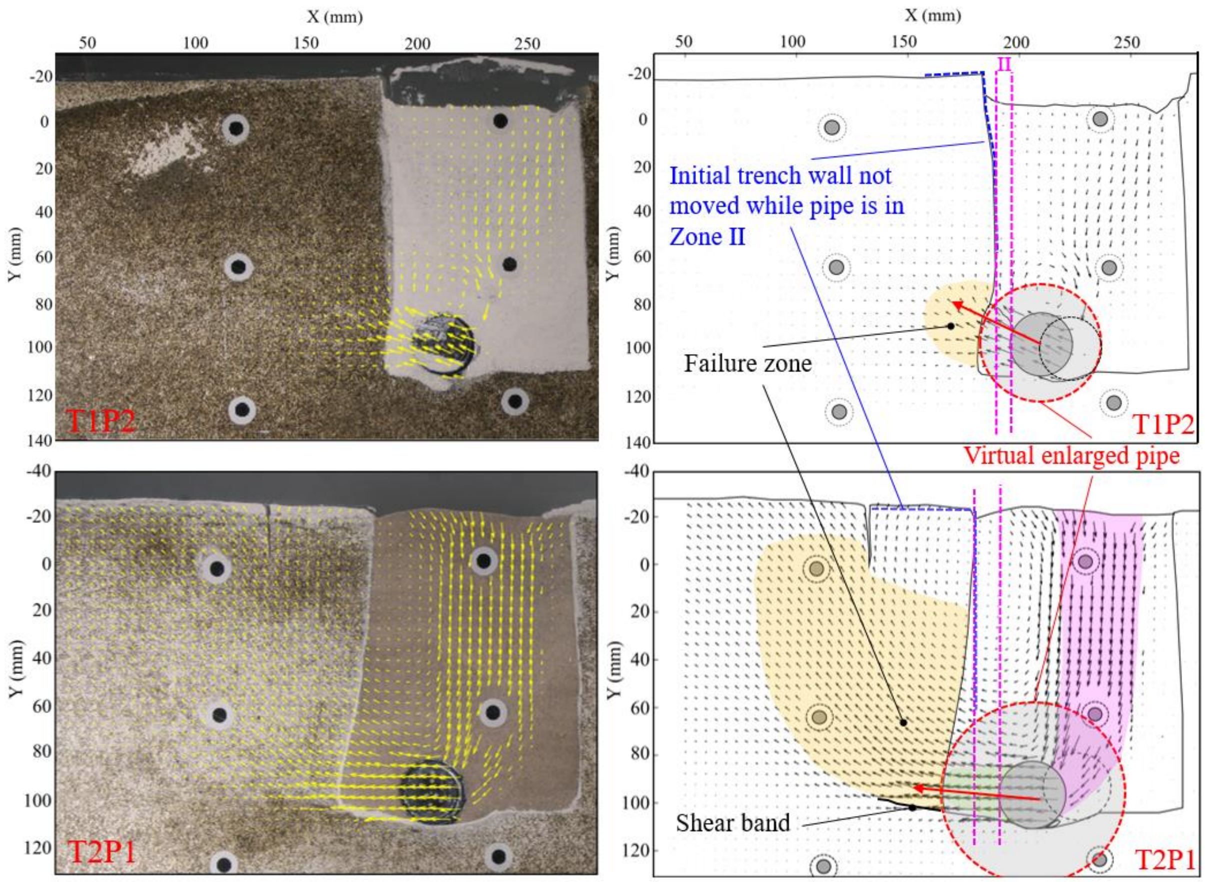


Figure 6.9. **Sample of PIV results for T2P1 and T1P2 in zone II.**

Also, a closer look at the pipes trajectories shows that having no vertical restriction, the pipe in T1P2 has naturally followed the route with least work, i.e., upward towards the backfill with softer material. Despite the T1P2 (slurry backfilled), the higher backfilling weight in T2P1 (sand backfilled) has not allowed the pipe to freely move upward. This resulted in the mobilization of a larger soil volume and larger lateral soil resistance, consequently. In the test T2P1, another mechanism has also contributed to increasing of the p-y response. This mechanism is related to the stiffness of the backfilling material that will be further discussed in zone III.

Zone III, pipeline penetrating into the trench wall

Zone III starts by arriving at the pipe front at the initial trench wall location. Figure 6.10 compares samples of PIV analysis and soil deformation mechanisms in the case of backfills with the highest and lowest shear strengths, i.e., the T2P1 and T1P2, consequently. At the start of zone III, a small triangular wedge was created in front of the pipe, while the shear band in the trench wall was developed. It was observed that the orientation of this shear band with the horizontal line depends on the orientation of the pipeline displacement and almost follows the same orientation as the pipeline. As the pipe moved forward, the cable angle offset from the horizontal line was increased, and this affected the pipe trajectory by pulling it down. By changing the pipe trajectory, a new global shear band formed underneath the pipe with a new orientation. The first global instability of the trench wall appeared on top of the pipe, as soon as the top-front edge of the real pipe touched the native trench wall. At this stage, the trench wall tends to slip down into the backfill. However, the backfilling material produces a passive pressure against the collapsing wall. This passive pressure is directly related to the shear strength of the backfilling material. The sand backfill has a shear strength higher than chunky and slurry backfills. In T2P1, the high shear strength of sand backfill did not allow full development of the shear bands in the trench wall towards the soil surface. Therefore, the mobilized energy in the native ground (trench wall) couldn't be released throughout the shear bands. This, in turn, resulted in higher lateral soil resistance in T2P1. In T1P2, the soft slurry backfill, could not produce sufficient passive pressure against the failing trench wall. Therefore, the shear bands in the trench wall were fully developed and arrived at the soil surface. This caused the trench wall to slip into the backfill and significantly release the lateral soil resistance. In T5P2 with chunky backfilling material, a moderate condition happened and the shear bands development was larger than T2P1 (sand backfilled) and smaller than T1P2 (slurry backfilled). Figure 6.11 shows the strain counters, soil deformations, shear bands, and failure mechanisms for T1P2 (slurry backfilled), T2P1 (sand backfilled), and T5P2 (chunky backfilled) at various stages of the pipeline displacements.

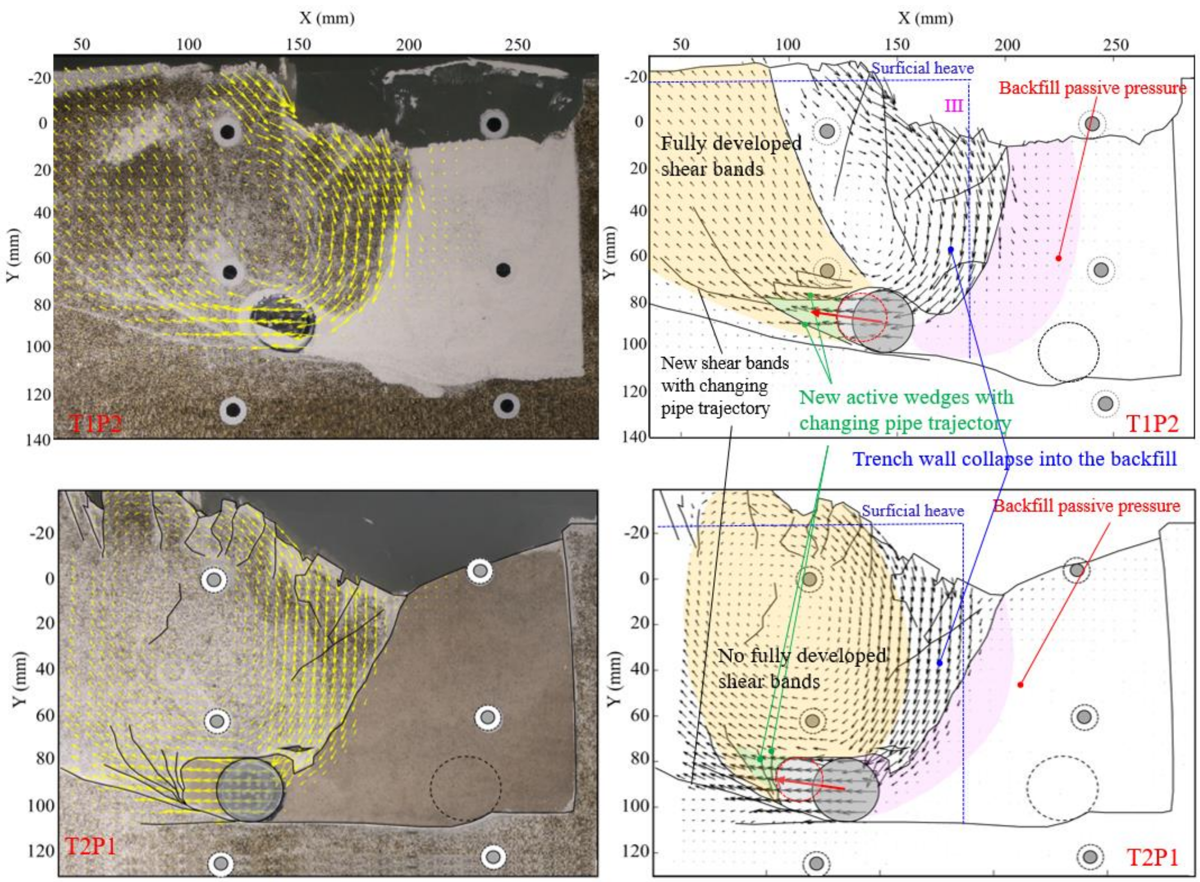


Figure 6.10. Sample of PIV results and soil displacement in zone III of T1P2 and T2P1.

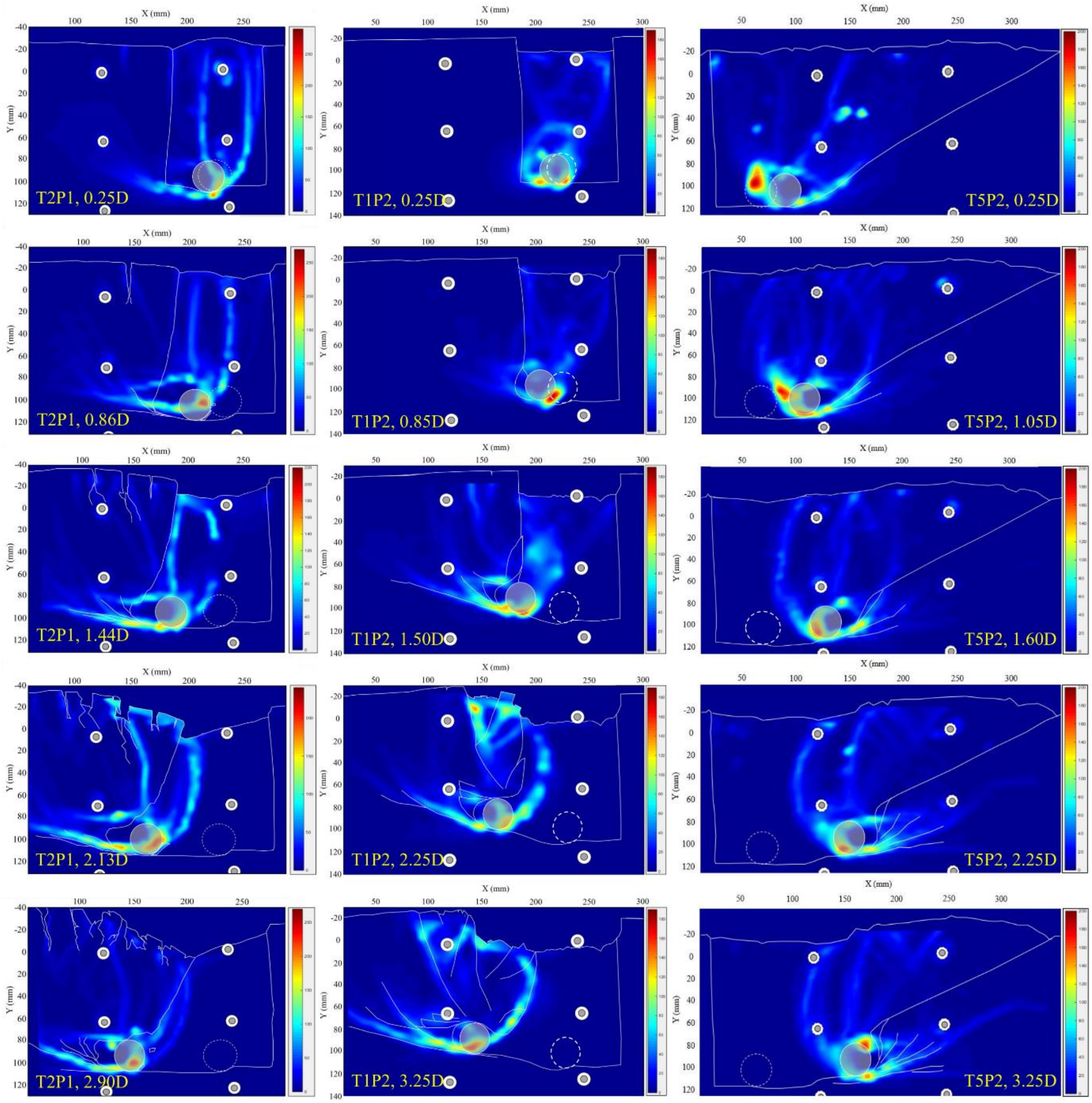


Figure 6.11. **Strain contours obtained from PIV analysis for T1P2 and T2P1.**

Overall, these mechanisms resulted in the highest lateral soil resistance in T2P1 (sand backfilled), the lowest resistance in the T1P2 (slurry backfilled), and the moderate resistance in T5P2 (chunky backfilled).

Pore pressure measurement

Pore pressure transducers (PPTs) were placed in various locations (see Figure 6.2) inside the pipe, backfill, and native ground. PPTs could capture the pore pressure variation and also the suction force behind the pipe. A surface PPT was also used to monitor the water table in the testbed. Figure 6.12 shows the variation of pore pressure against the pipe displacement in the rear of the pipe (Internal PPT), native ground, and in backfills.

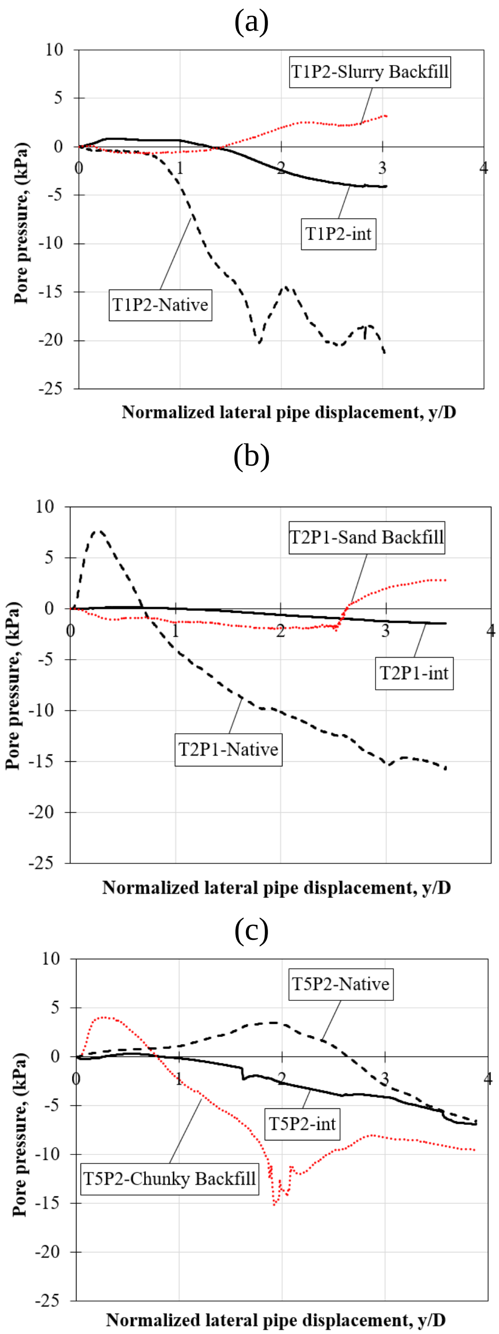
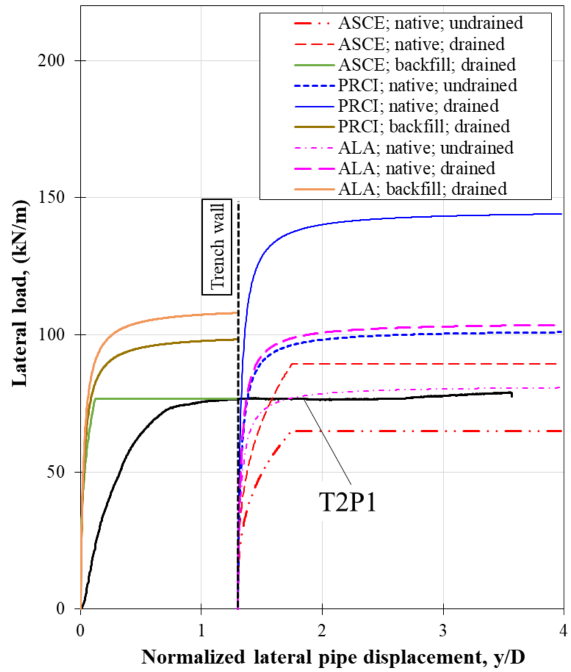


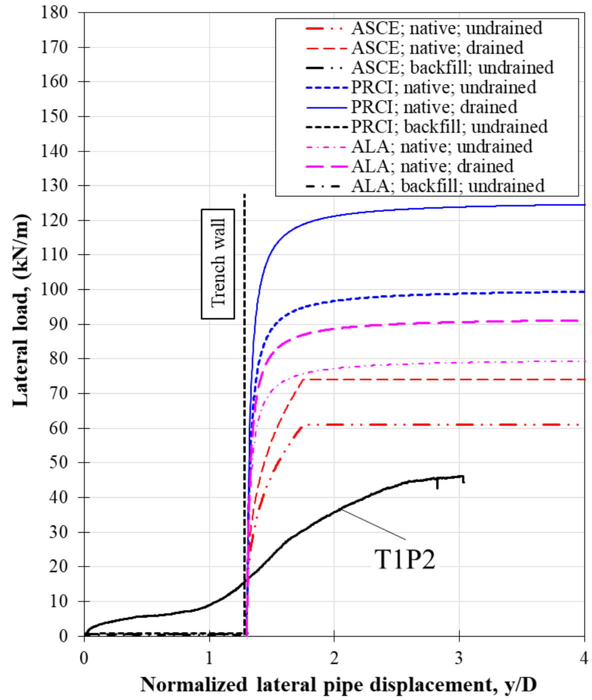
Figure 6.12. **Pore pressure variation in (a) backfill (BF), (b) native ground (NG), and (c) rear of the pipe.**

Overall, similar trends were observed in the PPTs used in T1P2 (slurry backfilled), T2P1 (sand backfilled), and T5P2 (Chunky backfilled). An initial increase of the pore pressure was observed in the internal PPTs (dashed lines in Figure 6.12) in the zone I followed by the dissipation of the excess pore pressure and developing a slight suction force behind the pipe in zones II and III. The magnitude of this suction force is not significant due to the low displacement rate of the pipe in a partially drained test condition. A similar trend with larger excess pore pressure and suction forces was observed in PPTs located in the native ground (solid lines in Figure 6.12). The PPTs inside the backfill (dotted lines in Figure 6.12) showed a slight suction at the beginning followed by an excess pore pressure in large displacements (except T5P2). The irregular fluctuation of the pore pressure variation in T5P2 is the result of the ununiformed distribution of the chunky backfill. The larger magnitude of the suction force in the native ground on T2P1 and T5P2 are in agreement with the soil failure mechanisms and progressive development of incomplete shear bands in the native ground due to delayed interaction of the pipe section and the trench wall, and consequently, the p-y responses showed in Figure 6.10.

6.4 Observed p-ys vs. existing solutions

The p-y responses in T1P2, T2P1, and T5P2 were compared with the solutions recommended by some of the most popular pipeline design codes (i.e., PRCI, 2009; ALA, 2005; and ASCE committee, 2014). There is still no systematic solution or recommendation by design codes to consider the effect of trenching and backfilling on lateral soil resistance. Therefore, the comparison was independently made only for backfill and native ground assuming uniform soil in the zone I and III. Figure 6.13 shows the p-y comparison both in drained and undrained conditions depending on the applicability.


ASCE; native; undrained
ASCE; native; drained
ASCE; backfill; drained
PRCI; native; undrained
PRCI; native; drained
PRCI; backfill; drained
ALA; native; undrained
ALA; native; drained
ALA; backfill; drained
Trench wall
T2P1
Lateral load, (kN/m)
Normalized lateral pipe displacement, y/D


ASCE; native; undrained
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ASCE; backfill; undrained
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PRCI; native; drained
PRCI; backfill; undrained
ALA; native; undrained
ALA; native; drained
ALA; backfill; undrained
Trench wall
T1P2
Lateral load, (kN/m)
Normalized lateral pipe displacement, y/D

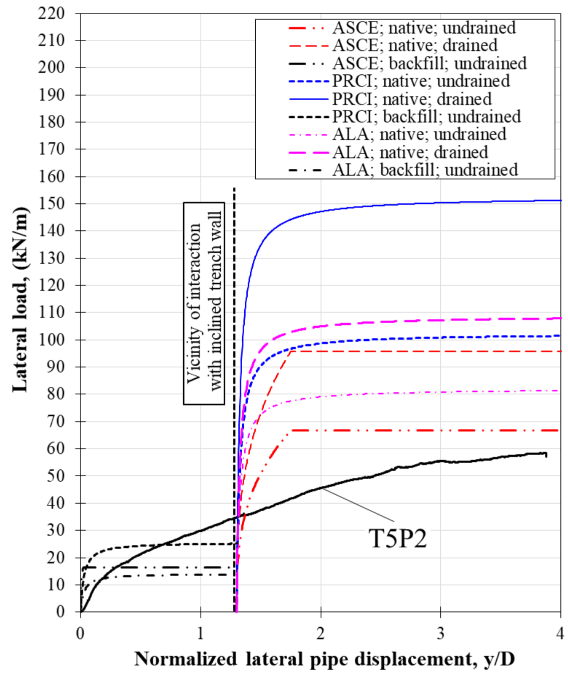

Figure 6.13. The comparison of the p-y responses between the test results and design codes.

Figure 6.4 was used to extract the soil strength parameters for the undrained condition. The soil parameters for drained parameters were adopted from the tri-axial tests (Paulin, 1998). It is worth mentioning that the PRCI (2009) and ALA (2005) guidelines have solutions both for drained and undrained conditions by superposing the contribution of effective friction angle and effective cohesion for drained conditions and considering the undrained shear strength for the undrained condition. The ASCE Committee (2014) provides separate solutions for sand and clay using the friction angle and undrained shear strength, respectively. To use the recommended equations for clay under drained conditions, the effective cohesion was disregarded. For sand, the effective friction angle was used in the proposed equations. Unlike ASCE committee (2014), the formulations recommended by

PRCI (2009) and ALA (2005) do not cover the low range of friction angles preventing them from being used for very soft slurry backfill used in this study.

The comparison of experimental p-y results with the existing solutions in Figure 6.13 shows that the lateral soil resistance can be significantly underestimated in zone I and overestimated in zone III, if the trenching/backfilling effects are ignored. Figure 6.14 schematically shows the comparison of the observed soil resistance trends.

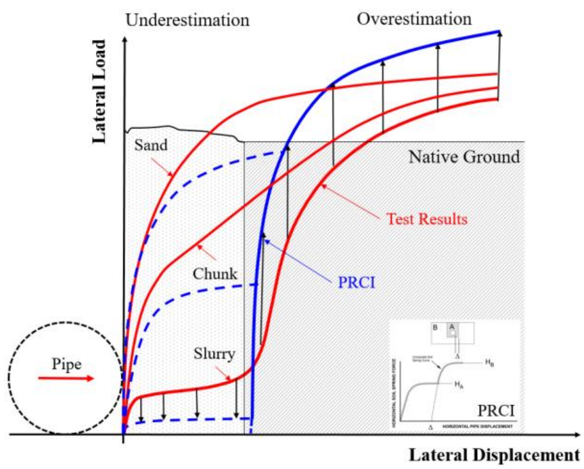

Figure 6.14. Schematic illustration of the difference between the p-y curves predicted by design code and experimental observations.

The PIV results suggest that the underestimation of the soil resistance inside the backfill (zone I) is due to neglecting the pipeline-trenchbed interaction. This effect is less significant in the case of sand backfill (T2P1). The overestimation of ultimate soil resistance in the native ground (zone III) is because of the trench effect and the lower magnitude of the passive pressure against the collapsing trench wall that is provided by a softer backfill. Both of these effects are currently neglected by design codes (e.g., DNVGL-RP-F114, 2017;

PRCI, 2009; ALA, 2005); and ASCE committee, 2014) due to less explored soil deformation mechanisms. Also, Figure 6.13 shows that the design codes underestimate the lateral soil resistance in zone II or transition from backfill to native ground. This implies more risk on pipeline design since the majority of pipeline displacements occur in these ranges. A proper estimation of the soil resistance in these domains could result in a more cost-effective design.

6.5 Conclusions

The effect of backfilling shear strength on the lateral soil resistance against the moving pipeline was investigated by performing three small-scale centrifuge tests. Sand, slurry. And chunky clay backfills were investigated. Internal soil deformations and failure mechanisms in the soil surrounding the pipe were investigated by using a transparent observation window and PIV analysis. The partially drained condition was adopted to magnify the significance of pipeline-backfill-trench interaction. Several important aspects were observed that are shortly summarized as below:

- Regardless of the backfill stiffness, the presence of the trench results in less ultimate lateral soil resistance against the pipe when approaching/penetrating the trench wall. This resistance reduction is due to the progressive collapse of the trench wall into the backfill. The magnitude of the reduction depends on the stiffness of the backfill and the amount of passive lateral pressure that the backfill material mobilizes against the active trench collapse.

- Soft backfill, i.e., slurry, mobilizes less passive pressure against the trench wall failure. This, in turn, causes full development of shear bands in the trench wall towards the soil surface, slipping the slices of the trench wall into the trench, and reduction of lateral soil resistance. In the case of backfills with higher shear strength, i.e., sand, the passive pressure is increased and prevent developing full shear bands. This, in turn, increases the lateral soil resistance. The chunky backfill with moderate shear strength shows a moderate p-y response in between the slurry and sand.
- Pipeline-trenchbed interaction significantly contributes to the lateral soil resistance against the pipe moving inside the trench, and also the ultimate soil resistance against the pipe penetrating into the trench wall. This effect is less significant in the case of sand backfill.

Overall, the study showed a significant effect of trenching/backfilling and the backfilling shear strength on the lateral soil resistance against the large pipeline displacements. These effects depend on a wide range of parameters including but not limited to the trench configuration, seabed soil properties, construction methodologies, nature of applied loads, site-specific conditions, etc. Incorporation of trenching/backfilling effect can result in safer and more cost-effective design.

6.6 Acknowledgments

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CHAPTER 7. The trench effect on the lateral p-y response of the shallowly trenched-backfilled pipelines in clay

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Abstract

Offshore pipelines need to be buried by trenching and backfilling in some shallow waters for physical protection. Trenched-backfilled pipelines may encounter significant pipe-soil interaction in various incidents such as ground movements, landslides, ice gouging, and external impacts of anchors and fishing gears. The pre-dredged material that is usually used for backfilling of the subsea trenches as a cost-effective solution is significantly remoulded due to the construction process and being exposed to environmental loads. The different stiffness between the highly remoulded backfill and the native ground and the pipeline-backfill-trench configuration may have a significant impact on the lateral soil resistance against the pipeline through influencing the internal soil deformations and failure mechanisms. However, this important aspect is currently not considered by existing design codes. The current study has closely investigated the soil deformation mechanism around a shallowly buried pipeline in clay through performing centrifuge tests, particle image velocimetry (PIV), and large deformation numerical analysis. The influence of different backfill/trench stiffness on failure mechanisms and resulting p-y response were investigated. The study showed that ignoring the trenching/backfilling effects may overestimate the ultimate lateral soil resistance for large displacements and underestimate it in small to medium displacements.

Keywords: Lateral pipe-soil interaction; pipeline-backfill interaction, p-y response; large deformation; centrifuge testing.

7.1 Introduction

Submarine pipelines continue to be buried for the purposes of reducing the risk of hydrodynamic force and increasing the stability of pipeline section; protecting the pipeline section from the external damage due to anchors, heavy dropped objects or fishing gear; and improving other pipeline structural performance, such as free span, lateral buckling, and insulation performance (Bai and Bai 2014). Pipeline burial is generally achieved by various trenching and backfilling methods. A variety of factors should be considered to select a trenching method for a specific pipeline route. The water depth range, maximum required trench depth are the primary considerations. If multiple trenching methodologies satisfy the primary considerations, secondary considerations must be used to determine the preferred solution. These include parameters such as seabed geology, backfill method, seabed slopes, and environmental sensitivity (Paulin et al. 2014). Most of the technologies utilize the same technology to backfill the trench as how it was excavated. Following the pipeline installation, the excavated material is normally used to backfill the trench. The properties of the backfill placed in the trench are dependent on the selected method and the construction process of trenching and backfilling. The trenching and backfilling process remoulds the excavated material and increases the water content of the backfill. As a result, the backfill will become of less strength and softer than the native ground, with a wide range of shear strengths ranging from negligible strength up to nearly the strength of native soil values.

There are a few experimental studies that have investigated the trenching and backfilling effect on large lateral pipe-soil interaction in clay. The most comprehensive experimental research is maybe the work conducted by Paulin (1998). Paulin's work was followed by the research conducted in the C-CORE centrifuge using the same methodology (C-CORE, 2003; Phillips et al., 2004). These studies were incorporated into the (PRCI 2009) guidelines, which also present practical design recommendations for buried pipes in clays that will be used herein for comparison. Paulin (1998) investigated the effect of a wide range of parameters including the effect of different backfills, soil stress history, trench geometry, pipe size, interaction rate, and burial depth through undrained, partial drained, and drained conditions. However, the author could not clearly observe the lateral pipeline-backfill-trench interaction mechanisms for more accurate assessments. The literature review presents more studies on exposed pipelines than on buried pipelines and also more emphasis on sands than on cohesive soils. Among those investigations around buried pipelines most of the guidelines and studies address the upheaval interactions (Cathie et al. 2005).

There has been a very limited number of experiments to demonstrate soil failure mechanism development with a focus on horizontal pipe-soil interaction of buried pipes in the cohesive testbed. Oliveira et al. (2010) observed the lateral pipe-soil interaction without considering the effect of the trench. In contrast, the current study was able to clearly and directly observe progressive pipeline-backfill-trench interaction mechanisms. The recorded images were analyzed by applying Particle Image Velocimetry (PIV) technique.

To investigate the lateral pipe-soil interaction, either pipe or soil will have to move relatively against each other to induce lateral resistance. This process takes place in various natural events differently, as illustrated in Figure 7.1. For example, in the ice gouging event, sub-gouge soil attacks the pipeline in the scour zone. Consequently, the trench wall will collapse as a result of sub-gouge deformations before the interaction with the pipeline. Meanwhile, in the pipeline displacement zone, the deflected pipeline approaches the trench wall. In the current experiments, the pipe moves towards the trench wall that essentially models the pipeline displacement zone rather than the scour zone. It is noteworthy that the mechanism of the pipe-soil interaction as an outcome of sub-gouge deformation in the scour zone is different from the pipeline displacement zone which is modeled in the current study. The same process occurs in the landslide ground movement (Figure 7.1). The pipeline always attacks the trench wall when the drag anchor pulls the pipeline which is exactly in accordance with the modeled condition in the current study. Inclined pulling may also be the case in anchor clash which in turn has an influence on the mechanisms of the pipe-trenchbed interaction.

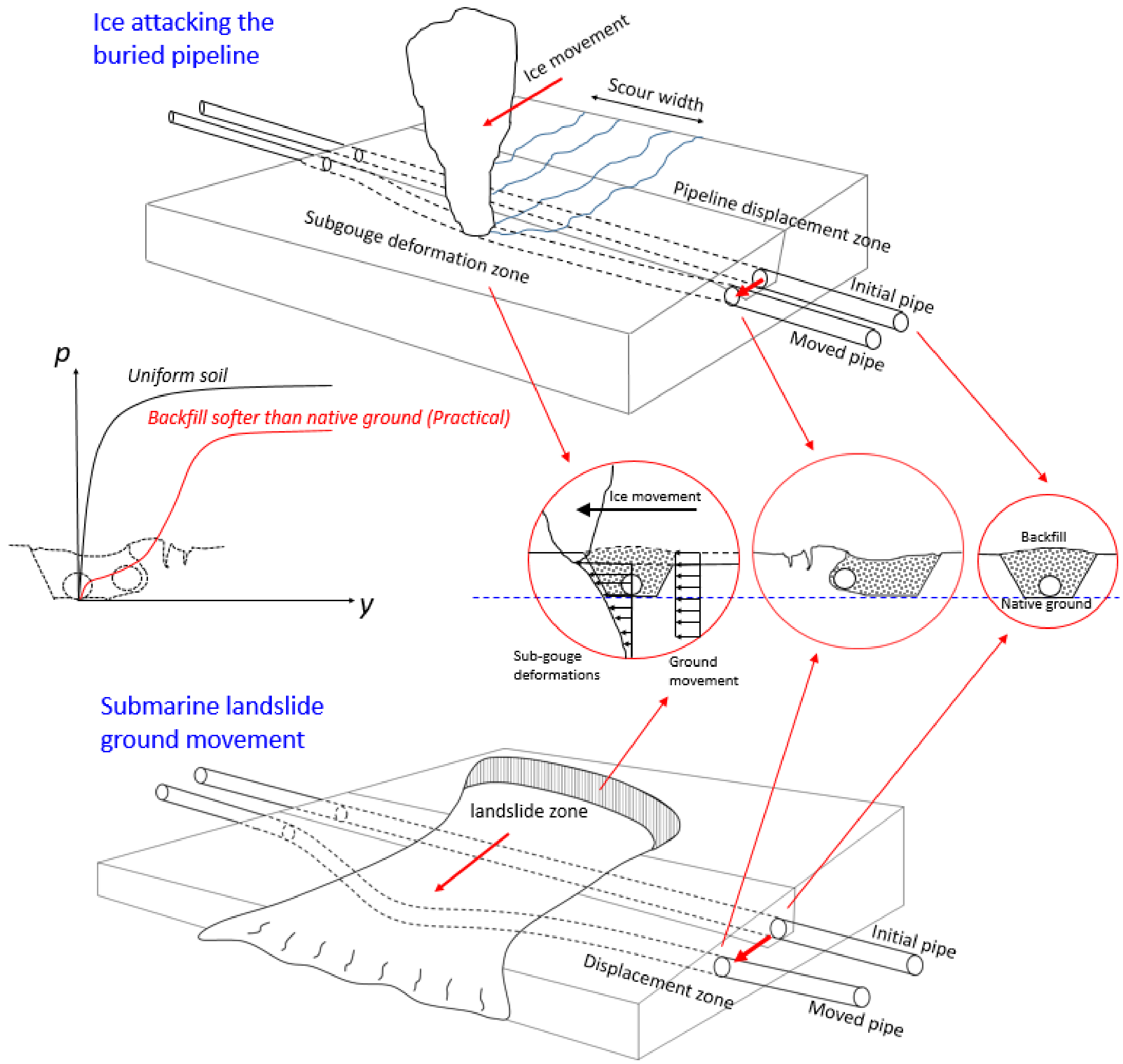


Figure 7.1. The lateral response of trenched pipeline to ice-gouging and landslide ground movement

The current study reveals the significantly important mechanisms that contribute to the mobilization of the soil resistance in the lateral pipeline-backfill-trench interaction process. The amount of initial lateral stiffness of the trenched pipeline is also attributable to the revealed mechanisms. The identified mechanisms are very dependent on the stiffness of backfill in developing the response. The mechanisms were investigated by some tests of having almost the same specifications and various backfill strengths. The study provided an insight into this challenging area of the pipeline design and prepared the ground to take a significant step ahead by incorporating the pipeline-backfill-trench interaction effects into the pipeline design practice. Exploring the identified mechanisms is expected to significantly improve the safety and the cost-effectiveness of the current practice.

7.2 Modeling considerations and testing

The testing program was conducted at C-CORE centrifuge facilities. The main objective of the project was to investigate the lateral pipeline-backfill-trench interaction in clay through monitoring of the failure mechanisms and soil deformations. The tests were prepared in a way to explore the effect of the trench in the plane strain conditions. The significant advantage of the current test set up compared to the earlier studies was the observation window that enabled direct observation of failure mechanisms. Trenches were excavated using a blade with an adjustable side angle that was mounted on a guide beam sitting on the strongbox. A trench width of about 3D was considered. The burial depth ratio (H/D) was defined as the initial ratio of the pipe springline depth to the pipe diameter. A 2D clearance was considered between the trench bottom and the lower drainage layer in the bottom of the test box to ensure that there will be no boundary effects.

The model pipe was fabricated from stainless steel pipe (31.75 mm in diameter) and instrumented with two sets of strain gauges, one internal pore pressure transducer (facing the rear of pipe), two strings of pulling cables, two rubber end caps (Figure 7.2).

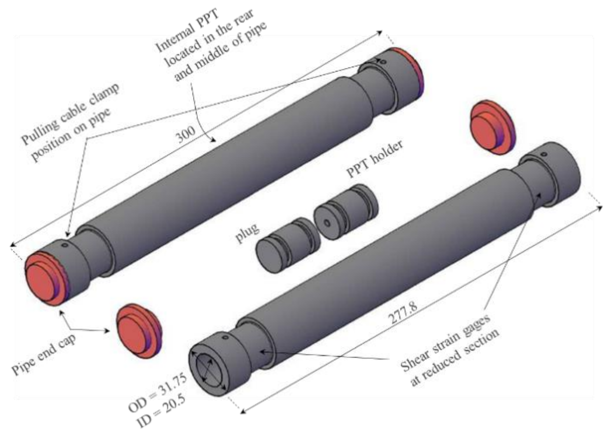


Figure 7.2. Pipe design and instrumentation

A range of instruments integrated with 3 parallel strong data acquisition systems (each has 8 individually configurable inputs) were used for full monitoring of the testing program such as pore pressure transducers (PPTs), strain gauges, load cells, conventional and laser linear variable differential transformers (LVDTs), T-bar load cell, vertical drive motion controller, and digital cameras. This enabled the full recording of progressive failure mechanisms and shear bands development in backfill and native soil, the lateral force-displacement response of the pipeline, the suction force variation behind the moving pipe, and the pore pressure variation both in the backfill and native ground. Figure 7.3 visualizes the key steps of the adopted procedure.

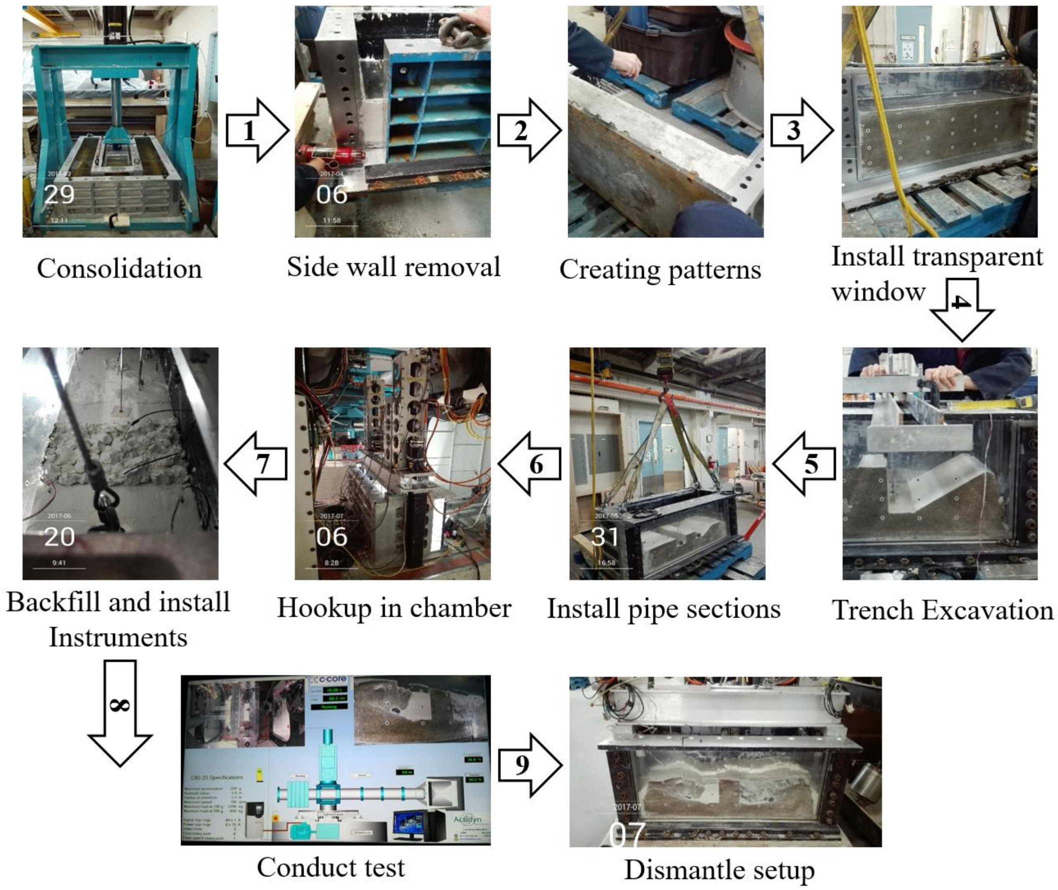


Figure 7.3. Visualized key steps of test setup preparation procedure

Two pipes were pulled in the lateral direction (T3P2 in almost horizontal and T4P2 in the inclined downward direction). Both of the pipes were pulled in partially drained condition (Normalized velocity, vD/c_v = 0.43 and 0.14). This ratio changes with the variation of pipe depth during the test. Depending on the pulling cable initial inclination, each pipe selects a special path that causes a special soil failure mechanism and force-displacement response. Figure 7.4 shows a post-test image of the trench and pipe arrangement and the corresponding failures.

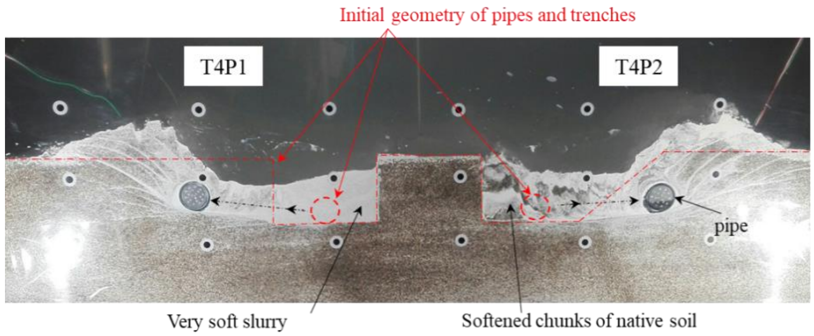


Figure 7.4. A post-test view of test-4, and the corresponding failures

The soil bed was incrementally consolidated up to the effective stress of 400 kPa and then was unloaded to 100 kPa with an open drainage valve. Then, it is unloaded to zero with the close drainage valve. This level of consolidation yielded a soft clay with an undrained shear strength of 15 to 25 kPa. However, the water introduced on top of the surface of the testbed during in-flight had a significant influence on softening the soil. To obtain the in-flight undrained shear strength of the backfill and native ground, a T-bar penetrometer was mounted on the strongbox. The spoil material that previously excavated from the trench was used for producing two types of backfill. The first type is called “slurry” which was produced by mixing water with the native soil up to water content of 3 times liquid limit. The second backfill type made up of chunks of native soil that were left in a container of water for a period of time. During this period which was shorter in test T3P2 than test T4P2, the chunks of native soil had time to take in water and soften. The difference in the period of exposure to water before backfilling caused a slight difference in the stiffness of the chunk backfills in test T3P2 and test T4P2. As a general rule, the backfill properties are a function of how the trench is excavated and backfilled. The former backfill type (slurry) may represent the backfill following jetting in soft clay producing water clay suspension immediately after trenching (DNV.GL 2017). Jetting in stiff clay may yield to a backfill consisting of lumps of semi-intact clay in a matrix of unconsolidated slurry. The latter backfill type (chunk) is considered as a model of backfill in soft clay after plowing or mechanical backfilling (Cathie et al. 2005). Plowing is probably the only method of trenching that produces sloped trench walls.

The summary of test setup preparation is explained in Kianian et al. (2018). Table 7.1 summarizes some details of the tests discussed in the current paper. Take note that the loading direction at the beginning of the test was almost horizontal in test T3P2 and slightly inclined to downward in T4P2.

Table 7.1. Summary of the experiments

Test ID	Burial depth ratio, H/D	Trench backfill type	Trench wall	Model pipe velocity (μm/s)	Normalized velocity $V_n = vD/c_v$
T3P1	1.5	Slurry	Vertical	9.4	0.43
T4P1	1.5	Slurry	Vertical	3.0	0.14
T3P2	1.6	Chunk	Inclined (36°)	9.2	0.43
T4P2	1.5	Chunk	Inclined (32°)	3.0	0.14

Table 7.2 shows a summary of the backfilling and native material properties prepared and tested in this study. Also, the proposed parameters of linear undrained shear strength are also included in this table.

Table 7.2. Soil properties

Test ID	Trench backfill type	Native water content before and after the test (%)	Υ_{sat} (kN/m3)	Native soil mudline strength, S_{um} (kPa)	Native soil Strength gradient, k_{su} (kPa/m)	Backfill mudline strength, S_{um} (kPa)	Backfill strength gradient, k_{su} (kPa/m)
T3P1	Slurry	32.04 - 32.97	18.51	15	0.15	0	0.1
T4P1	Slurry	30.12 - 32.13	18.45	15	0.15	0	0.1
T3P2	Chunk	32.04 - 32.97	18.51	15	0.15	2	1.6
T4P2	Chunk	30.12 - 32.13	18.45	15	0.15	2	1.0

7.3 Test results

7.3.1 Lateral load-displacement response

In this section, the lateral force-displacement response of the pipeline is presented for a total pipe displacement of about 3.0D to 4D. The PIV analysis results are presented in a later section of this paper for different stages of pipeline-backfill-trench interaction to explore how the obtained soil resitance is inferred from the soil deformations and failures. The effort is made to compare the results with some of the published theoretical and empirical predictions. Figure 7.5 shows the force-displacement responses against the normalized lateral displacement (y/D) of the conducted tests with the key configuration and test parameters. It is usually appropriate to normalize the load with pipe diameter and undrained shear strength to obtain the interaction factor. In this case, normalizing the load with highly different magnitude of undrained shear strength in backfill and native will result in discontinuities in p-y response, particularly with the complex form of interaction in the transition from backfill to native soil. Therefore, the load response is not represented in normalized form.

Exposed pipelines are allowed to move laterally at prescribed locations so as to relieve the axial stress. The typical lateral pipe movement within an engineered buckle is several pipe diameters (Cheuk et al. 2007). Unlike exposed pipelines, buried pipelines are designed to stay in place. And, large lateral movements of several diameters are not defined within the service state of the pipeline (DNV.GL 2017). The current range of lateral displacement (3D to 4D) is investigated to assess the displacement required to achieve the ultimate load and the rationale behind the obtained response using PIV results. The ultimate resistance may occur as a result of landslide, ice gouging, and external impact of the anchor. The results show that the ultimate load occurs after a certain penetration into the trench wall. Therefore, considering a trench would dramatically widen the required displacement to achieve the ultimate resistance. As Figure 7.5 shows, regardless of the trench slope, the ultimate resistance is influenced by the backfill strength. Trench slope and trench width affect the distance required for reaching the ultimate resistance and the shape of the p-y curve up to ultimate resistance. ALA (2005) and PRCI (2009) suggested the following equation limiting the required displacement to 10 to 15 percent of the pipe diameter.

$$\Delta_p = 0.04\left(H + \frac{D}{2}\right) \leq 0.1D \ to \ 0.15D \tag{7.1}$$

Note that the equation (2.2) is recommended for the buried pipe in the uniform soil without considering the trench effect.

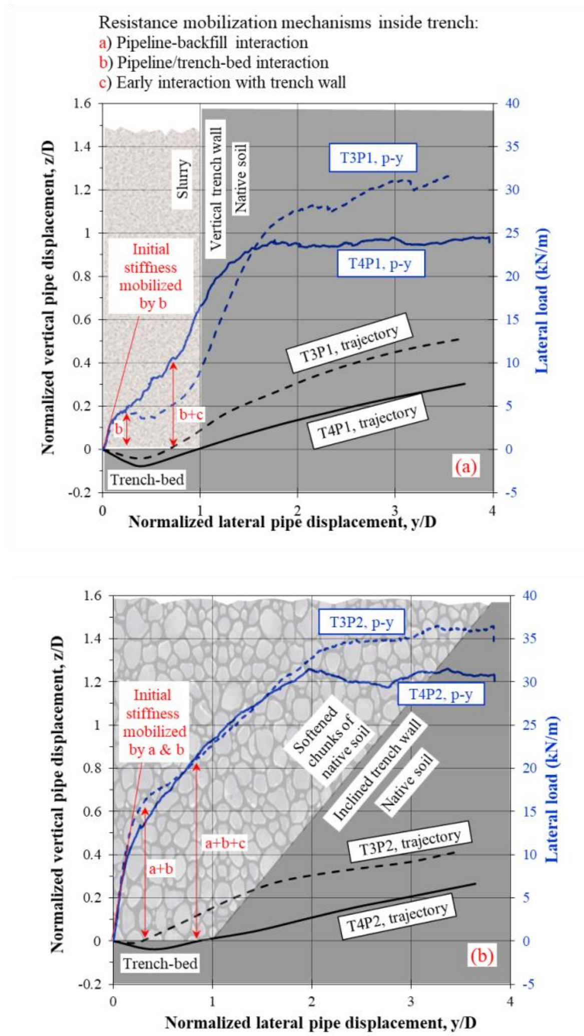
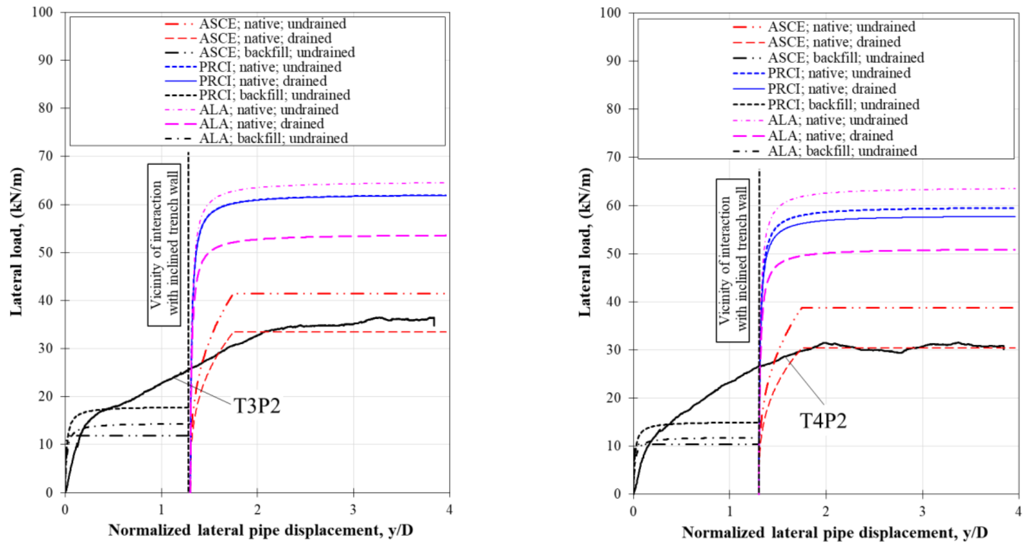


Figure 7.5. The lateral load-displacement response and corresponding pipe invert trajectory (a) very soft slurry (b) chunky backfill

The mobilized resistance of the pipeline during lateral interaction is governed by several mechanisms: a) pipeline-backfill interaction, b) pipeline/trench-bed interaction, and c) interaction with the trench wall. Figure 7.5a shows the load-displacement curve of the pipe moving in slurry backfill with negligible stiffness which cancels the influence of the pipeline-backfill interaction (mechanism a). T4P1 involves b and c mechanisms more intensively than T3P1 because of the downward component of the loading cable. Therefore, the resistance of about 4 kN/m is attributed to the pipeline/trench-bed interaction. The early interaction with the trench wall in T4P1 was due to soil berm entrapped in the trench corner. The material obtained from scrapping the trench-bed forms the berm ahead of the pipeline which was much stiffer than the slurry. This material is capable of causing early interaction with the trench wall when is entrapped between the pipeline and the trench wall. In T3P2 and T4P2 the backfill itself is stiff enough to interact with trench wall before the pipe enters into the trench wall (mechanism a). In Figure 7.5b the softened chunks of the native soil are used to backfill the trench. Further resistance running over 4 kN/m during the first half diameter of pipe movement in backfill can be attributed to the pipeline-backfill interaction. The mechanisms that contribute to the initial response of lateral pipe-soil interaction up to breakout resistance is illustrated in Figure 7.5. The initial stiffness of Figure 7.5a, and Figure 7.5b are respectively 70.2 and 132 (kN/m^2).

It is worth comparing the p-y responses with the predictions of existing design guidelines that are based on some of the key publications. Figure 7.6 presents the comparison of the test results with the load-displacement curves predicted by PRCI (2009), ALA (2005), and ASCE committee (2014). Both of the undrained and drained conditions were assumed depending on the possibility, and the plots were produced. The soil strength parameters for the undrained condition were extracted from Table 7.2, and the drained parameters were adopted from the tri-axial tests (Paulin (1998)).



(a) (b)

Figure 7.6. The comparison of the p-y responses between the test results and design codes

It is noteworthy,the PRCI (2009) and ALA (2005) guidelines are capable of considering both drained and undrained conditions to evaluate the lateral soil resistance. It is done by superposing the influence of effective friction angle and effective cohesion for drained conditions and considering the undrained shear strength for the undrained condition. The methodology proposed by the ASCE Committee on thrust restraint design of buried pipelines (ASCE committee 2014) provides separate equations for sand and clay using the friction angle and undrained shear strength, respectively. In this set of formulation, the clay under drained condition may be considered by disregarding the effective cohesion and using the effective friction angle in sand equations. Unlike ASCE committee (2014), the formulations recommended by PRCI (2009) and ALA (2005) do not cover the low range of friction angles preventing them from being used for very soft backfills under drained conditions. The results presented in Figure 7.6 shows that the design codes overestimate the ultimate load for a pipe penetrating the trench wall. This difference is attributed to the effect of the trench. This effect has not been considered by design codes. Also, the design codes underestimate the mobilized load of the pipe inside the trench by ignoring the pipe/trench-bed interaction, which is an important aspect and needs improvements to come up with a more conservative design formulation.

Figure 7.6 shows that, the initial stiffness predicted by PRCI (2009), ALA (2005), and ASCE committee (2014) is higher than the conducted tests (T3P2 and T4P2)

The normalized ultimate lateral resistance of the conducted tests against the burial depth ratio were compared with some of the key published studies in Figure 7.7 includes current experimental study, and the other experiments conducted in C-CORE (Phillips et al. (2004), and Paulin (1998)). Notably, due to the complex nature of the problem, the normalized lateral resistance is also governed by some other parameters in addition to burial depth. For example, the normalized resistance may also be regarded as a function of dimensionless overburden pressure (Rowe and Davis 1982, and Merifield et al. 2001). Guo and Stolle (2005) concluded that the non-uniqueness of the N_c (normalized ultimate resistance) versus H/D relation is induced by a combination of the effect of pipe size, model scale, and burial depth H, or more general stress level. However, illustrating the normalized resistances in Figure 7.6 as a function of burial depth is just an attempt for approximate comparison between previous works. The perfect comparison should be made by considering the effect of displacement rate, depth, trench geometry, properties of both backfill and native soil, etc. As there is enough evidence of the influence of the mentioned factors. It was observed that the results of the tests are in agreement with the study conducted by Paulin (1998). The predictions made by Wantland et al. (1979) and Tschebotarioff (1973) are also fairly close to the conducted test results. The lower and upper bound solutions proposed by Merifield et al. (2001) for anchor and the equations recommended by ALA (2005) overestimate the resistance. Note that trench influence is ignored in the recommended guidelines. The overestimation is due to ignoring the pipeline-backfill-trench interaction on the ultimate response, as shown in Figure 7.5. A similar overestimation trend was observed in the predictions made by Oliveira et al. (2010).

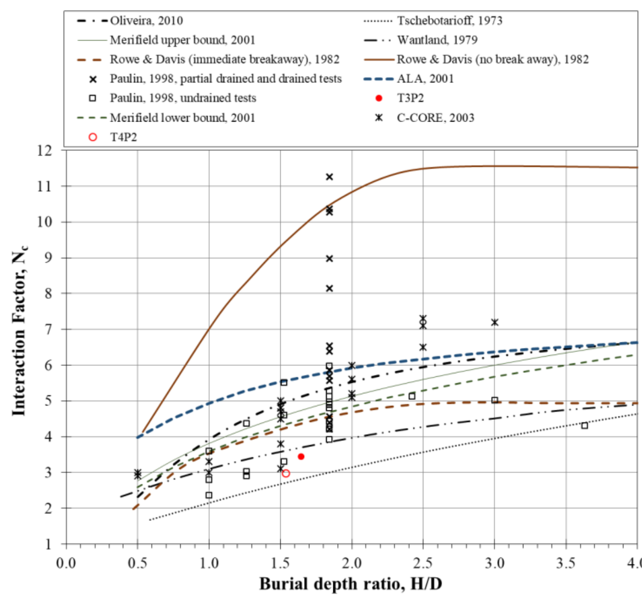


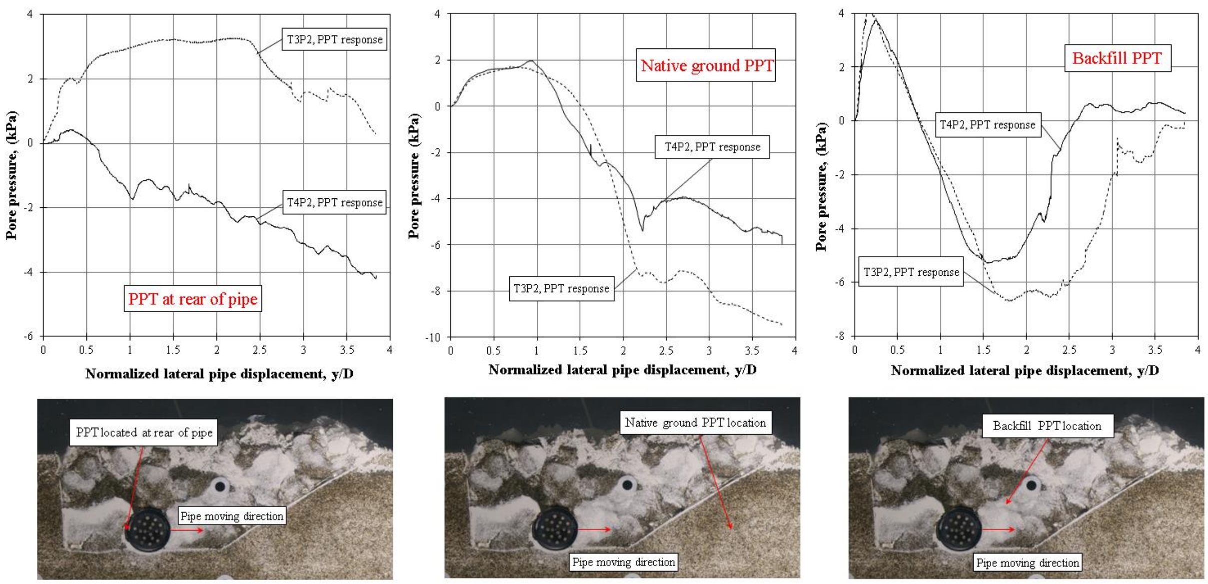
Figure 7.7. Interaction factor against the burial depth ratio

Overall, the proposed solutions and the guidelines that consider uniform soil around the pipe and ignore the highly different stiffness between the backfill and the native soil, underestimate the lateral load inside the trench, and overestimates the ultimate response.

Moreover, the required displacement to reach at the ultimate resistance is also underestimated. The presence of trench delays the occurrence of ultimate resistance which is majorly built up by the contribution of mechanism c (interaction with trench wall). Deep understanding of the source of these needs an accurate investigation of the soil deformation and failure mechanisms. These aspects are illustrated in Figure 7.9, Figure 7.10, and Figure 7.11.

7.3.2 Pore pressure variations

PPTs were placed along the pipeline moving path inside the backfill and native ground to assess the history of pore pressure variation during pipe movement. One surface PPT was used to monitor the depth of the water table on the testbed. Figure 7.8 show the variation of pore pressure against the pipe displacement at the rear of the pipe, native ground, and backfill. These variations reasonably show the partially drained condition as a fully drained condition should not depict any form of induced pore pressure due to the pipe movement. The location of PPTs was also illustrated in the mentioned figures. Figure 7.8a shows the pipeline-backfill interface pressure/suction variation that has been measured at the rear of pipes. The trend of pore pressure in the rear of the pipe in T4P2 indicates an initial increase of the pore pressure followed by the development of the negative pore pressure. The pore pressure at the rear of the T3P2 appears to contradict the measured response in T4P2. The rear pipe pore pressure response of T4P2 seems to be more reasonable and there might be something wrong with the captured pressure in T3P2.



(a) **(b)** **(c)**

Figure 7.8. Variation of pore pressure in a different location in response to pipe movement

Figure 7.8b illustrates the pore pressure variations measured at the marked position in the native ground. The measurements in test 3 and test 4 are at a high level of agreement. Test 4 with a lower displacement rate was closer to the drained condition. Thus, the induced pore pressure had enough time to be dissipated and resulted in lower induced pore pressure. Overall, with the insight obtained in the tri-axial tests on over-consolidated clay, the soil compression corresponds with positive pressure and soil failure through the shear surfaces corresponds with negative pressure in undrained or partially drained conditions (soil expansion in drained condition). The positive pressure developed at the small pipe displacements up to about 0.5D Figure 7.8(c) is in agreement with the observed mechanisms (Figure 7.9, point A) that shows the compression in the backfill. After points A and B the pipe passes the location of PPT embedded in the backfill.

7.3.3 Deformation and failure mechanisms

Observation system

The internal soil deformations and interactive failure mechanisms were observed through an acrylic window on one side of the test box using PIV analysis. High-quality images were continuously captured by two digital cameras during the pipes movement (see Figure 7.4). Each camera was set for one pipe, but the visual field of the cameras was overlapping. The cameras were fixed to the centrifuge swinging platform.

The consecutive images were introduced to particle image velocimetry software (GeoPIV-RG, originally developed by White et al. (2003) and further improved by Stanier et al. (2016) to measure the displacements and obtain strain levels at any observable domain. Circular markers shown in Figure 7.4 were attached to the internal side of the window to enable image and object space calibration in the PIV analysis. In PIV analysis, the soil particles were tracked and compared with the reference image as the pipes were being pulled. The software creates thousands of divisions called patches or subsets (e.g., ~ 2200) in part of the camera's field of view. Each soil patch covers about a 5.5 mm squared (40 pixels) soil zone, and strong correlation algorithms are then used to track these patches and extract their movement between a pair of images. This process produces a displacement field containing displacement vectors of each subset. A precision of about 0.25 pixels corresponding to a measurement precision of 0.034 mm was obtained. The nominal velocity of T3P1 and T4P1 were respectively 0.01 and 0.003 mm/s at which shutting intervals were 83 and 25 seconds to approximately capture images at 0.25 mm increments. These intervals of shutting frame were sufficient to ensure capturing of the soil deformation mechanisms.

Observations

When a buried pipe moves laterally, a complex state of shear and compressive stresses are applied in the surrounding soil. Soil inherently is capable of bearing the compressive stresses mobilizing positive pore pressure in undrained conditions (compaction in drained condition). Like ductile materials, failure of soil occurs when it cannot bear the applied shear stress and is loaded beyond material shear strength capacity. This point corresponds with maximum developed resistance and dilation in drained condition (negative pore pressure in undrained condition, see Figure 7.8). The soil failure resulted from the exceedance of a certain shear stress form mechanism of failure.

As the buried pipe moves laterally, the resulting shear stress of the surrounding soil exceeds the material shear strength in some specific surfaces called shear bands. The behavioral property of the shear band results in concentrating the further shear strains induced by pipe displacement along with the shear band (Figure 7.9, and Figure 7.10). The shear strength of the native soil decreases to residual strength along with the shear band as a result of extensive shear displacement. In order to capture the strain localizations in numerical simulations, the soil shear strength is reduced with increased plastic shear strains which is called strain softening. This would minimize the plastic work-integrated over the failure mechanism.

The shear strain concentration along with the shear band only remain active as long as the changing geometry and compatibility of the problem permits. As the pipe moves forward, the updating geometry gradually forms new shear bands and deactivates the existing shear bands. The activation process of the shear bands takes place gradually and overlaps the deactivation of the existing ones.

Typical failure mechanism in uniform soil without trench contains three distinct soil zones, including an active zone, located behind the pipe, a passive zone located in front of the pipe, and a middle static zone, positioned above the pipe crown (Burnett, 2015; Pike, 2016). With the presence of trench and backfill, the general form of the failure mechanism differs. As the shear strength and the submerged weight of the backfill material is different from the native soil, there is a need to assess their individual displacements and their interaction. This enables more accurate evaluation of the p-y response. Pipeline-backfill-trench interaction briefly refers to this context. The progressive failure taking place in large lateral movement of a trenched pipe entails three mechanisms: a) Pipe-backfill interaction; b) Pipe/trench-bed interaction; c) interaction with trench wall or native soil. Mechanism ‘a’ includes the influence of any backfill deformation or displacement considering the unit weight and strength of backfill. Mechanism ‘b’ involves any soil failure/deformation, and lateral breakout at the trench-bed in addition to soil berm developments. Mechanism ‘c’ engages the failure and deformation and displacement of the native soil (except trench-bed) considering the unit weight and strength of the native soil. The total p-y response is generated by superimposing the load mobilized by these three mechanisms.

The incremental displacement field and the corresponding real images are presented in Figure 7.9, and Figure 7.10.

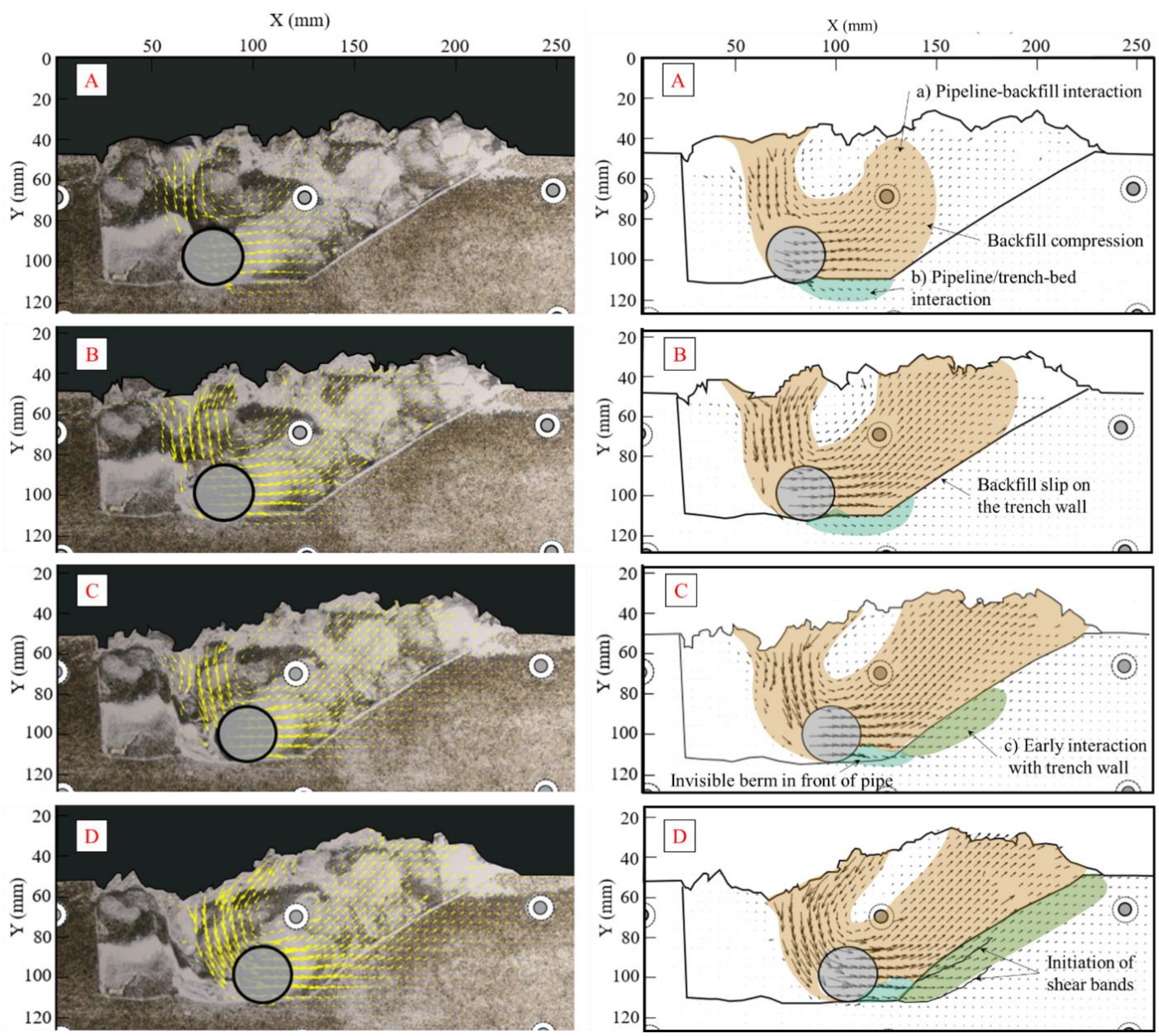


Figure 7.9. Displacement fields and contribution of mechanisms (inside the trench, point A to D)

At small values of pipe displacement, ‘a’, and ‘b’ mechanisms mobilize the resistance up to half of pipe diameter displacement. The chunk backfill consisted of softened lumps of native soil in a matrix of slurry. The backfill in front of the pipe is compressed and the slurry expelled out of the voids of the backfill. This is also consistent with the positive pore pressure mobilized in the backfill at small values of pipe displacement (Figure 7.8c). The compression at point A leads to volume reduction and strength increase of the backfill. At point C and to some extent B, the backfill is no more compressible like point A. Pipe-backfill interaction (mechanism a) is the resistance mobilized as a result of the flow of backfill both in a passive zone (in front of the pipe) and active zone (behind the pipe). The suction behind the pipe was developed as the material flows into the void left behind the displaced pipe (Figure 7.8, and Figure 7.9). The mechanism ‘c’ was initiated before the pipe itself physically reach the trench wall. This is called early interaction with the trench wall that may be induced by the mechanisms ‘a’ and/or ‘b’. Mechanism ‘a’ may cause early interaction with the trench wall when the backfill is strong enough to intervene in the pipe and the native soil interacting with the native soil before the pipe reaches the trench wall. On the other hand, mechanism ‘b’ may produce an intervening berm in front of the pipe interacting with the native soil ahead of time. There is another scenario in which the mechanism ‘b’ leads to early interaction with the trench wall. In this scenario, the limited trench width may extend the pipe-trenchbed failure to outside of the trench preventing the break out from being developed before the pipe reaches the trench wall. As soon as the mechanism ‘c’ is activated around point C, its portion in building up the total resistance dominates. And accordingly, the influence of other mechanisms is gradually faded. This is because of the higher shear strength and unit weight of the native soil in comparison to the backfill material.

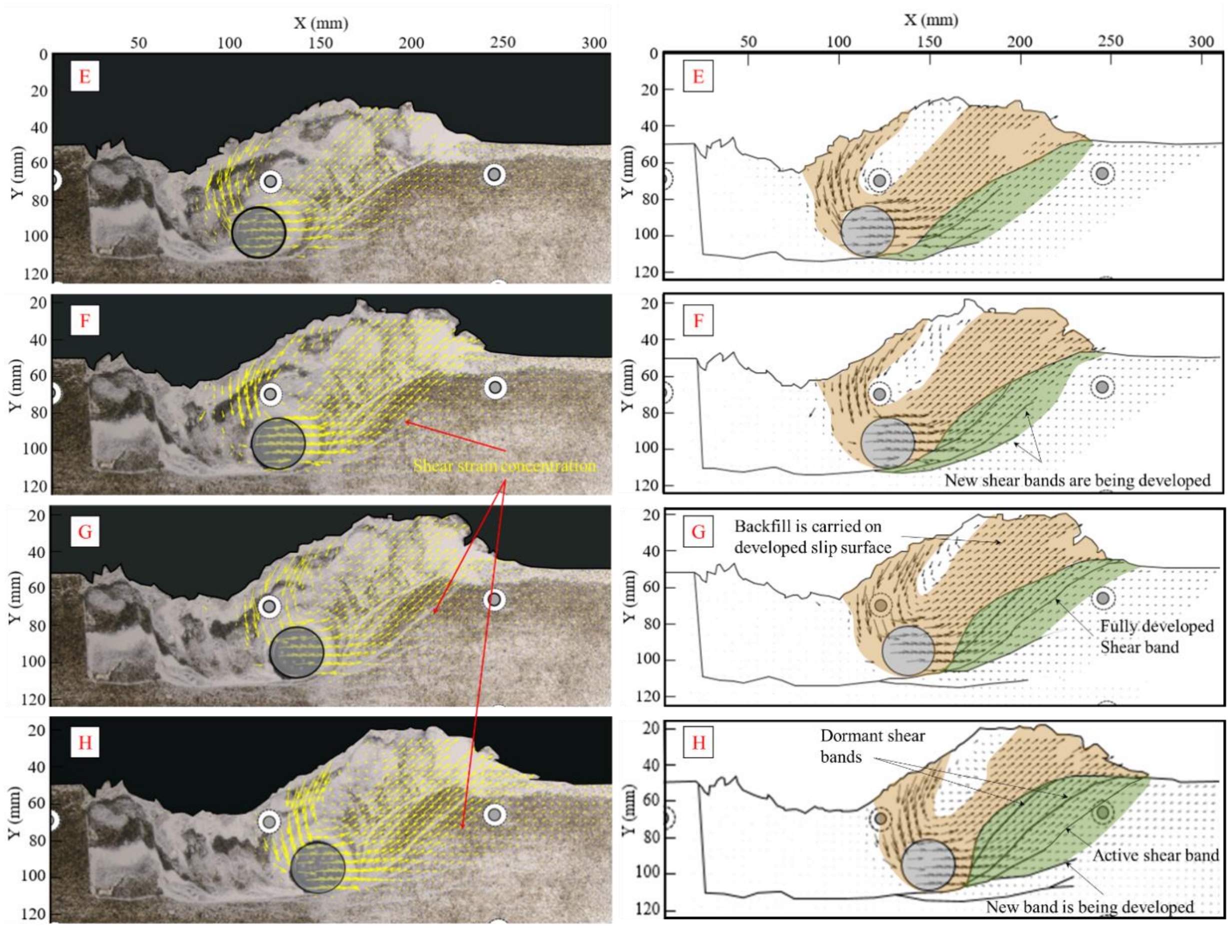


Figure 7.10. Displacement fields and contribution of mechanisms (outside trench, point E to H)

The displacement fields have been plotted at preselected stations of normalized lateral displacement (y/D). The chosen points have also been highlighted on the p-y curve (see Figure 7.11) along with the pipe trajectory in the normalized space. This figure also depicts the conceptual contribution of each individual mechanism in forming the total response of the pipe along its trajectory. The resulting total shear strains at the corresponding points are shown in Figure 7.12.

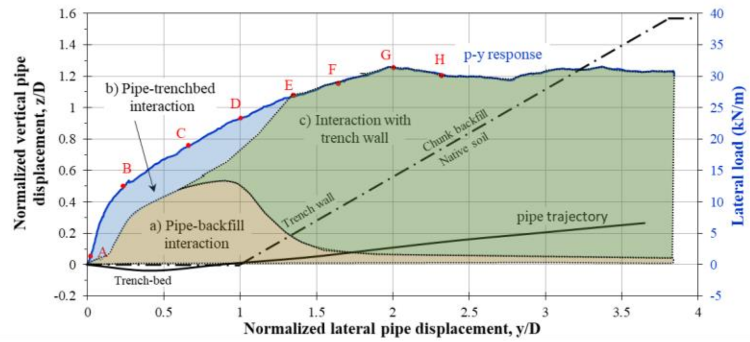


Figure 7.11. Superposition of the mechanisms contributing to p-y response

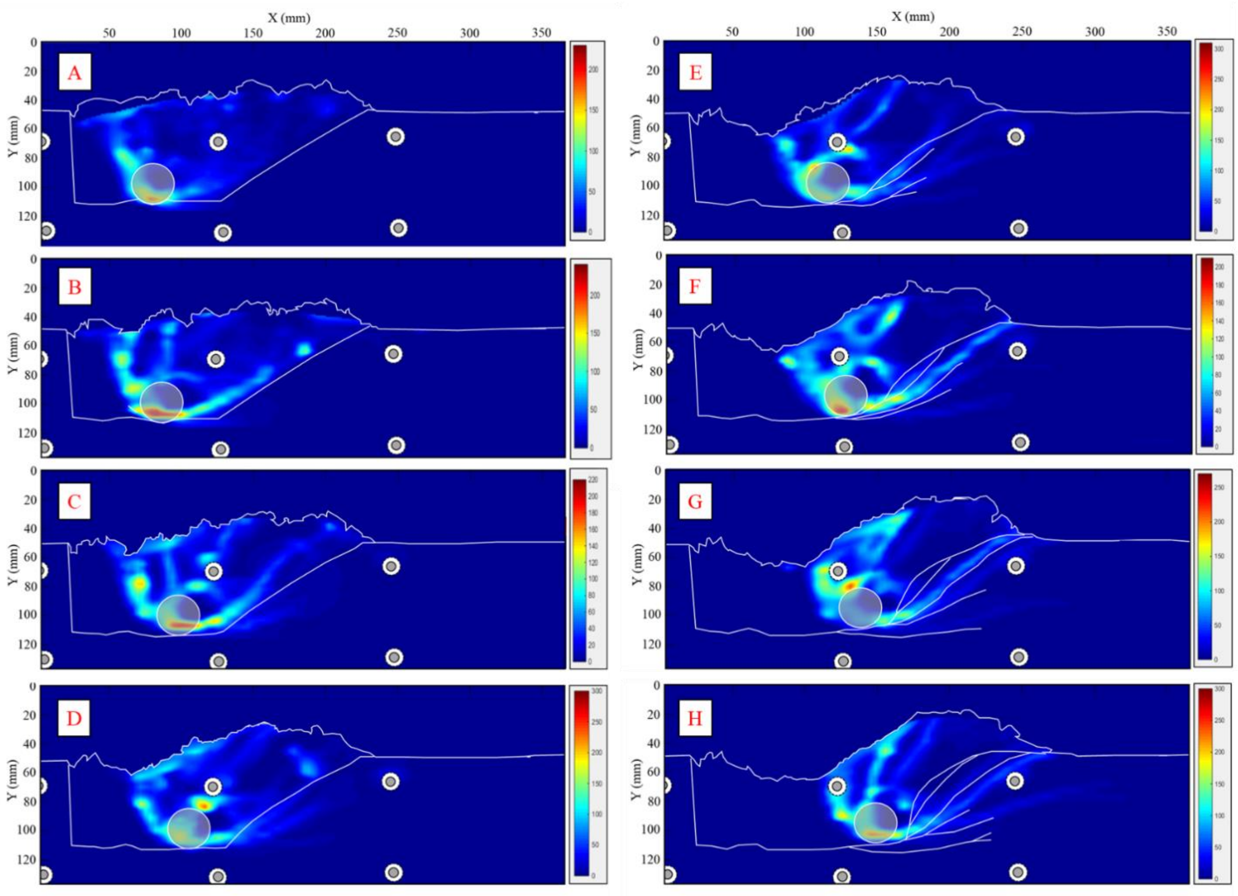
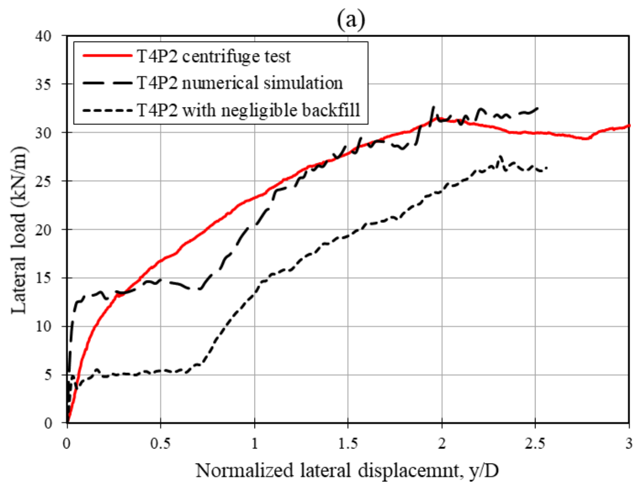


Figure 7.12. Total shear strains at the corresponding points

7.4 Numerical simulation

The numerical simulations in this study were conducted using Coupled Eulerian-Lagrangian (CEL) analysis technique in Abaqus/Explicit (SIMULIA 2017). The eight-node linear brick Eulerian elements were used with reduced integration (one integration point) and hourglass control (EC3D8R). The CEL framework allows only three-dimensional modelling. Therefore, the analyses were performed with only one element in the pipeline axial direction, and the plane strain conditions are imposed. This reasonable assumption is due to a practical fact that the lateral and vertical displacement of the pipeline takes place over a very long section of the pipeline. Two different Eulerian materials were used for modelling the backfill and native soil. The soil is modelled as an anisotropic continuum, with considering the Tresca yield criterion to simulate the undrained condition. The undrained shear strength of soil was considered to be varying with the soil depth. The elastic behavior was defined by a ratio of Young's modulus to the shear strength of E/s_u = 300 and a Poison's ratio of v = 0.495. The effect of strain-softening is also adapted based on the empirical equation proposed by Zhou and Randolph (2007). Penalty method was used for considering friction between pipe/backfill and pipe/native soil. The friction coefficient is taken as 0.1. The maximum shear stress at the interface is limited to half of the soil's undrained shear strength at the pipe center (interface roughness = 0.5). Figure 7.13 shows the load-displacement of the T4P2 along with another simulation with a negligible backfill strength but with intact backfill density. This would cut out the effect of pipeline-backfill interaction and the resulting resistance inside the trench is purely under control of pipeline-trenchbed interaction and also early interaction with the trench wall. Although the pipeline-trenchbed interaction in the original simulation is not exactly equivalent to the model with negligible backfill, it can represent the contribution of mechanism 'b' in Figure 7.11. The pressure of backfill on the trenchbed and backfill shear strength resists against the formation of surface heave. This would also have an impact on the lateral resistance and pipe trajectory. The deeper penetration of pipe into trenchbed during the lateral breakout in the model with negligible backfill is reasonably shown in Figure 7.13b. The difference between the pipe invert trajectory in test and numerical simulations in the larger displacements may be originated from the downward component of the pulling cable in the centrifuge test. The larger the displacement, the greater the downward component of the pulling cable. This behavior has not been simulated in the numerical model. The numerical simulations were modeled with a constant lateral velocity and constant vertical force as the pipe weight.



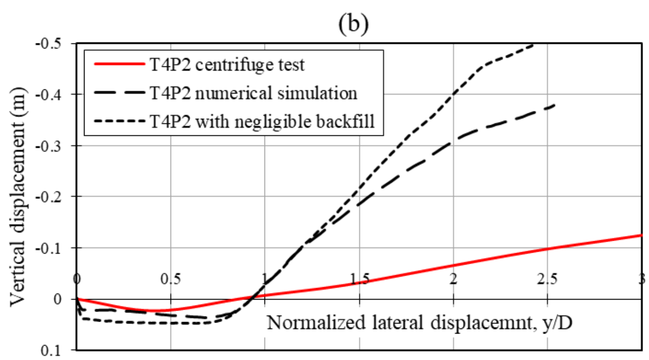


Figure 7.13. Test and numerical simulation results (a) load-displacement response (b) pipe trajectory

7.5 Conclusions

To improve the understanding of lateral pipeline-soil interaction and aid the development of numerical and analytical models for predicting the response, an experimental study was conducted in C-CORE centrifuge center. The lateral force exerted on the buried pipe during large lateral displacement has been examined in some centrifuge models and simulated using the finite element method. The results presented in the paper illustrate some potential mechanisms observed using the PIV analysis. The mobilized resistance of the trenched pipeline is developed by superimposing the effect of several mechanisms: a) pipeline-backfill interaction; b) pipeline-trenchbed interaction; c) interaction with the trench wall. There is a need to evaluate the influence of every mechanism individually as the native/backfill properties are different. This paper concentrated on pipeline-backfill interaction by using some backfills with different shear strengths. It has been demonstrated that backfill shear strength plays a key role in mechanism 'a'. This mechanism not only actively contribute to the p-y response inside the trench, but also has its own share in the ultimate resistance. The significance of the backfill strength on the pipe trajectory and amount of berm is also illustrated. Although, its contribution to ultimate resistance is gradually compensated by mechanism 'c'. The rationale behind the amount of initial lateral stiffness is interpretable by the identified mechanisms.

7.6 Acknowledgments

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CHAPTER 8. Undrained large deformation analysis of trenched pipelines under lateral displacements

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This chapter has been submitted as a journal paper.

Abstract

Subsea pipelines are commonly trenched and backfilled in shallow waters for physical protection. Using the pre-excavated spoils for backfilling is a cost-effective solution that is widely used in practice. This kind of backfilling material is highly remoulded during the construction process and being exposed to environmental loads. The buried pipelines may undergo large lateral displacement because of the ground movement, ice-gouging, etc. The experimental studies have shown the different stiffness between the backfill and native ground along with pipeline-backfill-trench configuration may significantly affect the interactive soil deformation and failure mechanisms around the pipe and the lateral soil resistance, consequently. However, the trenching and backfilling effects are currently neglected by the existing design codes. In this paper, the pipeline-backfill-trench interaction was comprehensively investigated using Coupled Eulerian-Lagrangian (CEL) analysis and verified against the test results. The study revealed the significance of several important parameters and their effect on lateral response of the buried pipelines including pipeline-trenchbed interaction, strain softening, pipe surface roughness, backfill shear strength, etc.

Keywords: Lateral pipeline-backfill-trench interaction, trench, load-displacement response, large deformation finite element analysis, Numerical modeling, Coupled Eulerian-Lagrangian method

8.1 Introduction

Subsea pipelines are key elements for the transportation of hydrocarbons in offshore oil and gas fields. Subsea pipelines may be either laid on the seabed or buried inside the trenches along their route (Figure 8.1). Pipeline burial in trenches could efficiently protect the subsea pipelines from potential instability caused by the environmental and external loads. It also improves the structural performance of the pipeline by providing further lateral/vertical restraint. Pipeline burial inevitably includes trenching/backfilling processes that disturb and remould the surrounding soil.

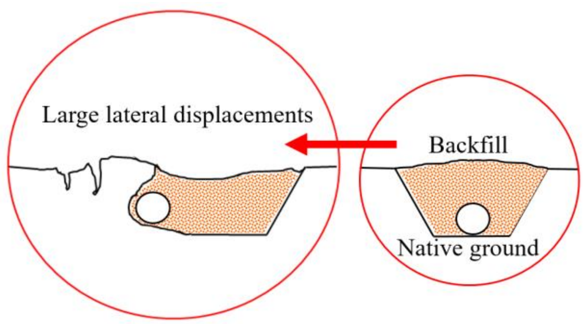


Figure 8.1. Schematic view of pipeline protection by trenching and backfilling

The lateral load-displacement response of the trenched pipelines is of practical importance in pipeline engineering. Landslides, ground movements, ice gouging, drag anchor are several sources of risks that may cause pipelines to experience large lateral deformations. Several parameters affect the lateral response of the trenched/backfilled pipelines such as shear strength of backfill and native ground, pipeline-trenchbed interaction intensity, burial depth ratio, pipe weight, and pipe roughness. The trenching effect is not considered by current pipeline design codes resulting in an overestimated or underestimated lateral soil resistance depending on displacement magnitude. An accurate assessment of the trenching/backfilling effect on lateral soil resistance needs in-depth and comprehensive investigation of these parameters through experimental and numerical studies. There are only a few experimental studies that have investigated the trenching effect on lateral pipe-soil interaction (e.g., Paulin 1998; Kianian et al. 2018; Kianian and Shiri 2019). However, this kind of experimental in cohesive soil with a large number of parameters to be investigated can be very costly and time-consuming. On the other hand, there is still no plasticity solution for large deformation problems and the limit analysis, which can be an efficient approach in small deformation problems are not capable of simulating the progressive formation of failure surfaces and shear bands in large deformations problems. Therefore, the large deformation finite element (LDFE) verified against the test results can efficiently contribute to this field of study.

In this study, the trenching/backfilling effect was comprehensively investigated through performing undrained analysis using advanced Coupled Eulerian-Lagrangian (CEL) analysis in Abaqus. The CEL analysis in ABAQUS works under an explicit scheme and is not able to model the coupled effect of pore pressure and soil particle interaction that needs implicit analysis. Therefore, only the undrained soil condition was considered in the conducted analyses. Although, assuming undrained soil condition is appropriate in the most of the natural subsea geohazards, but accurate assessment of the pore pressure variation effect and suction force mobilization behind the moving pipe needs other coupled numerical techniques such as remeshing and interpolation technique with the small strain (RITSS) (Hu and Randolph 1998). The effect of strain-softening on the soil shear strength was also incorporated in the numerical model. The analysis results were verified against the existing experimental studies (Paulin 1998; Kianian et al. 2018; Kianian and Shiri 2019) and a comprehensive parametric study was conducted to investigate a series of influential parameters. Several important observations were made that showed a good agreement with conducted test results in undrained conditions. Overall, the study showed that the current design practice that uses uniform soil condition and neglects the trenching effect overestimates the lateral soil resistance for large pipeline displacements (several diameters) and underestimate it for small (inside the trench) to moderate displacements (approaching the trench wall).

8.2 Earlier studies and current practice

The trenching/backfilling effect is currently not considered by pipeline design codes and recommendations (e.g. DNV.GL 2017; PRCI 2009; ALA 2005; and ASCE committee 2014) due to less-explored pipeline-backfill-trench interaction and its impact on the soil failure mechanism. The existing solutions for prediction of lateral soil resistance are usually based on lateral pipe-soil interaction in uniform soil (Wantland et al. 1979) or anchor or pile soil interactions in uniform soil as well (Hansen 1948; Hansen and Christensen 1961; Das and Seeley 1975; Rowe and Davis 1982; Merifield et al. 2001).

Despite the exposed pipelines laid on the seabed and the pipeline buried in the uniform soil, there is a very limited number of experimental studies (Paulin 1998; Kianian and Shiri 2019; Ng 1994; Popescu et al. 1999) and a few numerical studies (Phillips et al. 2004; Kouretzis et al. 2013; Chaloulos et al. 2015) that have considered the effect of trenching

/backfilling on lateral response of the pipelines. Phillips et al. 2004 examined the trench effects using numerical models and a centrifuge model (under an acceleration of 50 g). The results showed that the existence of a trench and an increase in trench width mitigate the pipe response in lateral displacement. Kouretzis et al. (2013) investigated quantitatively the size and the shape of the failure surface for laterally displaced pipelines in loose and medium dense sand backfill. Chaloulos et al. (2015) investigated the pipeline backfilled with sand in a trench excavated in the cohesive ground. The shape and size of the failure mechanism, as well as the potential trench effects on soil pressures and pipeline strains in the case of a strike-slip fault rupture, were investigated. The study showed that for small embedment depths soil failure extends to the ground surface, in the form of a general shear failure mechanism, while for larger depths it becomes progressively localized and surrounds the pipeline. The authors also observed that ultimate soil pressure increases exponentially with decreasing trench width, leading to high bending strains in pipelines subjected to differential lateral ground displacements. The study is limited to the initiation of LDFE analysis in a case study including the sand backfilling of the pipeline trenched in soft clay. The authors reported a significant interaction between the pipeline, sand backfill, and the trench wall in cohesive native ground. Considering the advantages of the numerical studies and the limitations of the few published studies, there is still a significant need for comprehensive analysis and simulation of a range of key parameters affecting the trenching/backfill effect on lateral pipeline response which was the key objective of the current study.

8.3 Numerical model

8.3.1 Model configuration

Coupled Eulerian-Lagrangian (CEL) approach was undertaken using Abaqus/Explicit to conduct large deformation analysis of the laterally displaced pipeline. The CEL technique helped to overcome the serious mesh distortion and highly localized shear strain throughout the failure surfaces that can happen in conventional finite element analysis. The model configuration was adopted from some of the tests (T4P4 and T2P1) conducted by Paulin (1998) to enable comparison with published test results. Figure 8.2 shows the entire domain consisting of soil as Eulerian material, void space at the seabed surface, and the buried rigid pipe as Lagrangian material. Two Eulerian materials representing native and backfill soil flew through the fixed mesh.

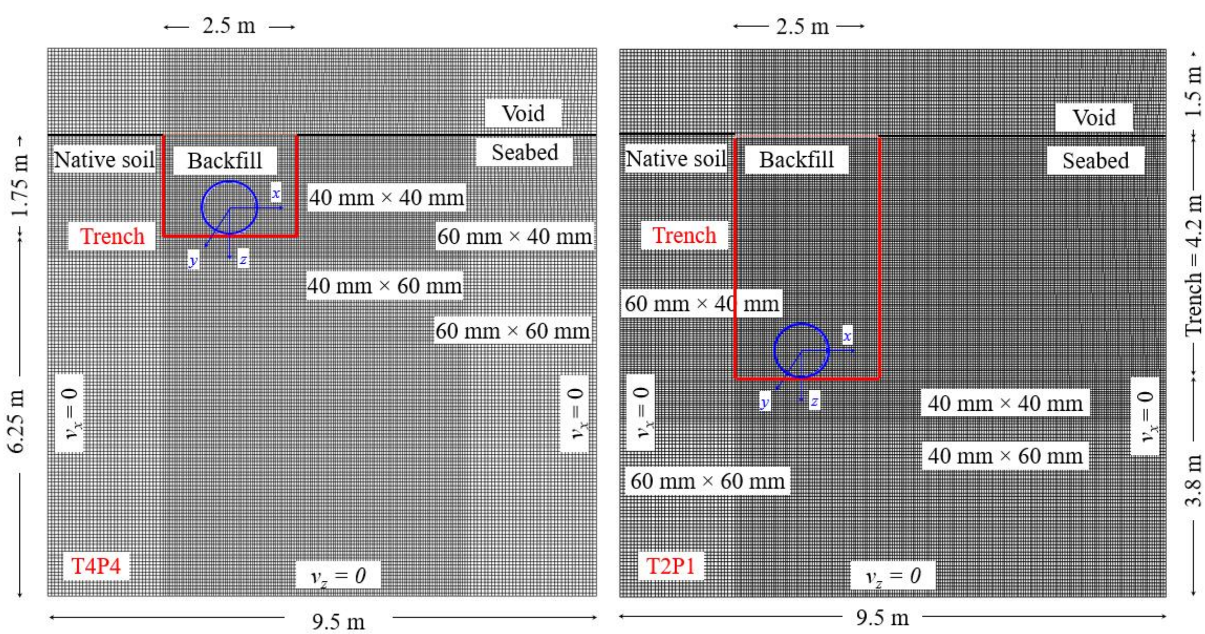


Figure 8.2. Model configurations adopted from the tests T4P4 and T2P1 of Paulin (1998)

The void space above the seabed was defined to accommodate the surface heave during the pipeline movement. The Eulerian volume fraction (EVF) tool of Abaqus was used to define the initial conditions of the elements in terms of what fraction of each element is occupied by any of the Eulerian materials. The computed volume fraction for each element was a value between 0 and 1. The unit volume fraction means that the element is thoroughly occupied by that Eulerian material, whereas zero volume fraction denotes the zero fraction of element is filled by that Eulerian material. The EVF was computed for both the Eulerian materials in every increment. For instance, when the Eulerian Volume Fraction (EVF) of two Eulerian materials is both 0.5 for an element, it means that that the element is entirely filled with the same amount of native and backfill soil with no void.

The eight-node linear brick Eulerian elements (EC3D8R) were used with reduced integration (one integration point) and hourglass control. Linear brick element is the simplest and best element for this problem as there is no need for higher order elements. The CEL framework allows only three-dimensional modelling. Therefore, the analyses were performed with only one element in the pipeline axial direction, and the plane strain conditions were imposed. This assumption is originated from a practical fact that the lateral and vertical displacement of the pipeline takes place over a very long section of the pipeline. The finer mesh was used at the central strips of the domain, where the major displacements of the soil were presumed to take place. Table 8.1 summarizes some of the key geometrical and geomechanical parameters of the model and the centrifuge tests (T4P4 and T2P1) on a prototype scale.

Table 8.1. Trench geometry and soil properties adopted from the centrifuge tests

Property	Test	Value
Geometry		
Pipe diameter (m)	T4P4	0.95
	T2P1	0.95
Burial depth to pipe center (m)	T4P4	1.27
	T2P1	3.72
Trench width (m)	T4P4	2.5
	T2P1	2.5
Native soil		
Saturated unit weight (kN/m^3)	T4P4	19.12
	T2P1	18.98
Undrained shear strength at pipe centerline (kPa)	T4P4	33.1
	T2P1	41.2
Linear variation of undrained shear strength with depth (kPa)	T4P4	24.43+6.8z
	T2P1	20+5.69z
Backfill		
Saturated unit weight of backfill (kN/m^3)	T4P4	17.27
	T2P1	17.08
Undrained shear strength at pipe centerline (kPa)	T4P4	1.6
	T2P1	3.5
Linear variation of undrained shear strength with depth (kPa)	T4P4	1.26z
	T2P1	0.94z

The soil shear strength profile with depth was adopted from the linear curve fit proposed by Paulin (1998) and presented by using the undrained shear strength at mudline (s_{um}) and the shear strength gradient (k) with depth (z). The undrained shear strength was taken at the pipe springline elevation in constant strength simulations. The soil was modelled as isotropic/anisotropic continuum material with the Tresca yield criterion (equivalent to the Mohr-Coulomb yield criterion with zero friction angle). The soil elastic behavior was defined by Young's modulus to shear strength ratio of E/s_u = 500, and a Poison's ratio of v = 0.495 to ensure zero volume change. The effect of strain-softening was incorporated by using the empirical equation proposed by (Zhou and Randolph 2007).

$$s_u = \left[1 + \mu \log(\frac{\max(|\dot{\Upsilon}_{max}|, \dot{\Upsilon}_{ref})}{\dot{\Upsilon}_{ref}})\right] [\delta_{rem} + (1 - \delta_{rem}) e^{-3\xi/\xi_{95}}] s_{u0} \tag{8.1}$$

where μ is the rate of strength increase per decade, s_{u0} is the original shear strength at the reference shear strain rate prior to any softening, δ_{rem} is the inverse of the clay sensitivity ($S_t = 2$), ξ is the accumulated absolute plastic shear strain at the Gauss point, and ξ_{95} is the value of ξ for the soil to undergo 95% remoulding. The recommended values of ξ_{95} are in the range of 10-50 and here was taken as ξ_{95} =25 (i.e., 2500% shear strain). A typical value of $3 \times 10^{-6}\ s^{-1}$ was assumed for reference shear strain rate ($\dot{\Upsilon}_{ref}$).

8.3.2 Simulation procedure and schedule

The simulations started with a geostatic step to initialize the prototype stress condition in the soil body. In the second step, the pipe was pushed downwards to achieve the specified initial embedment depth with a velocity of 0.05 m/s. In the third step, the pipe was subjected to a constant lateral velocity of 0.046 m/s in T4P4, and 0.026 m/s in T2P1 under a constant vertical load (i.e. the pipe weight). The pipe had no vertical displacement restraint. The lateral velocity of pipe in the centrifuge test was set to be sufficiently fast for being considered as undrained conditions, while the strain rate effect was negligible (Phillips et al. 2004).

A total number of 20 case studies were conducted with the configurations adopted from the tests T4P4 and T2P1 of Paulin (1998). Table 8.2 shows the case studies map including the examined and default parameter values, where the impact of several influential parameters on the lateral response and failure mechanisms of the buried pipelines were investigated one at a time.

Table 8.2. Simulations for assessing the influential parameters

Case name	Mesh size (mm)	Strain softening	Soil strength	Pipe roughness	Submergence	Pipe weight (kg/m)	Initial embedment (mm)	Backfill su (kPa)	Tension cut-off stress (kPa)	Burial depth (m)
T4P4-1	30	Yes	Linear	Rough	No	4300	4	1.6	Non	1.27
T4P4-2	40	Yes	Linear	Rough	No	4300	4	1.6	Non	1.27
T4P4-3	50	Yes	Linear	Rough	No	4300	4	1.6	Non	1.27
T4P4-4	60	Yes	Linear	Rough	No	4300	4	1.6	Non	1.27
T4P4-5	40	No	Linear	Rough	No	4300	4	1.6	Non	1.27
T4P4-6	40	No	Const.	Rough	No	4300	4	1.6	Non	1.27
T4P4-7	40	Yes	Const.	Rough	No	4300	4	1.6	Non	1.27
T4P4-8	40	Yes	Linear	Smooth	No	4300	4	1.6	Non	1.27
T4P4-9	40	Yes	Linear	Rough	Yes	4300	4	1.6	Non	1.27
T4P4-10	40	Yes	Linear	Rough	No	1000	4	1.6	Non	1.27
T4P4-11	40	Yes	Linear	Rough	No	6000	4	1.6	Non	1.27
T4P4-12	40	Yes	Linear	Rough	No	4300	74	1.6	Non	1.27
T4P4-13	40	Yes	Linear	Rough	No	4300	149	1.6	Non	1.27
T4P4-14	40	Yes	Linear	Rough	No	4300	4	0.1	Non	1.27
T4P4-15	40	Yes	Linear	Rough	No	4300	4	5	Non	1.27
T4P4-16	40	Yes	Linear	Rough	No	4300	4	1.6	4	1.27
T4P4-17	40	Yes	Linear	Penalty	No	4300	4	1.6	Non	1.27
T2P1-1	40	Yes	Linear	Rough	No	4300	4	3.5	Non	3.72
T2P1-2	40	Yes	Linear	Smooth	No	4300	4	3.5	Non	3.72
T2P1-3	40	Yes	Linear	Penalty	No	4300	4	3.5	Non	3.72

The test T4P4-2 was set as the baseline test. The pipe-soil interface was considered as two extreme conditions, i.e., rough and smooth. The friction coefficient was set to be 0.1 in T4P4-17 and T2P1-3 and the penalty formulation of Coulomb friction was adopted. The maximum shear strength at the interface element was specified by the interface roughness factor (α=0.5) which is multiplied by the pipe centerline strength of the backfill/native soil. A prototype pipe weight of 4300 kg/m was adopted from the details of the model pipes that were used in centrifuge tests Paulin (1998). Saturated unit weight was considered to simulate the test conditions, where the water table was kept below the pipeline and Vaseline was used on the soil surface to prevent desiccation and the highly nonlinear undrained shear strength profile.

Paulin (1998) used heavy model pipes to fulfill the rigidity assumption and to accommodate instruments. This caused the pipe weight to look unrealistically heavy. The effect of using effective soil and pipe unit weight was investigated in T4P4-9.

8.3.3 Mesh sensitivity

The influence of mesh size was assessed in terms of the force-displacement, pipe trajectory, and failure mechanisms. Four different mesh sizes of 30, 40, 50, and 60 mm were examined and the results were compared with the test T4P4. Figure 8.3 shows that except T4P4-4 with 60 mm element size, the mesh sizes in the rest of the cases were sufficient to provide mesh convergence with acceptable accuracy.

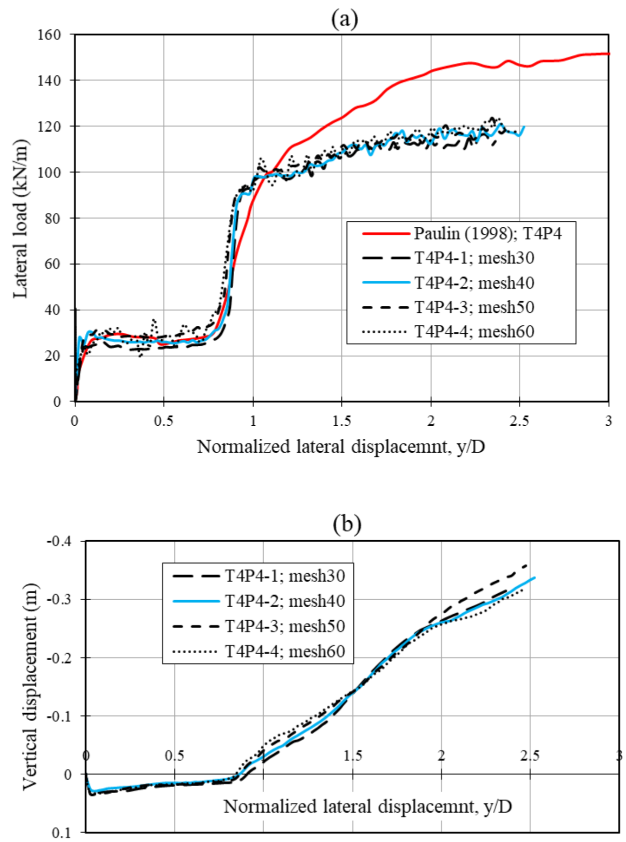


Figure 8.3. Mesh sensitivity (a) force-displacement (b) pipe trajectory during lateral displacement

The mesh sensitivity was further assessed in Figure 8.4 by examining the volume fraction average of plastic strain (maximum principal strain) contours during the formation of shear bands. Figure 8.4 also illustrates the initiation and development of shear bands that progressively update with the changing geometry and soil softening based upon the computed strains.

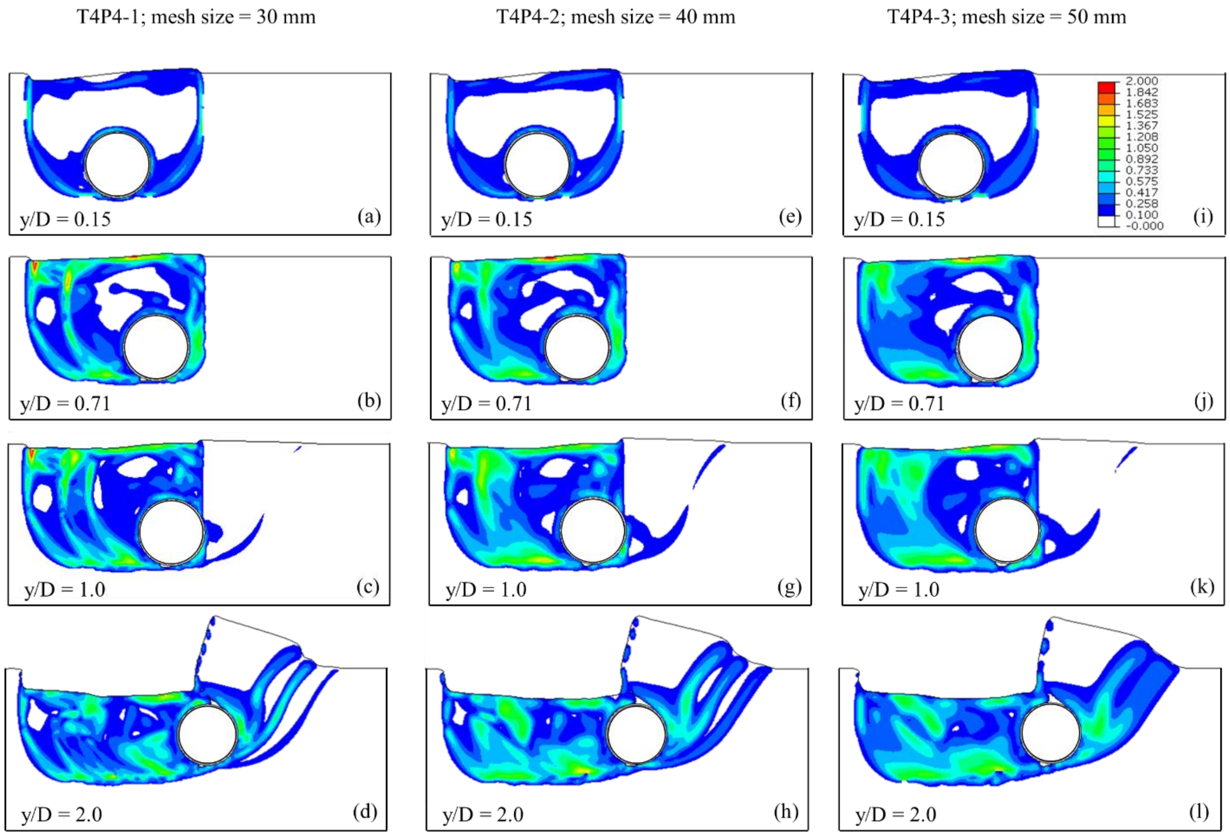


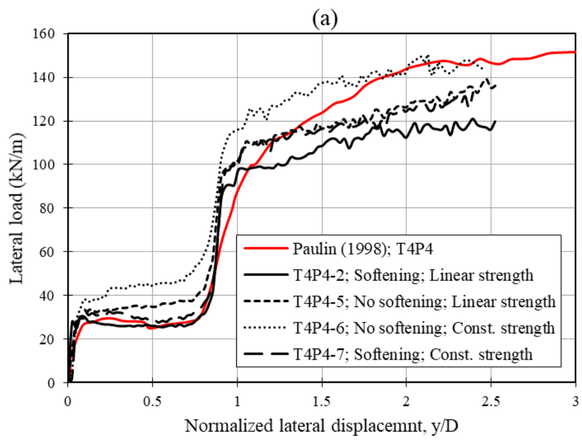
Figure 8.4. Volume fraction average of plastic strain during development of shear bands from fine (a-d), medium (e-h), and coarse mesh (i-l)

The parameter PEVAVG in Abaqus outputs a variable that refers to the plastic strain computed as a volume fraction weighted average of all Eulerian materials that present in the element (Abaqus 2017). Figure 8.4 shows that T4P4-2 with a 40 mm element size was sufficiently capable of showing the shear bands formation. Based on these observations, the case T4P4-2 simulation was chosen as a baseline case to study the effect of other parameters.

8.4 Parametric study results

8.4.1 Strain softening and soil strength variation with depth

Four different case studies (T4P4-2,5,6 and 7) were conducted with and without a softening effect and assuming linear and constant shear strength profiles. The lateral p-y responses and pipe trajectories were obtained and compared with centrifuge test T4P4 (see Figure 8.6).



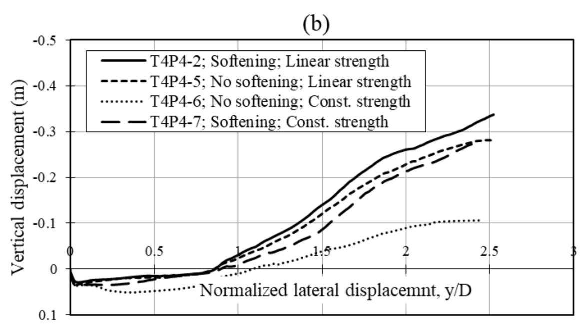


Figure 8.5. Strain softening and soil strength variation with depth (a) load-displacement (b) pipe trajectory

In order to facilitate the interpretation of the results shown in Figure 8.5 the volume fraction average of plastic strain with (a-c) and without (d-f) strain-softening were extracted and shown in Figure 8.6. In Figure 8.6a, a shear band was almost appeared forming a global wedge quickly after the pipe reached the trench wall. Another minor shear band joining the global wedge shear band was also emerged forming a triangle in front of the pipe. Further lateral movement of the pipe from this position causes strain localization mainly along with this band in which the shear strength is decreased by strain-softening equation (Figure 8.6b). As the pipe moved forward, the geometry of the problem and the shear surface were continuously updated. The soil shearing could not keep sliding over the previous surface anymore. Therefore, the mobilized resistance increased until the new shear band was formed. This process similarly continued as shown in Figure 8.6c. The developed shear strain did not localize into shear bands the same as the abovementioned process, where the strain-softening was not introduced to the simulation (see Figure 8.6(d-f)).

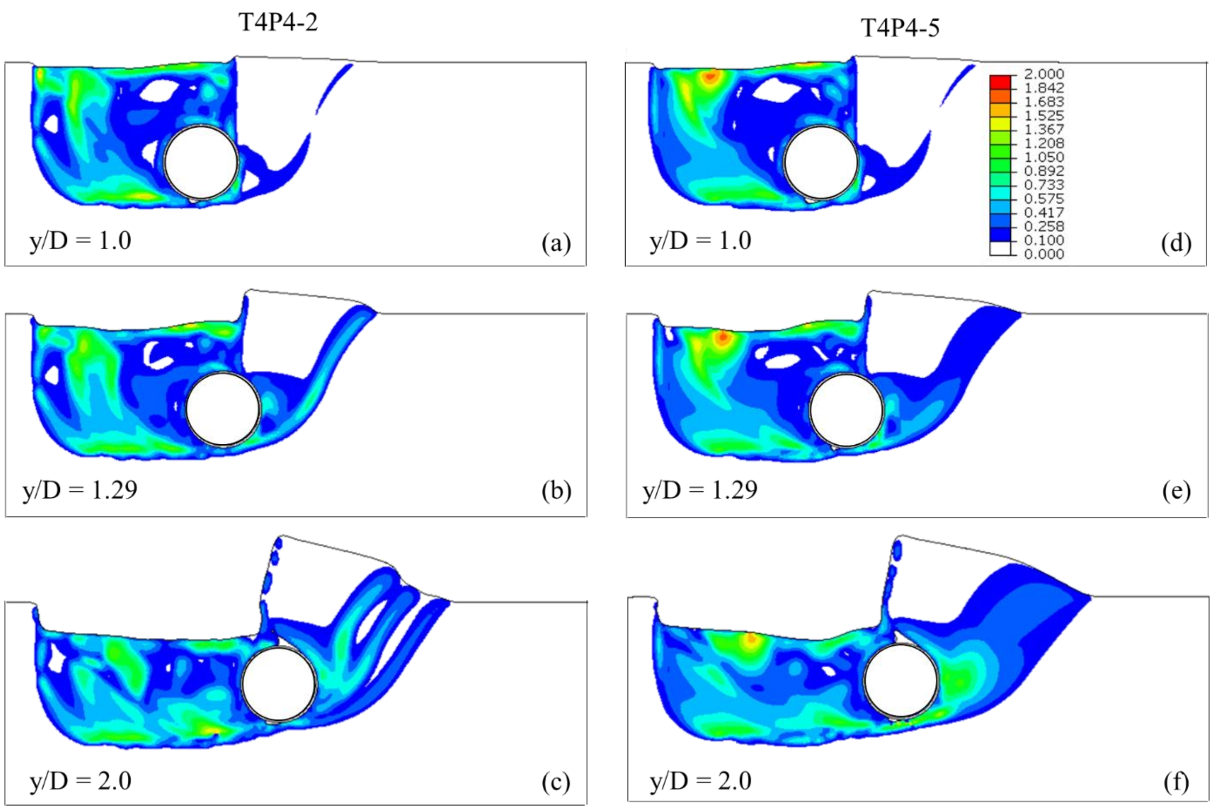


Figure 8.6. Volume fraction average of plastic strain with strain softening (a-c) and without strain softening (d-f)

Eulerian material instances interact with each other with sticky behavior. This sticking occurred because of the kinematic assumption that a single strain field is applied to all materials within an element (Abaqus 2017). This showed its impact in Figure 8.6c, and Figure 8.6f, where the backfill material has stuck to the trench wall.

The case studies T4P4-6 and T4P4-7 were the simulations with constant undrained shear strength in depth both for native soil and backfill. The value of shear strength was taken as the amount reported at the depth of pipe springline at the beginning of the test (Paulin

1998). In other simulations, the strength of backfill and native soil was assumed to be linearly increasing with depth of soil (see Table 8.1).

Figure 8.7 compares the failure mechanisms in the tests T4P4 conducted by Paulin (1998) (Figure 8.7e and f) with the velocity field vectors and failure mechanisms obtained from the numerical simulations with strain softening effects (T4P4-2, Figure 8.7a), without strain softening (T4P4-5, Figure 8.7b), and without strain-softening but with 4 kPa tension cut-off (T4P4-16, Figure 8.7c).

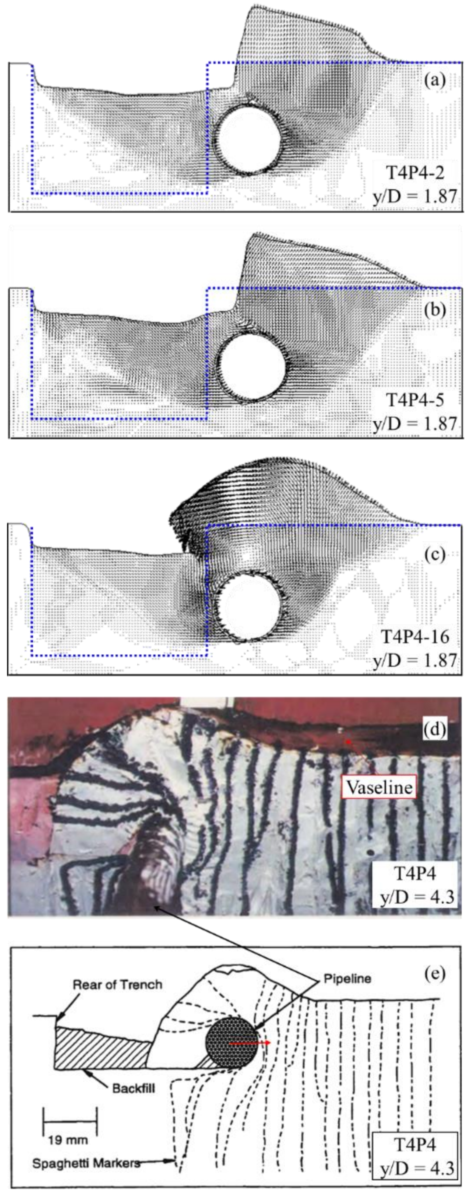


Figure 8.7. (a) Velocity field with strain softening (b) without strain softening (c) without strain softening and with 4kPa tension cut-off (d) test T4P4 at 4.3D lateral displacement Paulin (1998) (e) sketch of the test T4P4

It is worth mentioning that Paulin (1998) used Vaseline on the soil surface to prevent soil desiccation (Figure 8.7d). The native soil model after 4kPa tensile stress continues with plastic tensile stress as it faces further tensile strain. This is actually different from real soil behavior. The plastic deformation is not normally expected after the soil reaches its tensile strength. Soil is believed to be a brittle material in tension. The softening in tensile strength can be set to be zero in Abaqus. However, it dramatically increases the computational time and creates convergence problems. Vaseline usage on the soil surface might have negative side effects on the tensile mode of failure through highly plastic behavior of Vaseline. Unlike Paulin (1998), Kianian and Shiri (2019) conducted completely submerged tests without using Vaseline. This improvement helped to avoid the possible interference of Vaseline in the failure mechanisms. Figure 8.8 shows a sample of the failure mechanisms published by Kianian and Shiri (2019).

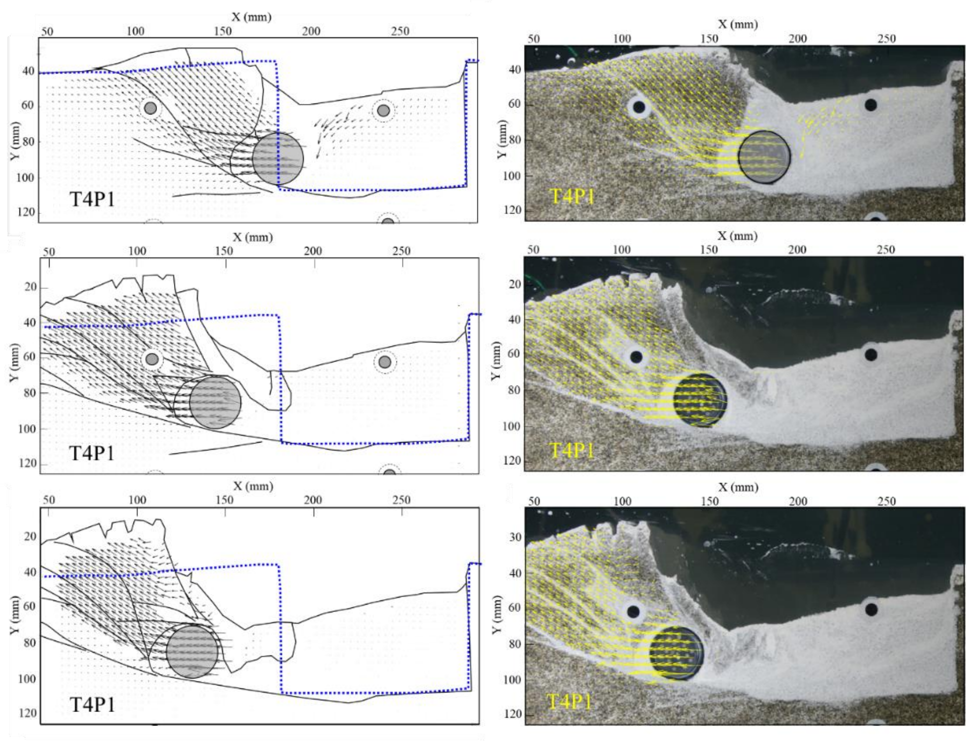


Figure 8.8. Failure mechanisms of shallowly buried pipe in drained condition (Kianian and Shiri 2019)

The pipeline shallowly trenched and backfilled in clay was pulled under the partially drained condition and the failure mechanism was obtained by PIV analysis. The failure mechanisms in Figure 8.7 and Figure 8.8 are different. The tests conducted by Kianian and Shiri (2019) under partially drained conditions have resulted in sliding failure surfaces in the trench wall that are extended towards the soil surface. However, Paulin's tests don't show any slide into the trench. Further details of these studies can be found in original publications (Paulin 1998 and Kianian and Shiri 2019).

8.4.2 Pipe weight and initial embedment into the trenchbed

The initial penetration of the pipeline into the trench bed is an important parameter in lateral soil resistance against the displacement of the trenched pipeline. However, the intensity of the effect depends on the load rate. Under the drained or partially drained conditions, the pipeline-trenchbed interaction may have a significant impact on the lateral p-y response both in small displacements inside the backfill and ultimate soil resistance (Kianian and Shiri 2019). In the undrained condition, the initial embedment effect may have a significant effect on the lateral p-y response for small displacements but a limited influence on ultimate soil resistance. This was examined by numerical analysis under the undrained condition assuming three different initial embedments of 4, 74, and 149 mm (T4P4-2, 12, and 13). Figure 8.9a shows the p-y responses and the pipeline trajectories in these analyses, where the initial penetration showed a significant effect in small pipeline displacements inside the backfill, and almost insignificant influence on the ultimate soil strength. For the pipeline moving inside the trench, increasing the magnitude of the initial embedment into the trenchbed resulted in rising up the lateral soil resistance. Figure 8.9b shows that the deeper embedment resulted in the faster upward movement of the pipeline during the lateral displacement.

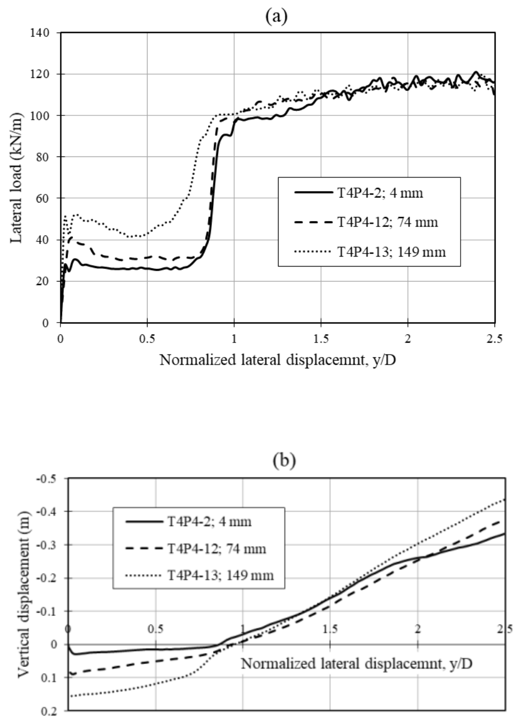


Figure 8.9. Initial pipe embedment into the trench-bed impact on (a) load-displacement (b) pipe trajectory

To better describe the bed interaction mechanisms, the volume fractions between the backfill and the native ground were extracted and presented in Figure 8.10 for cases T4P4-13 and 14. The results show that the higher the embedment depth, the larger the volume of berm forming in front of the pipe. This leads to the higher contribution of the pipeline-trenchbed interaction and mobilizing the lateral soil resistance, consequently.

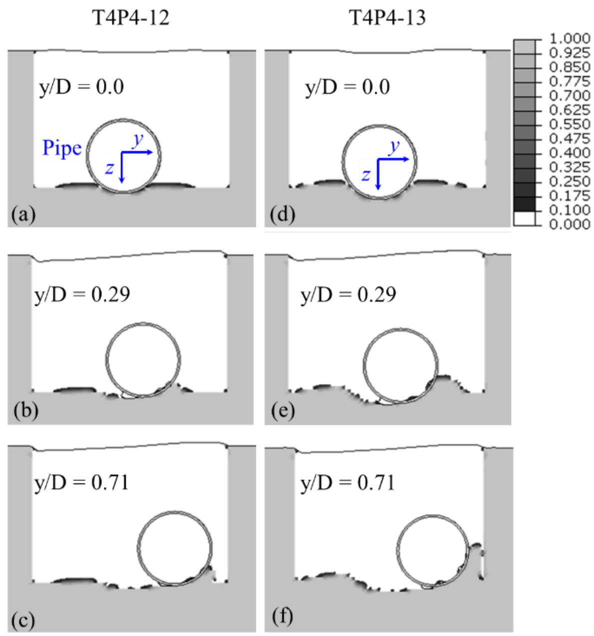


Figure 8.10. Native soil volume fraction contours showing the mobilized berm during lateral displacement

Also, Figure 8.10f shows an early interaction between the pipeline and the trench wall, where the soil berm in front of the moving pipe is enlarged and squeezed into the trench corner. This causes the pipeline and the trench wall to physically interact before the pipeline touches the wall. The mechanism shown in Figure 8.10 is in agreement with observations made by (Kianian and Shiri 2019). The pipeline weight may also affect the initial embedment. A heavier pipe can intensify the pipelined-trenchbed interaction and result in an earlier interaction with the trench wall. However, the pipe weight may also have a secondary effect on the pipeline trajectory. The heavier pipe may affect the upward movement of the laterally moving pipe. This was examined by making a comparison between the case studies T4P4-2, 9, 10, and 11 that correspond to the base case, submerged, light, and heavy pipes. Figure 8.11 shows the comparison of the p-y responses and the pipe trajectories.

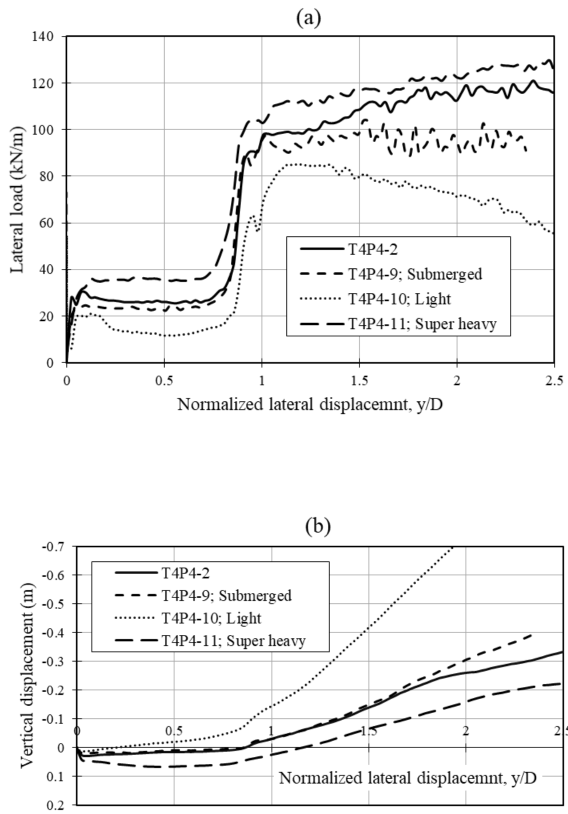


Figure 8.11. Pipe weight and submergence influence on (a) load-displacement (b) pipe trajectory

Figure 8.11 shows the effect of the pipe weight on initial embedment into the trenchbed and also the aforementioned secondary effect on pipe trajectory and the resultant lateral soil resistance. Despite the results shown in Figure 8.9, the heavier pipe not only increases the soil resistance in small pipe displacements but also increases the ultimate resistance. This happened by the weight effect on the pipe trajectory; the heavier the pipe, the less upward movement, the more volume of soil mobilization, and the more lateral resistance.

8.4.3 Backfill strength

Compared to the exposed pipelines, the trenched pipeline usually give rise to a lower volume of berm since the overburden pressure that the backfill inserts on the trench-bed and the strength of the backfill resist against the formation of surface heave. Subsequently, this would also influence the vertical/lateral resistance and the pipe trajectory. Figure 8.12 shows the lateral load-displacement response along with the pipe invert trajectory for the cases T4P4-2, 14, and 15 with different backfill shear strength of 1.6, 0.1, and 5.0 kPa.

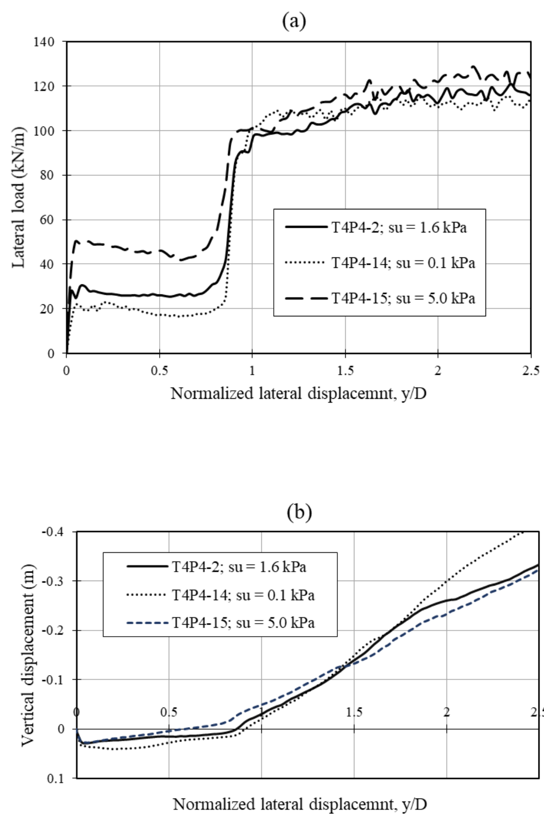


Figure 8.12. Backfill strength at pipe depth impact on the volume of the berm and mobilized resistance

The area under the trajectory curve in Figure 8.12b also is an index to the volume of the berm in front of the pipeline; the higher the shear strength of the backfill, the lower the volume of scraped soil from trenchbed. Figure 8.12 shows that the stronger/denser backfill results in a lower resistance mobilized by pipeline-trenchbed interaction, and a higher resistance developed by pipeline-backfill interaction. As observed in Figure 8.12a and will be shown in later sections through investigation of the internal soil deformation and failure mechanisms, when the pipeline penetrates into the trench wall, the wall collapsed into the trench. The backfilling material provides a passive pressure against the failing trench wall. The softer backfill produces a lower passive pressure resulting in a more significant wall failure and less ultimate soil resistance. Inversely, in the case of stronger backfill, the passive pressure against the wall collapse is increased and mitigates the failure extension and results in large lateral soil resistance.

8.4.4 Pipeline surface roughness

The pipeline surface roughness could have a significant effect on pipe-soil interaction, the failure mechanism, and the resultant lateral soil resistance. This important effect was investigated by comparing different case studies with a rough surface, smooth surface, and penalty friction (T2P1-1 and 2). The results were also compared with a test result (T2P1) published by Paulin (1998). Figure 8.13 shows that the numerical simulations with rough and smooth pipes well correlate with centrifuge test results. As the surface roughness is increased, the lateral soil resistance is also increased both for small displacements inside the trench and for the large displacements penetrating g into the trench wall.

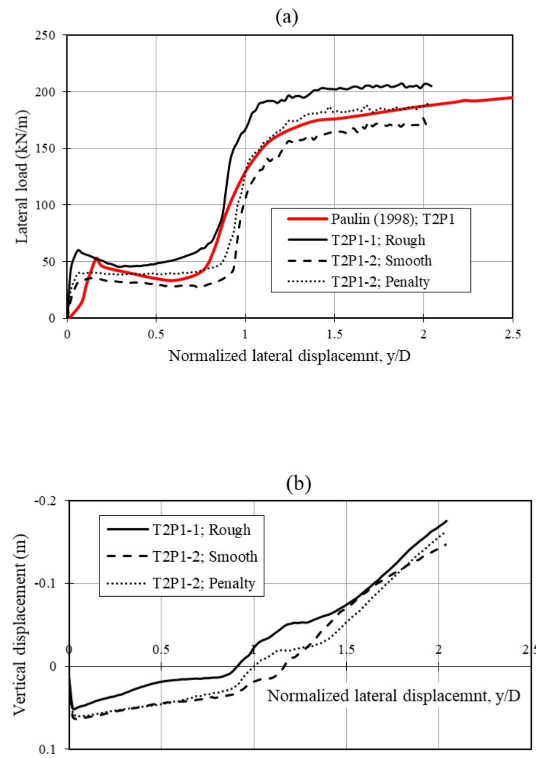


Figure 8.13. The pipe surface roughness effect on (a) load-displacement (b) pipe trajectory

To better interpret the results obtained in Figure 8.13, the shear bands and failure mechanism (velocity vectors) were produced for these case studies (T2P1-1 and 2) and compared in Figure 8.14.

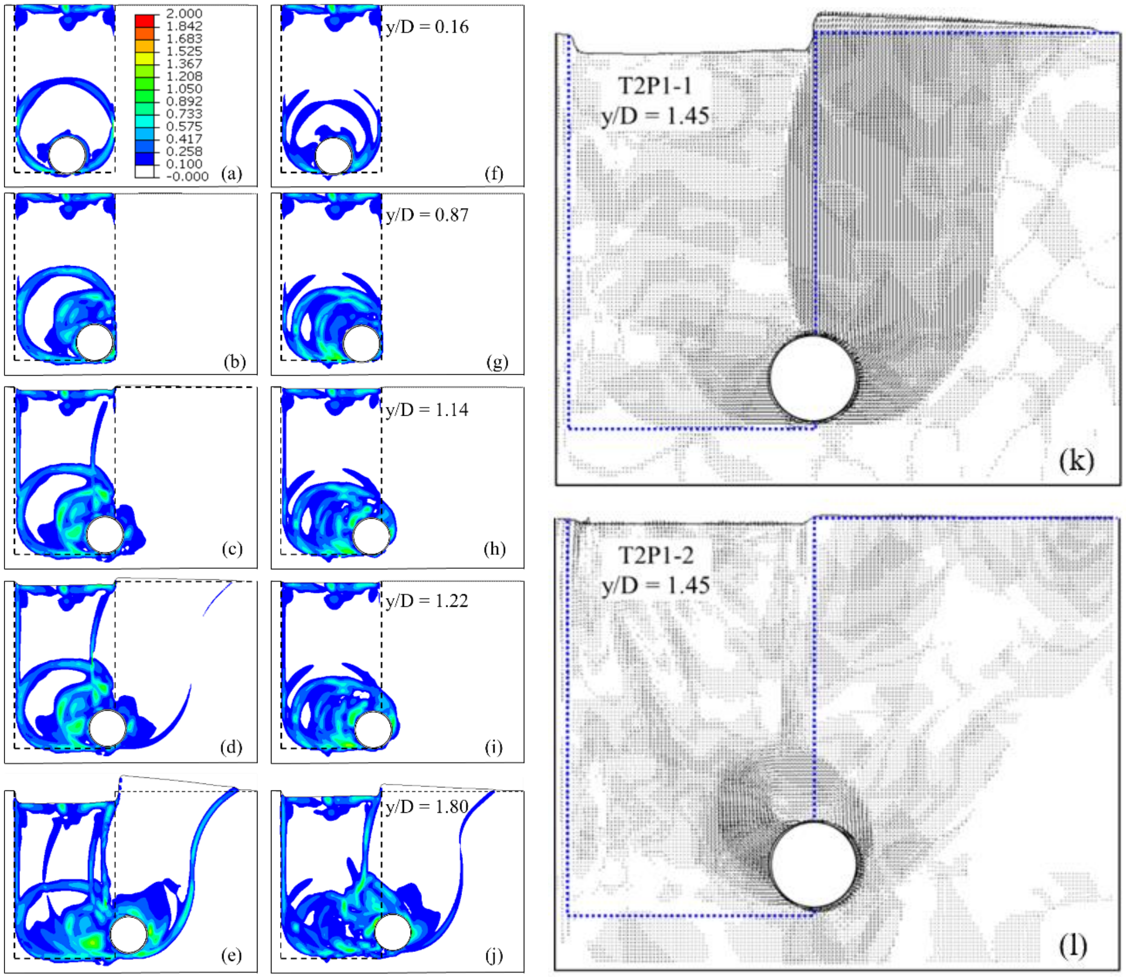


Figure 8.14. Development of shear surfaces in T2P1-1 with a rough surface (a-e) and T2P1-2 with a smooth surface (f-j), and a sample of failure mechanism in both (k and l)

Figure 8.14 shows that while the pipe moves inside the trench and only interacts with backfilling material, a rotational flow occurs (Figure 8.14a and f). This flow of backfilling material interacts with the trench boundaries as pipe move forward. The backfill flows from a high-pressure zone in front of the pipe to the low-pressure zone behind the pipe with minimal disturbance or heaves on the soil surface. The full circle of backfill flow is comparable with the shallow burial (Figure 8.14a-c). As the pipe interacts with the trench wall, the upper part of the trench wall starts to gradually slide towards the trench (Figure 8.14c and h). This mechanism particularly occurs in the pipe with a smooth surface, where the soil slides back into the trench and over the pipe without friction. The pipe with rough surface heavily interacts with the soil and pushes the soil ahead causing the earlier formation of the global shear band (Figure 8.14d). This causes a larger heave at the seabed surface in the pipe with a rough surface (Figure 8.14e). The enlarged view of the failure mechanisms using velocity field vectors in Figure 8.14k and l shows that overall the rough pipe surface results in a global shear failure mechanism in the trench wall and a local shear flow in the pipe with the smooth pipe. This, in turn, resulted in a higher lateral soil resistance in the case of pipe with a rough surface.

8.5 Conclusions

The lateral p-y response of the trenched/backfilled pipelines subjected to large lateral displacements were numerically investigated and compared with published test results. Large-deformation finite element analysis was conducted by using an advanced CEL model in Abaqus. The performance of the numerical model was verified against the published centrifuge test results in terms of the failure mechanisms, pipe trajectory, and load-displacement response. A comprehensive parametric study was conducted investigating the effect of several key parameters including pipe roughness, pipe weight, pipe initial embedment into the trench-bed, backfill strength properties, soil strain-softening, native soil tension cut-off, and burial depth on failure mechanisms and lateral soil resistance. The key conclusions can be summarized as follows:

- The developed numerical model was found to be a strong tool to simulate the complex pipeline-backfill-trench interaction problem. However, the model is only able to simulate the undrained conditions, since the CEL module of Abaqus is not able to implicitly model the pore pressure and soil matrix interaction through coupled analysis. Other LDFE methods such as RITSS might be adopted to simulate the coupled analysis of the partially or fully drained condition.
- The strain-softening reduces the volume of the soil mobilized against the moving pipe and lowers the lateral soil resistance both for the pipe with the small displacements inside the trench and large displacement penetrating to the trench wall.
- The pipeline initial embedment into the trenchbed was found to have a significant effect on the lateral p-y response of the pipeline in small displacements. This parameter showed a minimal effect on ultimate soil resistance in large displacements. The initial embedment is commonly neglected in analysis and design. Further research works are still required to equate the effect of bed interaction on failure mechanisms and lateral soil resistance.
- The pipe weight has a primary and secondary effect on lateral soil resistance. The primary effect appears in initial embedment and the secondary effect is observed in the pipeline trajectory. The heavier the pipe, the higher the soil resistance both for small and large displacements.
- The pipeline surface roughness showed a significant influence on the lateral soil resistance. The rough surface develops an intense interaction with the surrounding soil

and increases the soil resistance. The failure mechanism in the trench wall is global shears failure for the pipe with a rough surface and local flow for the smooth pipeline.

- The backfilling strength has a significant effect on lateral soil resistance in three different ways. First, increasing the pipeline-trenchbed interaction intensity; second, increasing the soil resistance for the pipe moving inside the trench; and third, by increasing the ultimate lateral soil resistance through producing a passive pressure against the collapsing trench wall and mobilization of a large volume of soil in front of the moving pipe.

8.6 Acknowledgments

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CHAPTER 9. Conclusions and Recommendations

9.1 Summary

The safe transportation of offshore hydrocarbons is a critical element of the oil producer's economy. The eastern coast of Canada produces a large volume of hydrocarbons that are being transported primarily via subsea pipelines buried in seabed. A better understanding of the pipe-soil interaction is necessary for safe design of buried pipelines against the geo-hazards and external threats. An accurate estimation of the resistance developed in lateral pipe-soil interaction would contribute to the maintenance and improvement of the integrity and safety of existing pipelines, and reducing the capital costs of new pipelines

An integrated research framework including centrifuge modeling, and numerical simulation was used to develop and validate numerical/analytical/empirical models in the context of pipeline design against geo-hazards leading to large lateral deformation. This research has been written in the form of several research papers including five (5) journal manuscript (chapter 4 to chapter 8) and three (3) conference papers (appendix A to appendix c).

The significant advantage of the current test set up compared to the earlier studies was the transparent sidewall that enabled direct observation of failure mechanisms using Particle Image Velocimetry (PIV) techniques. PIV were used to measure the soil deformations and calculate the shear strains. Shear strain localization of the soil into shear bands was also observed. Given the lack of direct observations of failure mechanisms and soil deformations during large lateral displacement of the pipelines, prudent action was taken in this study to build confidence in validating the numerical models in terms of development of lateral loading and failure mechanisms in partially drained conditions.

The numerical study addressed this large-deformation problem by developing an advanced numerical model using CEL method in Abaqus and conducting a comprehensive parametric study. The performance of the numerical model is verified against the published centrifuges tests in terms of the failure mechanisms, pipe trajectory, and load-displacement response in shallow and deep burial depths.

9.2 Conclusions

- The results presented in this thesis adds value to the existing knowledge base concerning the lateral response of the trenched pipelines. It provides data on the force-displacement response of pipe, pipe invert trajectory, and soil failure mechanism that occurs in horizontal pipe-soil interaction of the trenched pipelines.
- Regardless of the backfill stiffness, the presence of the trench results in less ultimate lateral soil resistance against the pipe when approaching/penetrating the trench wall. This resistance reduction is due to the progressive collapse of the trench wall into the backfill. The magnitude of the reduction depends on the stiffness of the backfill and the amount of passive lateral pressure against the active trench collapse. Further investigation is needed to propose a model that incorporates the effect of backfill stiffness and buoyancy on the magnitude of resistance reduction.

- The lateral soil resistance in the backfill is not dominantly governed by backfill material. The pipeline-trench bed interaction, including the magnitude of the initial pipe embedment into the trench bed and the lateral failure mode of the partially embedded pipe (immediate or slow breakout), makes a significant contribution to the lateral soil resistance. The backfill stiffness and its passive downward pressure against the developing soil berms in front of the pipe can have a significant impact on pipe-bed interaction and consequently on the ultimate lateral soil resistance. This important aspect has not been properly investigated in the past and needs further comprehensive investigations.
- The pipe-bed interaction and the squeezing of the trench bed material into the trench corner cause the pipe to move upwards, and approach/penetrate the trench wall in an oblique direction ($\alpha°$ from the horizon). The magnitude of α and consequently, the laterally mobilized soil wedge in the trench wall depends on several aspects of pipe–trenchbed interaction. Different mechanisms were observed in shallow and deep trenches at the stage when the pipe approaches the trench wall. The complex pipe–backfill–trench interaction at this stage needs further examination to propose possible solutions.
- A spiral contracting mechanism was observed around the moving pipe inside the backfill that interacts with the trench wall and the bottom in a complex manner. This interaction turns the pipe into a virtual pipe moving inside the soil. The size of this virtual pipe is larger than the real pipe when the pipe is approaching the trench wall and is less than the real pipe when the pipe penetrates into the trench wall.

Proposing models to quantitatively predict these mechanisms warrants further study.

- The observations shortly referred above denote the potential influence of several parameters on lateral soil resistance against the largely displaced pipeline that have not previously been addressed or investigated. Some of these parameters include pipe weight, pipe type, backfill properties, backfill buoyancy, trenching and backfilling methodology, construction procedure, construction season, operational loads, thaw settlement and permafrost, longitudinal seabed profile, etc. A close examination into each of these parameters opens up new research avenues that considerably influence the ultimate lateral soil resistance against the large displacement of the pipelines.
- Pipeline–trenchbed interaction significantly contributes to the lateral soil resistance against the pipe moving inside the trench, and also the ultimate soil resistance against the pipe penetrating into the trench wall.
- More severe pipeline–trenchbed interaction results in an earlier shear band formation in the trench wall, arriving the failure surfaces to the ground surface, and consequently less magnitude of ultimate soil resistance. The shear band initiation in the trench wall is delayed by reducing the intensity of pipeline–trenchbed interaction. This, in turn, results in a series of premature failure surfaces that never arrive at the ground surface and accumulate a larger amount of lateral soil resistance against the pipe penetrating into the trench wall.

- The intensity of the pipeline–trenchbed interaction is governed by the pipeline moving direction and the initial embedment into the trench-bed. These two parameters can be affected by several aspects such as pipe weight, pipe type, pipe laying method, construction season, construction procedure, trenching and backfilling methodologies, longitudinal ground profile, seabed soil properties, environmental loads, operational loads, etc. Therefore, an accurate prediction of the lateral soil resistance needs to account for the project-specific conditions.

- The developed numerical model in chapter 8 was found to be a strong tool to simulate the complex pipeline-backfill-trench interaction problem. However, the model is only able to simulate the undrained conditions, since the CEL module of Abaqus is not able to implicitly model the pore pressure and soil matrix interaction through coupled analysis. Other LDFE methods such as RITSS might be adopted to simulate the coupled analysis of the partially or fully drained condition.
- Considering the effect of strain-softening in the numerical simulations reduces the volume of the soil mobilized against the moving pipe and lowers the lateral soil resistance both for the pipe with the small displacements inside the trench and large displacement penetrating to the trench wall.
- The pipeline initial embedment into the trenchbed was found to have a significant effect on the lateral p-y response of the pipeline in small displacements. This parameter showed a minimal effect on ultimate soil resistance in large displacements. The initial embedment is commonly neglected in analysis and

design. Further research works are still required to equate the effect of bed interaction on failure mechanisms and lateral soil resistance.

- The pipe weight has a primary and secondary effect on lateral soil resistance. The primary effect appears in initial embedment and the secondary effect is observed in the pipeline trajectory. The heavier the pipe, the higher the soil resistance both for small and large displacements.
- The pipeline surface roughness showed a significant influence on the lateral soil resistance. The rough surface develops an intense interaction with the surrounding soil and increases the soil resistance. The failure mechanism in the trench wall is global shears failure for the pipe with a rough surface and local flow for the smooth pipeline.
- The backfilling strength has a significant effect on lateral soil resistance in three different ways. First, increasing the pipeline–trenchbed interaction intensity; second, increasing the soil resistance for the pipe moving inside the trench; and third, by increasing the ultimate lateral soil resistance through producing a passive pressure against the collapsing trench wall and mobilization of a large volume of soil in front of the moving pipe.

9.3 Recommendations for future studies

Following items are recommended to be considered in future studies:

- Further centrifuge tests are recommended to investigate the failure mechanisms both in drained and undrained conditions.

- The important observations of the current study such as pipeline-trenchbed interaction intensity due to various constructional and operational parameters need to be investigated through a larger series of centrifuge tests. This is also necessary for quantitative studies targeted to come up with new solutions for estimation of the trenching/backfilling effect on lateral soil resistance of subsea pipelines.
- The developed numerical model in chapter 8 was found to be a strong tool to simulate the complex pipeline-backfill-trench interaction problem. However, the model was not able to properly show the collapse of trench wall into the trench. Development of such numerical model to show the trench wall instabilities would result in better estimation of the mobilized loads on the pipeline.
- The developed numerical model based on CEL method was not able to implicitly model the pore pressure and soil matrix interaction through coupled analysis. Other LDFE methods such as RITSS might be adopted to simulate the coupled analysis of the partially or fully drained condition.

Appendix A

Lateral Response of Trenched Pipelines to Large Deformations in Clay

This paper has been published and presented in Offshore Technology Conference 2018, held in Houston, Texas, USA on 30 April–3 May.

OTC-28842-MS

Lateral Response of Trenched Pipelines to Large Deformations in Clay

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This paper was prepared for presentation at the Offshore Technology Conference held in Houston, Texas, USA, 1–4 May 2018.

Abstract

Subsea pipelines are usually buried in shallow waters for physical protection. Buried pipelines may experience large lateral displacement in different occasions such as ice gouging, ground movement, significant thermal gradients, and dragging by anchors, fish traps, etc. Backfilling materials are often heavily remoulded under functional and environmental loads and are considerably softer than trenched native ground. This, in turn, affects the failure mechanisms in the surrounding soil and the lateral load-displacement response of the pipeline, consequently. These important considerations are covered less often in the design codes and standards. In this study, the lateral pipeline-backfill-trench interaction was studied through centrifuge testing of sixteen distinct pipe-soil configurations under drained and partially drained conditions. A transparent observation window combined with digital cameras were used for Particle Image Velocimetry (PIV) analysis. A range of instruments was installed on the pipeline, backfill, and the trench to obtain the key data and the lateral p-y response of the buried pipe. The influence of several key parameters on the lateral pipeline response was also investigated including backfilling properties, trench geometry, interaction rate effect, and burial depth. The results showed that the failure mechanisms, affected by various pipeline-backfill-trench interaction parameters, have a significant impact on the lateral p-y response and the ultimate soil

resistance. The study program provided an in-depth insight into this challenging area and prepared the ground for proposing new models and methodologies for incorporating more realistic conditions for pipeline design subjected to large lateral displacements.

Introduction

Subsea pipelines may be buried inside excavated trenches in cohesive soils for protection against external and internal loads. Both trenching and laying the pipeline may take place at the same time, or at different times, depending on the construction methodology. Using the dredged material for simultaneous or delayed backfilling of the pipeline is an economical solution and commonly performed in practice. Depending on the trenching/dredging methodology, construction strategy, and environmental loads, the backfilling material may undergo different degrees of remoulding leading to different geomechanical properties. This, in turn, affects the failure mechanisms and the pipeline's response to large lateral displacements that may be caused by ground movement, faults, slope instabilities, ice gouging, etc. In other words, this relative displacement between the pipeline and surrounding soil exerts forces on pipelines. The magnitude of these forces and the force-displacement response of the pipeline to large lateral deformations depend on several parameters including the submerged weight of the mobilized backfilling and native soil, the horizontal component of shearing resistance offered by the soil, and the suction behind the pipe. These parameters, in turn, depend on the geomechanical properties of the backfill, native soil, trench geometry, burial depth, confining pressure, pipeline roughness, pipeline size, loading rate (drained/undrained), soil stress history, the backfill extent of consolidation, and the over-consolidation ratio of native soil (OCR).

In practice, the structural response of the pipeline is generally analyzed by defining the force-displacement relationship for a set of independent springs (e.g., ALA 2005), where the behaviour of the springs is expressed by bilinear or hyperbolic functions (PRCI 2009; ALA 2005). However, large discrepancies are observed in the recommendations provided by different design codes and the existing empirical equations (Trautmann and O'Rourke 1985; Paulin 1998; ALA 2005; PRCI 2009; Rajah et al. 2014; Pike 2016). The main sources of observed discrepancies are simplified assumptions in determining the values of key parameters which rarely consider the effects of pipeline-backfill-trench interaction and the inherent differences in the framework of the conducted studies. In addition, the models proposed for the prediction of the lateral pipeline response in clay usually use the undrained shear strength in the analysis, which may not be appropriate for lower rating loads. In general, there is a lack of information about the actual lateral force-displacement response of pipelines in clay. Therefore, an accurate prediction of the pipeline force-displacement curve within large deformations requires an in-depth investigation of the progressive failure mechanisms around the pipeline considering the pipeline-backfill-trench interaction effects.

In this study, a comprehensive centrifuge testing program was conducted to investigate the response of buried pipelines to large lateral displacements. The interactive and progressive failure mechanisms both in the backfilling and the native soil were obtained through direct observation from a transparent acrylic sheet mounted in the sidewall of the test box. A range of tests was conducted using a fully stocked test instrument setup to capture the influence of various parameters including the undrained shear strength of the backfill and the native soil, trench geometry, burial depth and loading rate (drained/undrained). This paper describes the experimental test setup and a summary of the initial test results. Further post-processing of the results is still ongoing and will be published shortly.

Previous experimental studies in clay

In the literature, most experimental pipeline studies were conducted in sand. There is a very limited number of pipeline-specific theoretical and experimental models in the literature to predict the ultimate lateral resistance or force-displacement (p-y) curves for pipelines in clay. Many of the proposed models are based on anchor plates because they share behavioural characteristics with pipelines (Mackenzie 1955, Tschebotarioff 1973, Luscher et al. 1979, Rowe and Davis 1982, Das et al. 1985, Das et al. 1987, Rizkalla et al. 1992, Ranjani et al. 1993, Merifield et al. 2001). Many of the other solutions are developed based on piles (Hansen (1948), Poulos (1995), Hansen and Christensen (1961), Matlock (1970), Reese and Welch (1975), Bhushan et al. (1979), Edgers and Karlsrud (1982), ALA 2001, Klar and Randolph 2008). There is a limited number of models based on pipelines' lateral interactions (Oliveira et al. 2010, Poorooshasb et al. 1994, Paulin 1998).

Paulin (1998) conducted a series of centrifuge tests on lateral pipeline-soil interactions in clay to study the effects of trench width, burial depth, interaction rate, backfill properties, and stress history of the soil on force-displacement curves. The study was one of the first small-scale comprehensive study on the lateral response of fully buried pipelines in clay incorporating the effects of backfill material and the trench. The author used four aluminum pipes with instruments with a diameter of 19 mm and a length of 250 mm corresponding to a prototype pipeline with a diameter of 0.95 m and a length of 12.5 m (1:50 scale). A mixture of kaolin clay and Sil-Co-Sil silt (50%-50%) was used as a test bed with about 40 kPa undrained shear strength after consolidation. Actuators pulled the pipe horizontally with different velocities to obtain the lateral p-y responses. The authors observed that the trench width had little or no effect on an undrained interaction, while the undrained load on the pipeline increased with an increasing burial depth. The pipeline displacement rate (or drainage conditions) was found to have a significant effect on the loads transferred to the pipeline by the soil. The authors concluded that the undrained p-y response and ultimate loads could be predicted within ±20% using existing methods of analysis. The author tried to monitor the failure mechanisms using strands of painted spaghetti. This technique provided some qualitative information about the failure mechanisms, but lack of direct visualization made it less reliable. The author noted that the backfill properties can affect the overall normalized interaction between the pipeline and the soil. However, they could not determine if this is due to a change in failure mechanisms or a change in the separation conditions behind the pipeline. Paulin (1998) highlighted the need for further research to increase the size of the existing database to reduce scatter in the experimental data. This could result in an improvement in the existing analytical methods. It was also recommended that the effects of internal pressure, pipeline end conditions, and the backfill properties be further investigated.

The current research program was conducted to overcome the shortcomings of the project performed by Paulin (1998). A full range of state-of-the-art instruments was applied on the pipeline, backfill, native soil, actuation system, and the whole test setup. The progressive and interactive failure mechanisms were explicitly obtained by using a transparent acrylic sheet, digital cameras, and particle image velocimetry (PIV). Overall, the project significantly improved the understanding of the lateral response of fully buried pipelines to large deformations and provided an excellent insight into this challenging problem.

Moreover, the undrained shear strength parameter is commonly used in design practice to assess the pipe-soil interactions. This approach neglects the rate dependency of the pipeline response. Drained or partially drained conditions are quite common in real pipe-soil interaction events, where the relative displacement rate between the pipe and the soil is very slow. In such occasions, the soil surrounding the pipeline is consolidated to some extent during the displacement. Also, in many geographical locations, silt fractions are found in natural offshore soft clays (e.g., Gulf of Mexico, Schiffman 1982). The presence of silt in clay tends the consolidation characteristics of clay towards partially drained and even fully drained conditions. Other compositional and depositional fractions may also show a similar effect. The drained response of the pipeline to large deformations in clay has been less frequently explored (Paulin 1998). The current study focused more on the partially drained and drained responses of the pipeline throughout large lateral displacements to investigate the rate dependency of the pipeline response.

Testing program

The testing program comprised five series of tests involving the lateral pipeline-backfill-trench interactions in clay during large lateral displacements (up to 4D) at a centrifuge acceleration of 19.1g. Two similar pieces of pipe with different configurations were pulled in opposite directions and tested in each run resulting in ten tests in total. In addition, three series of tests (six pipe tests) were conducted in dry loose sand. However, the current paper only discusses the tests conducted in clay. The details of the interactive failure mechanisms were directly monitored from a transparent observation window mounted on the side of the test box. Two digital cameras were used to capture high-quality images for post-processing and particle image velocimetry (PIV) analysis. In each clay test, the fully instrumented model pipe sections were located on the bottom of the excavated trenches and backfilled with different backfilling materials. Two vertical actuators with pulleys and horizontal cables were used to pull the pipes in the opposite directions with pre-determined velocities, while the pipes were free to move vertically over the initial course of displacement.
The main objectives of the testing program included:

- Observation of failure mechanisms in the backfill and trench wall;
- Obtaining the lateral p-y curve and ultimate resistance for partially drained and drained conditions;
- Determining the pipeline-backfill-trench interaction characteristics;
- Assessing the influence of trench geometry (i.e., depth, width, and side angles), backfilling properties, interaction rate, soil stress history, and suction force

mobilization;

- Developing analytical models for the lateral p-y curves and ultimate soil resistances
- Evaluation and improvement of the current practice for lateral pipeline-soil interactions

The current paper focuses on an overview of the test set up, instrumentation, monitoring and the initial results obtained from the testing program in clay. Further post-processing of the data is still ongoing and the results will be published accordingly. Samples of failure mechanisms and corresponding PIV analysis are also provided. The testing schedule was defined to maximize the amount of required high-quality data obtained. Table A.1 gives a summary of the conducted testing program.

Table A.1. Summary of the conducted testing program

Test	Pipe	Test name	Model burial depth (mm)	Burial depth ratio, H/D	Trench backfill type	Trench wall	Model displacement rate (μm/s)	Normalized velocity vD/c_v	Normalized pulling distance
Test 1	Pipe 1	T1P1	104.5	3.29	Chunk	Inclined (32°)	8.96	0.407	2.61
	Pipe 2	T1P2	102.1	3.22	Slurry	Vertical	9.09	0.412	3.03
Test 2	Pipe 1	T2P1	115.1	3.63	Loose sand	Vertical	9.29	0.422	3.6
	Pipe 2	T2P2	112.5	3.54	Slurry	Inclined (62°)	9.16	0.416	3.5
Test 3	Pipe 1	T3P1	49.0	1.54	Slurry	Vertical	9.44	0.428	3.93
	Pipe 2	T3P2	52.2	1.64	Chunk	Inclined (36°)	9.23	0.419	3.82
Test 4	Pipe 1	T4P1	45.9	1.44	Slurry	Vertical	3	0.136	3.93
	Pipe 2	T4P2	48.8	1.54	Chunk	Inclined (32°)	3.01	0.136	3.87
Test 5	Pipe 1	T5P1	116.0	3.65	Slurry	Vertical	2.98	0.135	3.71
	Pipe 2	T5P2	121.6	3.83	Chunk	Inclined (38°)	3.01	0.137	3.85

Experimental setup and testing procedure

Modelling considerations

The main objective of the testing program was to investigate the pipeline-backfill-trench interactions and its impact on the force-displacement responses of pipelines during large lateral deformations. For this purpose, it was essential to monitor the interactive and progressive soil failure mechanisms around the pipe and interpret its impact on the measured p-y responses and the ultimate loads exerted on the pipelines. Therefore, a plane-strain container with an Acrylic side window was used to monitor the failure mechanisms for further PIV analyses. The effects of variations in burial depth, trench geometry, interaction rate, and backfill properties were other objectives of this study to ensure that the results could be confidently scaled up to full-scale conditions. Fig. 1 shows a sample schematic view of the test setup, where two pieces of model pipes were backfilled inside excavated trenches in a pre-consolidated soil bed and pulled apart over large displacements (3-4D) using horizontal cables driven by vertical actuators. Fig. 1 illustrates the boundary conditions normalized to the pipe diameter using dotted circles.

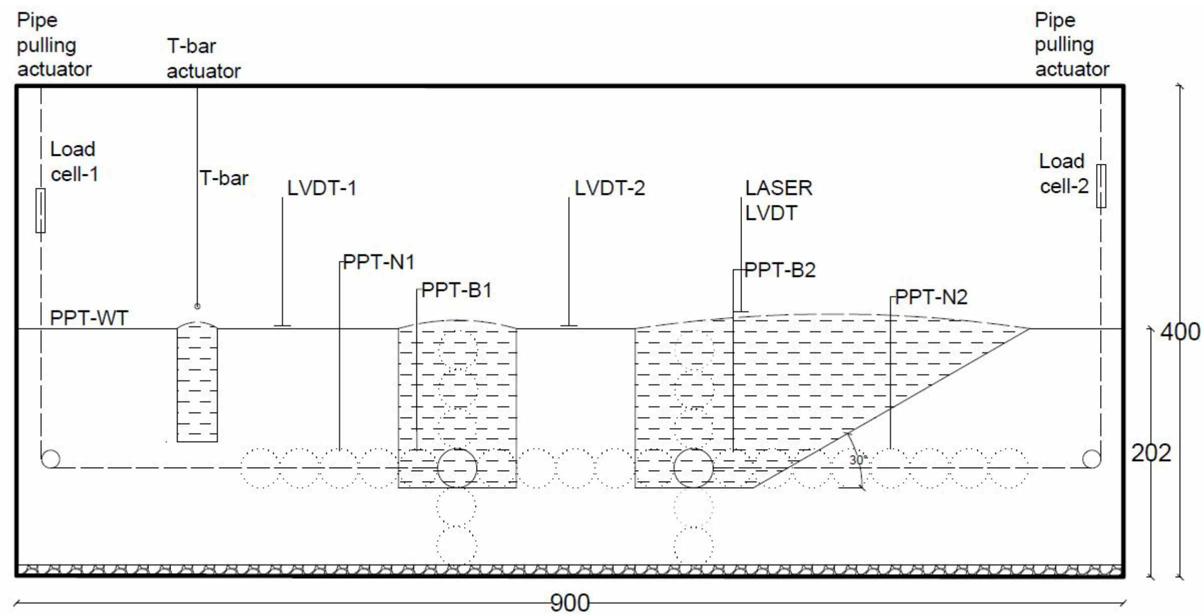

Fig. 1. Schematic view of test setup (cohesive test bed); Instruments are coded; all dimensions are in mm

The soil sample was consolidated to an effective stress of 400 kPa and was unloaded sequentially. This level of consolidation yielded soft clay with undrained shear strength profile in native soil (15-25 kPa). Three main types of backfill with various geomechanical properties were developed to model the significant difference between the strength of the native material and the backfill. The model pipe size was dictated by the dimensions of the internal pore pressure transducers that had to be incorporated inside the pipe to measure the pipe-soil interface pressure or suction in the rear of the pipe during pipeline displacement. The minimum possible bending radius of the cable connected to the pressure transducer imposed a minimum nominal pipe diameter of 32 mm to accommodate the transducer. The acceleration level was set to about 19.1g to model a real pipe diameter 610 mm, as targeted by the industry sponsor. This pipe size was the same as the earlier tests conducted in sand (Burnett 2015), representing the size range of export pipelines. Different burial ratios (H/D) ranging from 1.4 to 3.8 were tested to ensure covering shallow to deep burial conditions. Rectangular and trapezoidal trenches were considered with a fixed bottom width of 3D and top with varying from 3D to 10D depending on side angle of the trench wall (90°, 60°, and 30°). The trench wall behind the pipe was kept vertical, assuming a minor effect on the lateral pipe response moving in opposite directions.

The effects of the interaction rate has rarely been considered in developing existing prediction models (Paulin 1998). In reality, depending on the nature of the interaction, the pipeline displacement rate could range from millimeters per year (drained loading) to meters per second (undrained loading). This was investigated in the current testing program by performing partially drained and somewhat drained (not a perfect drained) tests. A range of instruments was used to monitor the testing program, such as pore pressure transducers (PPTs), strain gauges, load cells, linear variable differential transformer (LVDTs), T-bar, actuators and vertical drive motion controllers, digital cameras, markers and artificial textures.

Soil preparation

Different procedures were used to prepare the native soil bed and various backfilling materials to simulate realistic field conditions. A mixture by weight of 50% white kaolin clay and 50% Sil-Co-Sil silt was added by sufficient amount of water to form a slurry with a nominal moisture content of 70%. The mix was left for an hour or some to completely soak before mixing for about a half-hour followed by 3 hours mixing under a vacuum of 60-70kPa for de-airing. The mixture was poured into the container, closely observing to ensure it is homogeneous and free of lumps. The container was placed in the consolidometer and the top edge was checked and leveled to be horizontal. Incremental loads were applied to soil over a week or so and directly monitored by load cell of a hydraulic jack.

After achieving the desired stress level (400 kPa), the soil sample was sequentially unloaded up to 100 kPa with open drainage valve. Below 100 kPa, the flow of water into the sample was restricted by closing the base drain and removing excess water at the soil surface. After removing the box from consolidometer, the removable side wall of the box was removed by sliding parallel to the opposite side wall. Before installing the transparent window, the exposed side surface of the soil sample was artificially seeded by dark Frasier river sand using a regular salt pourer. This texture provided by artificial seeding allow both macroscopic and grain-scale deformation features to be identified by PIV analysis (Stanier and White 2013). The Acrylic sheet was carefully installed on the side of the box with a face-to-face approaching direction.

Trenching the soil bed

Shaving blades with desired side angles were used to cut the trenches and T-bar site. Shaving blades were attached to an adjustable shaft traveling inside a horizontal guide frame mounted on the top edge of the box. Samples were extracted from shaved material to determine the average water content. The height of the shaving arm was adjusted to ensure that the spring line of the pipe will be at the desired elevation from the prepared bottom of the testing box. To locate the pulling cables, 3 mm wide openings were created using narrow steel blades. The desired dimensions of the trenches were controlled by using marks on the internal surface of the steel rear wall and direct measurements through the transparent front wall. Fig. 3 and Fig. 4 show sample of excavated soil bed, where trenches with vertical and inclined walls have been tested. The trench depth was kept same for both of the pipes in a test. Trenches with three different side angles were created (i.e., 30°, 60° and 90°). To better simulate the real condition, the surfaces of the trench walls and trench bottom was slightly patterned using a wet canvas to prevent having a slippery smooth surface between the trench and backfill.

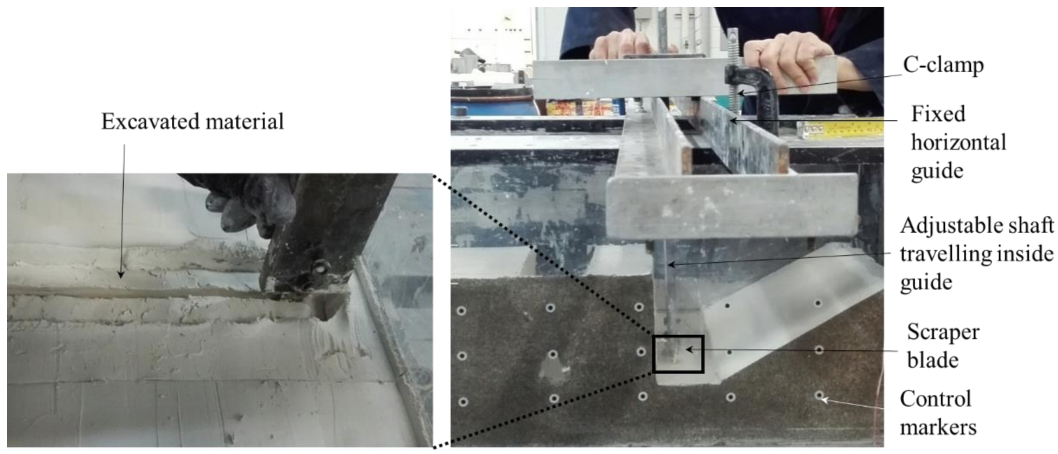

Fig. 2. Excavating trench bottom using blade

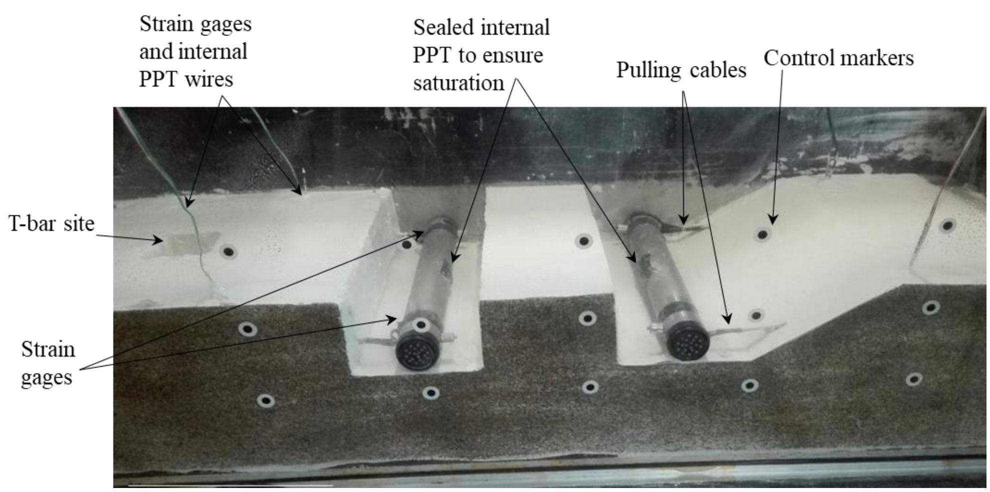

Fig. 3. Box front view; Pipes installed inside two excavated trenches before backfilling

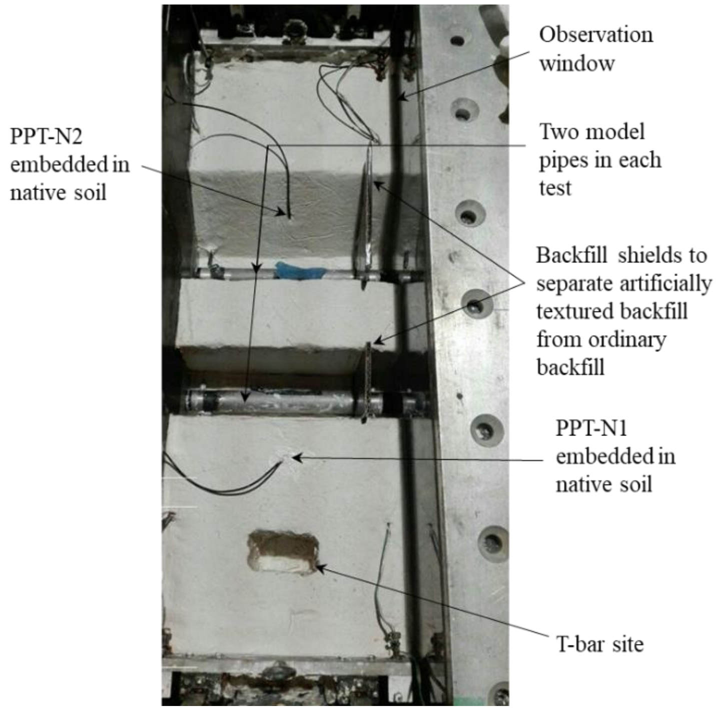

Fig. 4. Top view of instrumented box before backfilling

Backfilling material

The dredged material is usually used for backfilling the trenched pipeline. Depending on trenching and backfilling technique, and construction condition, the backfilling material may be remoulded to a different extent. Various backfilling material properties are expected depending on many parameters such as level of soil disturbance, size of clay lumps, potential high energy environment, whether the excavated spoil is left on the seabed or stored on land or barge, the period of exposure before placing in the trench, consolidation time after placing inside the trench and etc. In this study, in addition to silica sand, a range of cohesive backfills were reproduced from a shaved native material including very soft slurry and chunk materials with various strength. Different preparation methods were used to model a range of backfilling conditions and backfill properties. This enabled preparation of fairly soft backfills representing the strength difference between the real native soil and backfill material. Table A.2 shows the summary of the backfilling material prepared and tested in this study.

Table A.2. Soil properties of cohesive testbed

Test	Pipe	Test name	Trench backfill type	Trench backfill ID	T-bar site backfill	T-bar site backfill c_u (kPa)	Native c_u at pipe depth (kPa)	Native soil water content before test (%)	Native water content after test (%)	Native soil void ratio	Saturated unit weight Υ_{sat} (kN/m^3)
Test 1	Pipe 1	T1P1	Chunk	T1B1	Slurry	<< 1	16 - 19	32.04	32.97	0.864	18.33
	Pipe 2	T1P2	Slurry	T1B2							
Test 2	Pipe 1	T2P1	Loose sand	T2B1	Chunk	2 - 3.7	16 - 19.5	30.81	31.11	0.815	18.56
	Pipe 2	T2P2	Slurry	T2B2							
Test 3	Pipe 1	T3P1	Slurry	T3B1	NA	NA	17.5 - 20	31.24	31.47	0.825	18.51
	Pipe 2	T3P2	Chunk	T3B2							
Test 4	Pipe 1	T4P1	Slurry	T4B1	Slurry	<< 1	17.5 - 20	31.99	31.98	0.838	18.45
	Pipe 2	T4P2	Chunk	T4B2							
Test 5	Pipe 1	T5P1	Slurry	T5B1	Chunk	2.5 - 4.5	17 - 20.5	30.12	32.13	0.842	18.43
	Pipe 2	T5P2	Chunk	T5B2							

Slurry

To investigate the influence of different backfills on the pipeline response, a trenched but unburied base case was required. In reality, the trench may be naturally filled with fine sediments under the environmental loads action in the relatively shallow water, where seabed currents are sufficient to induce transport (Cathie et al. 2005). Also, the excavated material deposited into the spoil heaps and then left exposed to free water for a long period before backfilling causes the soil to become fluidized and produce a slurry. This kind of natural backfill is a soft slurry that has no or very low strength. A mixture of shaved native soil material and the water was used to create the backfilling slurry with water content about 100%, which is about three times the liquid limit of the native soil. The in-flight T-bar test showed almost zero undrained shear strength after inflight consolidation. However, the test results showed that despite low strength, the slurry contributes to the pipe-trench interaction to some extent (i.e., 5 kN/m for prototype-scale pipe with 610mm diameter). Fig. 5 shows a top view of the backfilled soil sample.

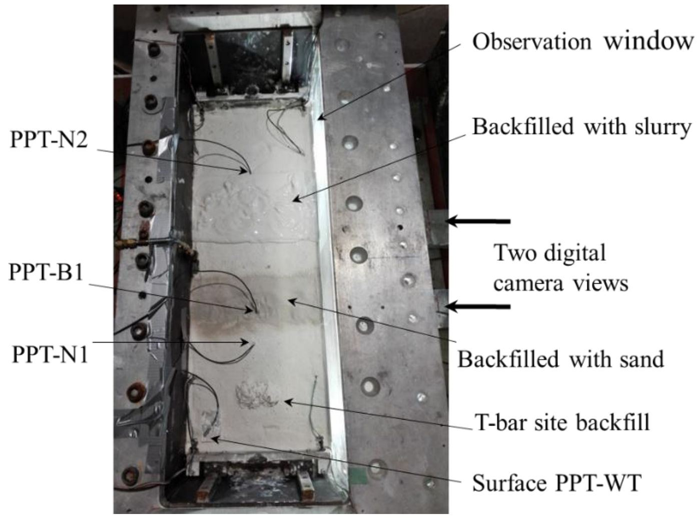

Fig. 5. Top view of the instrumented box after backfilling

Chunk of native soil

The chunks of around 25 mm were excavated from native soil and exposed to water for several hours. This backfill was heterogeneous and consisted of softened and remoulded or semi-remoulded chunks. The water content was kept slightly higher than the in-situ consolidated soil. The preparation process of this backfilling type can simulate the jet cuttings excavated and deposited inside the trench in a matrix of slurry while using the jetting technique. This backfill can also be taken as an attempt to model the backfills produced by mechanical excavation or backfilling techniques like plowing, backhoe and clamshell bucket. Four different chunky material with different stress history were produced and tested in this program.

Silica Sand

The granular purchased material may be used for backfilling of the pipelines in many cases. Fine Silica sand (D60 = 0.205 mm; D30 = 0.14 mm; D10 = 0.103 mm.) was used as backfilling material in one test (T2P1) to investigate the pipeline response surrounded by granular cohesionless materials. The silica sand was poured inside the trench after locating the pipe. The sand backfill achieved an extent of densification by water filling the test box and in-flight period for consolidating native soil.

A T-bar penetrometer (Stewart and Randolph 1994) was used to obtain the undrained shear strength profile of the native and backfilling material. A T-bar bearing factor of 10.5 was considered for deep penetrations. But for shallow depths, a reduced bearing factor arising from the soil buoyancy and shallow failure mechanism mobilized before the full flow of soil around the bar (White et al. 2010) was used to translate the measured bearing resistance to the undrained shear strength.

Instrumentation

The model pipe, backfilling and native soil was fully instrumented to ensure sufficient and reliable data will be recorded during the testing program. Table A.3 provides more detailed information about the test instrumentation.

Table A.3. Test instrumentation

Instrument name	Location	Description	Total number used per test
Internal PPT	Inside the pipe sensing the rear of pipe pore pressure	Druck PDCR81	1 per pipe
PPT holder, water plug and O-rings	Inside the pipe	Nylon	1 per pipe
Pore Pressure Transducer (PPT)	In backfill and native soil and at surface of soil	Druck PDCR81	2 per pipe
Strain gauge	At reduced section of pipe. One full Wheatstone bridge	Shear gauge which has been calibrated to shear force at reduced section of pipe	2 per pipe
Load cell	Connected to pulling cable measuring total pulling force including all frictions	3.5 kN capacity	1 per pipe
T-bar	T-bar site	Head bearing area: 30×7.4 mm^2	1 per test
Digital camera	In front of the viewing window	10.10 megapixel	1 per pipe
LVDT	Native soil surface	Linear Variable Displacement Transducer	2 per test
Laser LVDT	Backfill surface	There was malfunction because passing through water	1 per test
Control marker	Inner side of transparent window	Inner circle diameter: 6.27 mm; Outer diameter: 12.24 mm	18 per test
Sand for artificial seeding	Sprinkled on native soil and mixed with backfill just beside the window	Fraser River sand	NA
End caps & O-ring	The end of the pipes	Nylon	2 per pipe

Miniature pore pressure transducers (PPTs) were used to record the pore pressure variation in different spots of the test box. The internal PPT was installed inside the pipe facing the rear of the pipe to measure the suction force mobilization behind the pipe during the displacement. The curvature of the data acquisition cable connected to this PPT dictated the minimum diameter of the model pipe (i.e., 31.75 mm). Each backfill material equipped with one PPT and two more PPTs was installed in native soil with the locations shown in Fig. 1. The external PPTs were kept in position using supports on two I-beams carrying the actuators. These external PPTs were used to monitor the state of soil equilibrium assessing the soil drainage conditions under various pipeline displacement rates throughout the moving path. The external PPTs could be also used for monitoring the variation of the water table.

The strain gauges were installed in the reduced cross-section of the pipes to capture the lateral pipe response (Fig. 6). The strain gauges were calibrated to measure the shear force at the reduced sections. Calibration factors were extracted by simple analysis of load distribution along the pipe.
In addition to direct monitoring of surface variation of the soil surrounding the pipes via acrylic sheet, appropriate numbers of linear variable displacement transformers (LVDTs) were also used to measure the soil surface movement. The measuring shafts of the LVDTs rested on Plexiglas pads. These pads were penetrating into the slurry backfill with low strength, so laser LDVTs were replaced in the tests with slurry backfill. The clarity of the filled water inside the test box was not sufficient for traveling the laser beam and recording the surface movements.

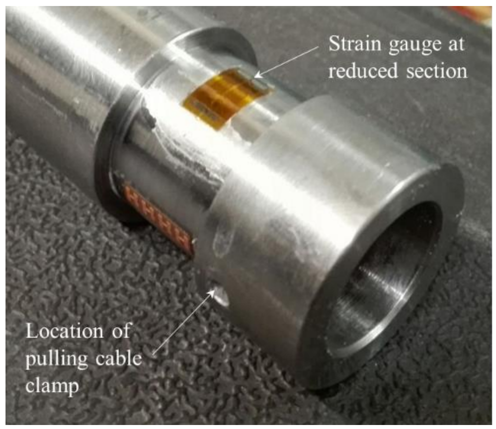

Fig. 6. Shear strain gauge installed at reduced section

Visualization and monitoring

Two Canon EOS DIGITAL Rebel XTi still cameras operating in continuous shooting mode were used to capture images of the moving pipes end cap and surrounding soil through the observation window. Each camera was intended for one pipe individually. Two cantilever beams fixed the cameras to the centrifuge swinging platform. Tight cables were used at the end of cantilever beams to secure the cameras at higher g-level.
Acrylic transparent window on one side of the test box enabled direct recording of soil failure mechanism, pipe trajectory, and lateral pipe response. The continuously captured high-quality images were used in particle image velocimetry (PIV) analysis to measure the displacements and obtain strains at any point observable from transparent window.
The PIV analysis was conducted using GeoPIV software originally developed by White et al. (2003) and further developed by Stanier et al. (2016), where the subsets of the image field were tracked and compared with the reference image as the pipes were being pulled. Black and white circle markers with the dimensions and layout shown in Fig. 3 were attached to the transparent window as the reference points in PIV analysis. Because of physical limitations in testing facilities and the actuators, the digital cameras couldn't be synchronized and moved with movement of the pipe. To limit the slight effect of varying

observation sight over the large lateral displacement in PIV analysis, a calibration sheet was used. This enabled the correction of image distortion because of noncoplanarity of the images and object planes, and the nonlinear fish-eye and barrelling effects. During the tests with model pipe nominal moving velocity of 0.01 and 0.003 mm/s, 25 and 83 second shutting intervals were used to capture images at 0.25 mm increments which is appropriate relative to total displacement domain and ensure sufficient capturing of the soil failure mechanisms.

Test results

This section of the paper reviews the force-displacement and pore pressure response obtained during the large lateral movement of the pipe. The sample results of the PIV analysis are also investigated to compare the observed failure mechanisms with existing solutions.

Force-displacement response

Prototype-scale force-displacement data is obtained by applying the appropriate scaling factors to model-scale data. In this testing program, it was observed that the lateral response of the pipeline could be significantly affected by several key parameters mainly including the strength and type of the backfilling material, burial depth, trench geometry, and interaction rate. All of these key parameters affect the failure mechanism and the pipeline response consequently. The post-processing of the test results is still ongoing. However, samples of the obtained results will be shortly discussed in coming sections.

Influence of backfilling material

In practice, the excavated soil is commonly used to backfill the trench. A wide range of backfill properties are expected depending on many parameters such as level of soil disturbance, size of clay lumps, potential high energy environment, whether the excavated spoil is left on the seabed or stored on land or barge, the period of exposure to seawater before placing in the trench, consolidation time after placing inside the trench and etc. This process results in weaker backfill in comparison with the native soil, which has been less explored in the literature. In this study, three major backfill types were investigated including the slurry, chunky material, and sand. The first two types of backfills were prepared using the native soil excavated material with different preparation process. Fig. 7 shows a sample of p-y responses obtained for different backfilling material.

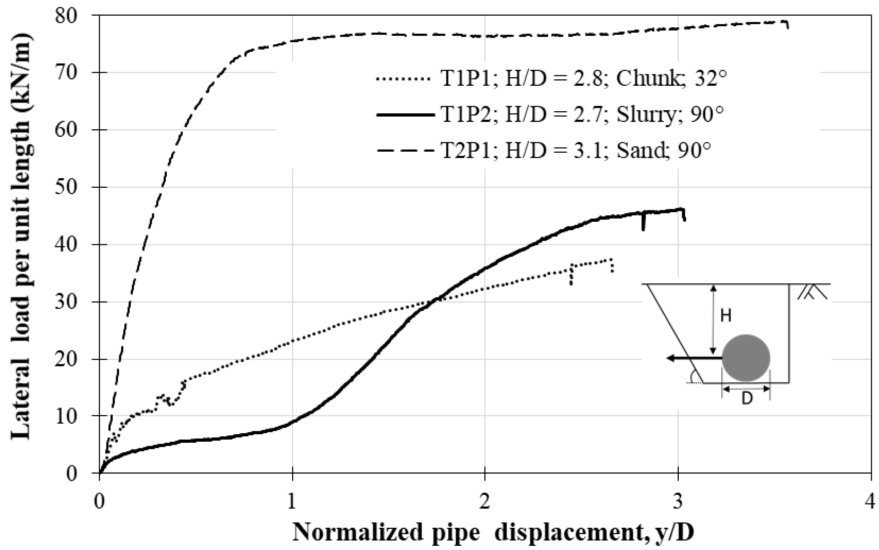

Fig. 7. Effect of backfill type on force-displacement response

As earlier shown in Fig. 1, the trench bottom width in all tests was three times the pipe diameter with the pipe section located in the centreline. The tests were conducted by a displacement-controlled approach with a constant displacement velocity. During the tests, the pipe is laterally displaced by 1D to arrive at the initial location of the trench wall. It is referred as an *initial location* because the pipe-backfill-trench interaction causes the trench wall deformation before having contact with the pipe. Depending on the side angle of the trench wall and the strength of backfill material the pipe begins to embed into the trench wall at different offsets from initial pipe position. Fig. 8 shows that in the case of a slurry backfill (base case) with extremely low strength, the pipe embedment into the trench wall occurs in 1D displacement with a very low magnitude of mobilized force before contact. This refers to no lateral deflection on the wall before pipe contact. The reason is the limited or no interaction of the slurry with the pipe and the trench wall regarding the material strength. The pipe response to lateral displacement in the sand backfilled case starts immediately upon pipe displacement. The force is then rapidly increased with a rate ten times faster than the slurry backfilled case. The ultimate magnitude of the mobilized force was increased by 67% in sandy backfill. The PIV analysis of the failure mechanism that will be discussed later in this paper shows that the sand backfill contributes to the p-y response in two different ways; first the resistance of the confined sand against the pipe displacement; and second, the passive pressure provided by the sand backfill against the collapse of the trench wall. The latter item is significantly affecting the failure mechanism and the total soil resistance mobilized against the pipe displacement. The response observed in chunky backfill is moderate in between the slurry and the sand. In this case, the ultimate resistance is higher than slurry and lower than sand. However, the results of chunk test presented in Fig. 8 is related to a case with trench wall angle of 30 degrees, which has not been yet correlated for different angle effect. In some of the cases (except slurry), the pipe does does not come to contact with trench wall, even after the full collapse of the wall.

There is always a compressed layer of the backfilling material separating the pipe and the trench wall. This will be further discussed in the section of failure mechanisms later in this paper. The test results show that interactive mechanisms between the pipeline, backfill, and trench can have a significant influence on lateral response and the ultimate soil resistance. This is not well considered in current design codes (e.g., PRCI 2009; ALA 2005). Further, post-processing is still going on to propose new sets of equations accounting for the effect of pipe-backfill-trench interaction on the prediction of lateral pipeline response.

Influence of Interaction rate and depth

In this testing program, the lateral pipe-soil interaction was studied under drained and partial drained conditions which have been less explored in the literature. Fig. 8 and Fig. 9 show the rate effect on the prototype-scale force-displacement of the pipes backfilled with slurry respectively for deep and shallow burial depth. The trench wall was vertical and the pipes started to touch the trench wall at 1D displacement form centreline. The lateral response of the pipe showed an earlier interaction with the trench wall under the drained condition, achieving an ultimate response of 25% higher than the partially drained condition. The interaction rate shows the slightly different effect on pipe response in shallow and deep burial depths. The ultimate resistance of the partially drained test in the shallow case is higher than the drained condition. This is inverse in case of deep burial, where the drained ultimate response is higher than the partially drained condition. This shows that rate effect is dependent on depth (effective vertical normal stress).

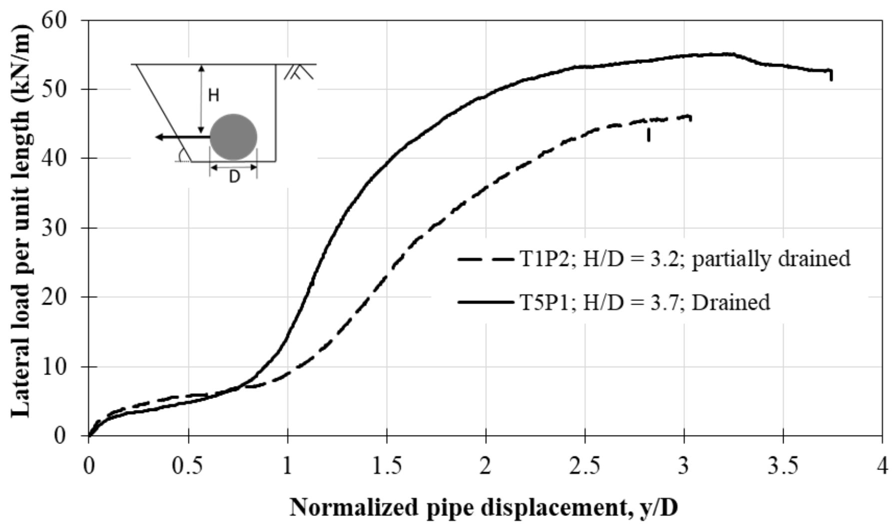

Fig. 8. Interaction rate effect on prototype force–displacement response (deep burial)

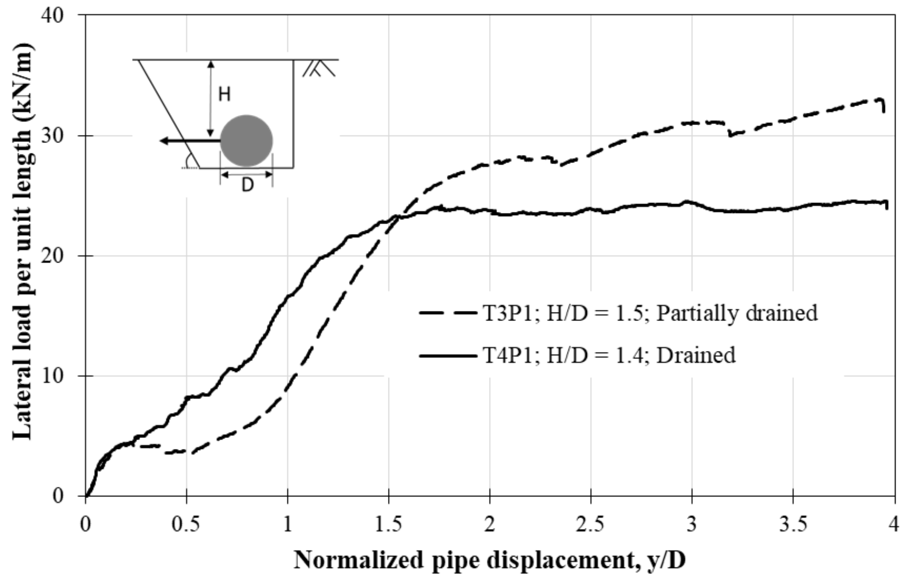

Fig. 9. Interaction rate effect on prototype force–displacement response (shallow burial)

Fig. 10 and Fig. 11 show the induced pore pressure in the rear of the pipe (internal PPT, inside slurry) and in front of the pipe (PPT-N1, in native soil), respectively. The pore pressure trend inside the slurry backfill shows almost no sensitivity to burial depth ratio and interaction rate. However, it is much different in native soil, where the pore pressure dissipation depends on both burial depth ratio and interaction rates. Fig. 10 shows that the pore pressure increases over the course of 0.25D penetration of the pipe into the trench wall in deeper burial case. The pore pressure is then continuously decreased in all cases, while the dissipation rate is different depending on burial ratio and interaction rates. Corresponding to the lateral responses discussed above in Fig. 10, the ultimate pore pressure in drained deeply buried pipe test is much lower than the shallowly buried pipe. Also, the results shows that the pore pressure arrives at a low ultimate state in the shallowly buried pipe.

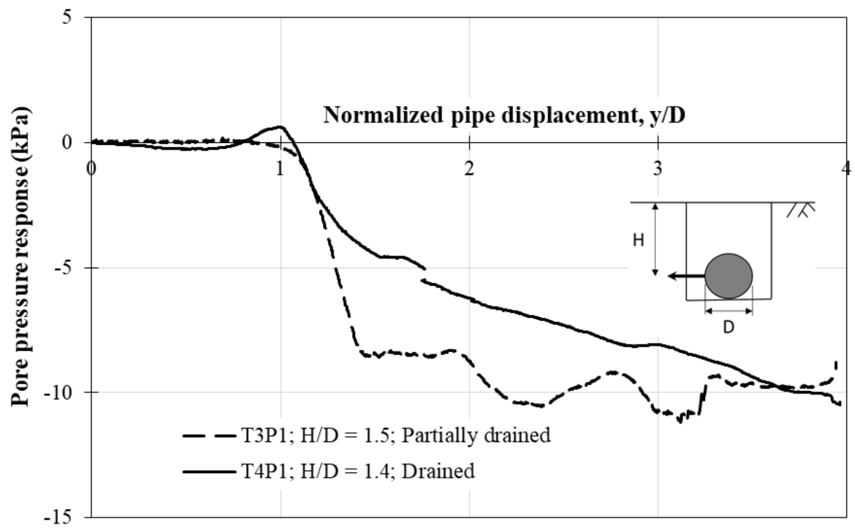

Fig. 10. PPT-N1 responses to pipe displacement

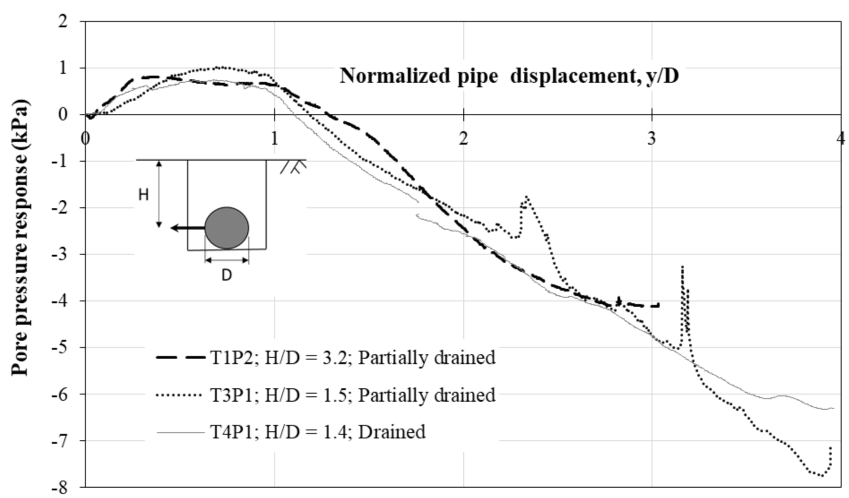

Fig. 11. Internal PPT responses to pipe displacement

The results showed the interaction rate might have a significant effect on lateral p-y response. In addition, different trends were observed in cases with different confining pressure. Neither of these effects is well considered in design practice, where the undrained shear strength is widely used for design purposes. The results presented above are samples of the obtained data. The post-processing along with advanced numerical simulations is still going on by authors to enable proposing new models for considering the consolidation and rate effects in the prediction of the lateral pipe response to large deformations in the cohesive material.

Failure mechanisms

The PIV analysis was conducted to reveal the deformations and failure mechanisms both in the backfill and trench. The load-displacement curve of a sample test (T5P1) is schematically illustrated in Fig. 12. The markers are referring to the intervals of the PIV analysis results. Ultimate resistance is obtained at about 3D of horizontal pipe displacement (2D penetration into the native soil). The developed shear bands are comparable in every stage with the corresponding force-displacement stage at Fig. 12. The slope of the pipeline force-displacement response has achieved its maximum value in the range of 1.0D to 1.5D.

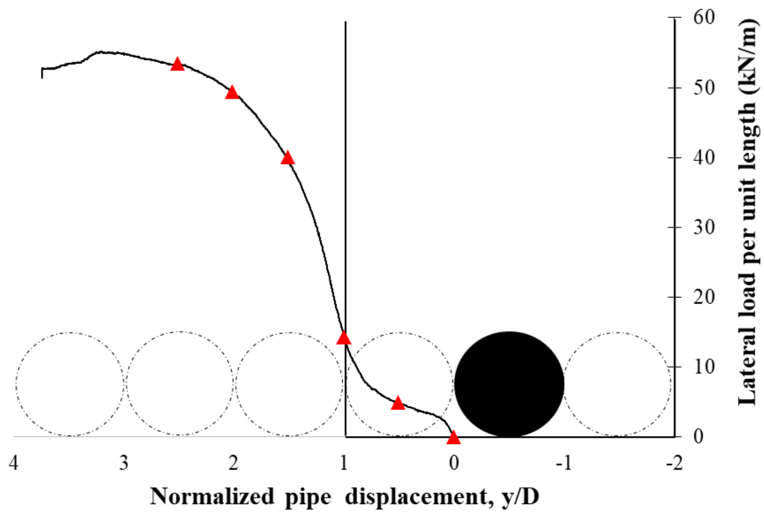

Fig. 12. Force–displacement of T5P1 in the schematic trench; PIV intervals are marked by triangle

Fig. 13 shows the displacement vectors in the range of 2.0D to 2.5D, where the pipe has penetrated into the trench wall. Gradual failure of the trench wall has caused the native soil to be pushed towards the backfill, where the backfill strength and the resultant passive resistance plays a vital role in achieving the ultimate resistance.

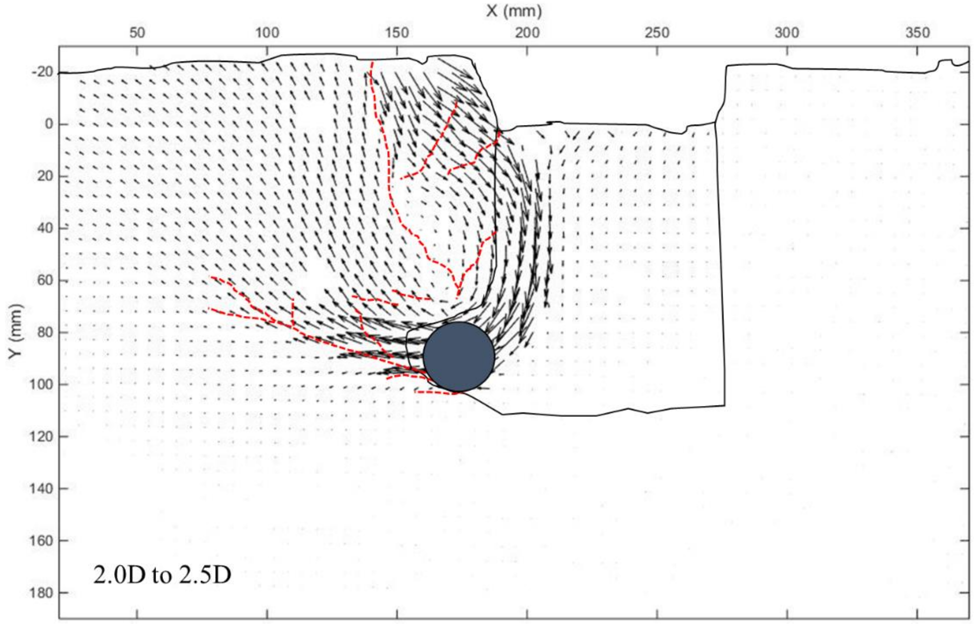

Fig. 13. Vectorial displacement for pipe movement from 2.0D to 2.5D

The progressive stages of soil deformation by 0.5D intervals are illustrated in Fig. 14. Considering a very soft backfill (slurry), there is no sign of strain in native soil from 0 to 1.0D. The low range of the resistance obtained in this region is due to the pipe friction with the trench bottom and the initiation of backfill flow around the pipe. From 1.0D to 1.5D, the native soil in front of the pipe is laterally compressed and vertically extended mobilizing the soil resistance in front of the pipe. When the pipe penetrates into the trench wall, the wall is gradually starting to fail, moving the surficial parts towards the backfill. This causes reducing the normal stress above the pipe and slightly vertical upward deviation of the pipe. However, the vertical tension component of the pulling cable restricts the pipe movement upward. After 2.5D displacement, the steady state soil resistance is almost achieved.

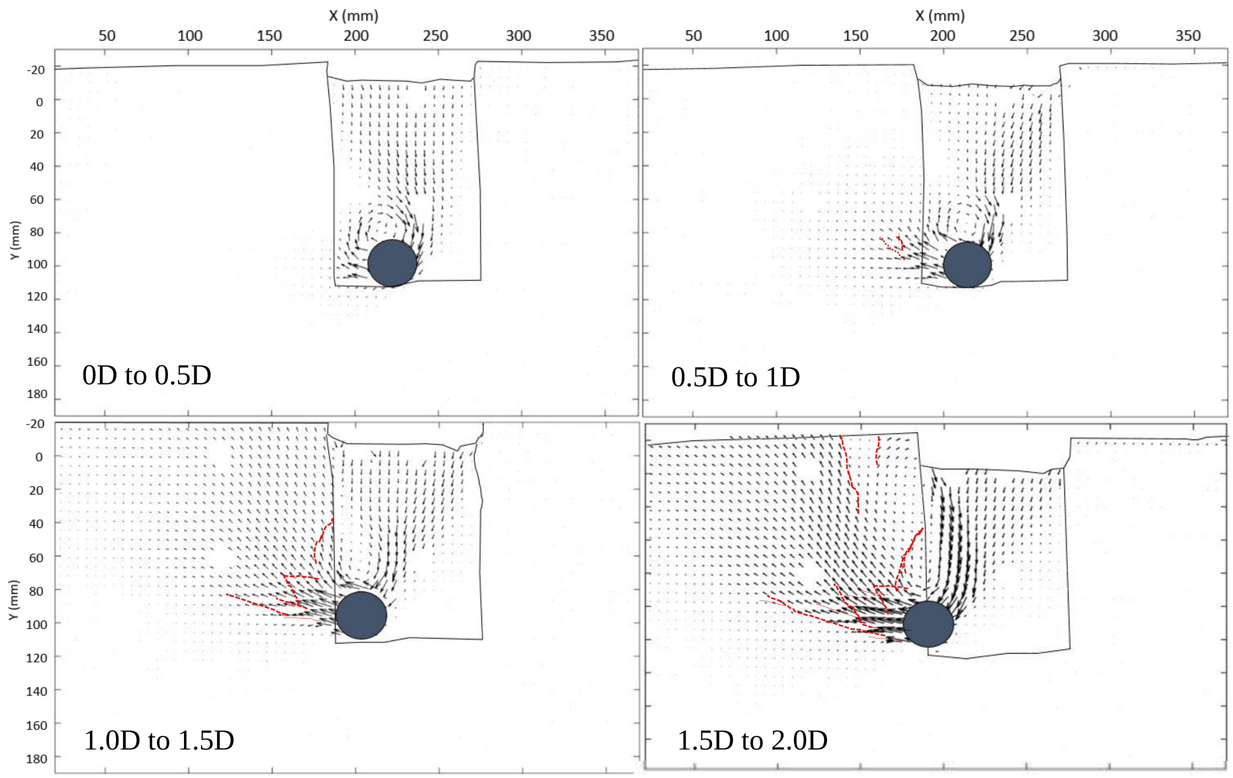

Fig. 14. Displacement fields during lateral displacement up to 2.0D pipe movement

The back-analysis of the test results is currently under process by authors. The results will enable proposing new failure models considering full scenarios of lateral pipe-soil interaction by incorporating the new finding in this program.

Summary and Conclusion

The lateral pipeline-backfill-trench interaction was studied through centrifuge testing of sixteen distinct pipe-soil configurations under drained and partially drained conditions. Transparent observation window and digital cameras were installed on one side of the plane strain testing box to capture the failure mechanisms of the backfill and trench wall within large pipeline displacements. Several key parameters affecting the lateral p-y response of the pipeline and ultimate resistance of the soil were investigated at 19.1 g acceleration including backfill properties, trench geometry, burial depth, and interaction rate effects. Full instrumentation was applied to pipes including strain gauges, load cells, conventional and laser LVDTs, miniature T-bar, internal and external pore pressure transducers, markers and patterns, etc. A comprehensive set of high-quality data was obtained, and the post-processing is still ongoing by the research team. The test set up and samples of initial results were discussed. As initial results of the conducted program the following conclusions were obtained:

- The backfilling properties which are governed by several constructional parameters may have a significant influence on lateral pipe response to large deformations.
- The lateral pipe response is governed by failure mechanisms in the backfill and trench wall which is affected by the relative strength of the backfill and native soil.
- Softer backfills result in less ultimate soil resistance.
- Pipeline may shift vertically upward during the trench failure. The magnitude of vertical displacement is increased by decreasing the backfill strength.
- The lateral pipe response is significantly affected by interaction rate. Considering the pipe-backfill-trench interaction, higher displacement velocity may result in lower or higher lateral resistance depending on the confining pressure. However, the variation trends are depending on trench geometry.
- The lateral pipe resistance has a direct relationship with changing the pore pressure.

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Appendix B

Centrifuge testing of lateral pipeline-soil interaction buried in loose sand

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Centrifuge testing of lateral pipeline-soil interaction buried in loose sand

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ABSTRACT

This paper presents the results of a series of small-scale centrifuge testing program conducted at C-CORE to explore the lateral response of pipelines to large deformations in dry sand. The model pipes were buried inside dry sand at different depths to investigate the load-displacement response to lateral displacement. Pipe diameter effect was also investigated by adjusting centrifuge acceleration. The results showed that the lateral resistance and the load-displacement response of the pipeline are significantly affected by burial depth, pipe diameter, relative density, and soil friction angle. The results were subsequently compared against the corresponding full-scale experiments, and a fairly acceptable agreement was observed. It was found that a smaller displacement is required in centrifuge to develop the peak resistance in comparison with the full-scale tests.

Introduction

Pipelines used for water or hydrocarbon transportation are exposed to environmental, geophysical, and operational risks. The risks include pipeline movements induced by internal pressure and temperature, ice gouging, fault activities, landslides, a range of field activity interference, anchors, environmental erosion, etc. In order to mitigate the risks, a common solution widely used in the industry is to bury the pipeline. The buried pipelines are confined and protected by the surrounding soil. Any factors which may cause a relative displacement between soil and pipeline are considered in the framework of pipe-soil interaction. In this framework, pipeline design engineers evaluate the estimated loads and deformations, using existing guidelines against ultimate and serviceability limit states. There are numerous analytical, numerical, and experimental studies that have been performed in the past to investigate pipe-soil interaction. These studies are categorized into axial, lateral, oblique, and uplift pipeline–soil interactions. Some of the physical models which have been executed in granular testbeds are summarized here:

Audibert and Nyman (1977) presented the results of tests using three different model pipelines with diameters of 25 mm, 60 mm, and 111mm in loose and dense sand with a cover depth ratio ranging from 1 to 24.

Trautmann and O'Rourke (1985) conducted 30 lateral pipeline-soil interaction tests using pipelines with 102 mm and 324 mm diameters buried in dry sand at various burial ratios of 1.5, 3.5, 5.5, 8, and 11. The investigated testbed densities were 14.8, 16.4, and 17.7 kN/m^3 representing loose, medium and dense sand. Hsu (1993) performed approximately 120 lateral pipe-soil interaction tests to investigate the effects of sand density, pipe diameter, pipe burial depth, and relative interaction velocity on pipe lateral soil restraint. Pipe diameters ranging from 38.1 mm to 228.6 mm were used and the pipe displacement rate ranged from 0.001 to 0.1 pipe diameters per second. The burial depth ratio varied from 0.5 to 20 and the embedment ratio varied from 1 to 20.5. In this study, the embedment depth ratio was used rather than the centerline depth, which was defined as the depth from the pipe bottom to the soil surface. Burnett (2015) conducted a series of large-scale tests at Queen's University. Pipeline-soil interaction was investigated through lateral imposed displacement. Two pipeline pieces of 914 mm length with various diameters were used in a test program under plane strain condition. Transparent windows mounted on both sides of the container, along with an image-capture system, enabled a detailed investigation of the failure mechanisms, soil deflections, and pipe trajectory path. The pipe diameter (D), burial depth (H), and sand density (γ) were the variable parameters studied throughout the testing program. The pipes (914 mm long with diameters 254 mm and 610 mm at burial depth ratios of 1, 3, and 7) were tested in both loose and dense sands. Debnath (2016) studied the axial, lateral, and oblique behavior of pipe–soil interaction using centrifuge small scale modeling. Karimian et al. (2006) conducted three lateral pipe–soil interaction tests in Fraser River sand with diameters of 324 and 457 at burial depth ratios 2.75 and 1.92, respectively. The relative density of the sand was considered to be around 70%.

In the present test program, the lateral pipeline-soil interaction was investigated through a series of centrifuge tests in both granular and cohesive testbeds. However, this paper only covers the dry sand tests. The experiments were conducted in a plane strain strongbox using small-scale centrifuge model, reproducing the pipe diameters studied by Burnett (2015). The tests performed in sand were intended to investigate the failure mechanisms, lateral resistance, and load-displacement response of buried pipelines to large displacements in granular material.

Testing setup and procedure

The testing program consisted of three series of tests engaging the pipeline-soil interaction in sand through large lateral displacements. The buried pipes were pulled in opposite directions over a large course of displacements (2.5 to 3.0D). In tests with deferent G-levels, the pipes were pulled in two individual stages. Sand was placed inside the box without any densification process. However, there would have been some slight levels of densification during the loading of box onto the platform and during centrifuge running.

Two model pipes were pulled in opposite directions and tested in each run resulting in six sand tests in total. The instrumentation of pipe-2 was not ready at the time of testing, therefore, the results of pipe-2 were not measured during the experiments. The assumed uniform lateral distributed force due to pipe-soil interaction was obtained using two shear strain gauges which were installed at two sections of the pipe. These two strain gauges measured all the shear force developed between the locations of the strain gauges. The schematic view of test-1 is shown in Figure 1. The internal dimensions of the testing box were 0.9 m by 0.3 m wide by 0.4 m high. The testing box was designed to simulate plane strain conditions, like an infinitely long-buried pipeline, which would experience similar conditions in the field. Both pipe and sand are restrained at two sides of the box and during the lateral movement of the pipe, sand cannot flow out of the plane.

The pipe diameter effect was investigated by changing the centrifuge acceleration in the second and third tests. The interactive soil deformation mechanisms were directly monitored through an observation window. One digital camera was installed in front of the observation window for the purpose of post-processing and Digital Image Correlation (DIC) analysis. Two vertical drivers were located on the strongbox in order to pull the cables through the pulleys at the level of the buried pipe. This configuration is designed to pull the pipes laterally in opposite directions with predetermined moving rates where pipes are free to move vertically. The test setup is designed to conduct two independent tests at the same time, therefore, sufficient margins and appropriate boundary conditions were incorporated to ensure that the interference between the soil failure zones in each test is prevented. Some of the dimensions shown in Figure 1 were compared with pipe diameters to facilitate an easier review of the boundary margins. More details about the test setup and comprehensive test program are discussed in Kianian et al. (2018).

The initial and post-test locations of the pipes and the trajectory of pipes are also incorporated in Figure 1. As the pulling lateral distances in the current study were up to 3 times the diameter of the pipe, the vertical displacement of the pipes also become a considerable value, generating an unrealistic vertical component that was introduced unintentionally to the system. This could be considered a limitation of the test setup where the pulling cable was not able to adjust itself with the vertical elevation of the pipe with the result that only a pure lateral force was produced. However, the vertical component was negligible. For example, at the end of test-1 (as shown in Figure 1), the final angle was 4° in T1P1, which imposed 4.7 kN/m extra vertical force in the prototype-scale at the end of the pulling distance. The downward vertical component has a slight increasing impact on the resistance of soil. The pipeline tendency to move upward during lateral pipe-soil interaction originates from the nature of the buried pipeline in terms of slip surface development toward the soil surface. The testbed was prepared using silica sand. The sand particle size analysis shows that the sand is poorly graded, having D_{50} = 0.19 mm and coefficient of uniformity C_u = 1.96.

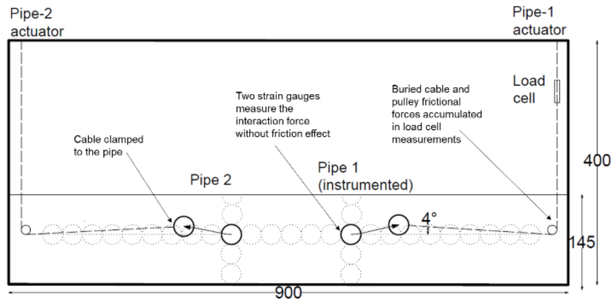

Figure 1. Schematic view of test 1; initial and post-test location of the pipe; all dimensions are in mm

The current experiments are designed with the purpose of investigating the behavior of the soil during large lateral deformations in comparison with the full-scale tests which have been done by Burnett (2015). The scales were selected in such a way to mimic the full-scale tests. The tests were designed to (a) investigate lateral pipe-soil interaction in a plane strain condition, (b) find more accurate analytical solutions for ultimate resistance, (c) reveal the failure mechanisms at different depths and scales, (d) determine the load-displacement (P-y) curves, and (e) assess the influence of depth, embedment ratio, and pipe diameter by changing the scale. Table B.1 summarizes the testing program. Burial depth ratio (H/D) is defined as the distance from the soil surface to the pipe centerline over the pipe diameter D. In test 2 and test 3 two various G-levels have been considered. Therefore the centrifuge conducted the tests in two stages with different accelerations.

Table B.1. Summary of the testing program

Test	Pipe	Test ID	Scale	Model pipe diam (mm)	Prototype pipe diam (mm)	Prototype depth (m)	Burial ratio, H/D	ϒ (kN/m3)	Confining pressure (kPa)	Resistance (kN/m)	Normalized resistance
Test 1	Pipe 1	T1P1	19.06	31.75	605.2	1.20	2.0	13.5	16.24	63.93	6.50
	Pipe 2	T1P2	19.06	31.75	605.2	1.20	2.0	13.5	16.24	-	-
Test 2	Pipe 1	T2P1	19.06	31.75	605.2	0.60	1.0	13.5	8.15	33.69	6.83
	Pipe 2	T2P2	7.95	31.75	252.4	0.25	1.0	13.5	3.40	-	-
Test 3	Pipe 1	T3P1	7.95	31.75	252.4	0.72	2.8	13.5	9.66	20.03	8.21
	Pipe 2	T3P2	19.06	31.75	605.2	1.72	2.8	13.5	23.16	-	-

Force-displacement and lateral resistance

This section presents a brief review of the force-displacement response obtained from the testing program. Prototype-scale force-displacement data was obtained by applying the appropriate scaling factors to model-scale data. The figures presented in this paper are all provided on the prototype-scale. It was observed in a granular material testbed that the lateral response of the pipeline could be significantly affected by several key parameters, mainly from pipe diameter, burial depth, and relative density. Lateral load-displacement relationships are commonly

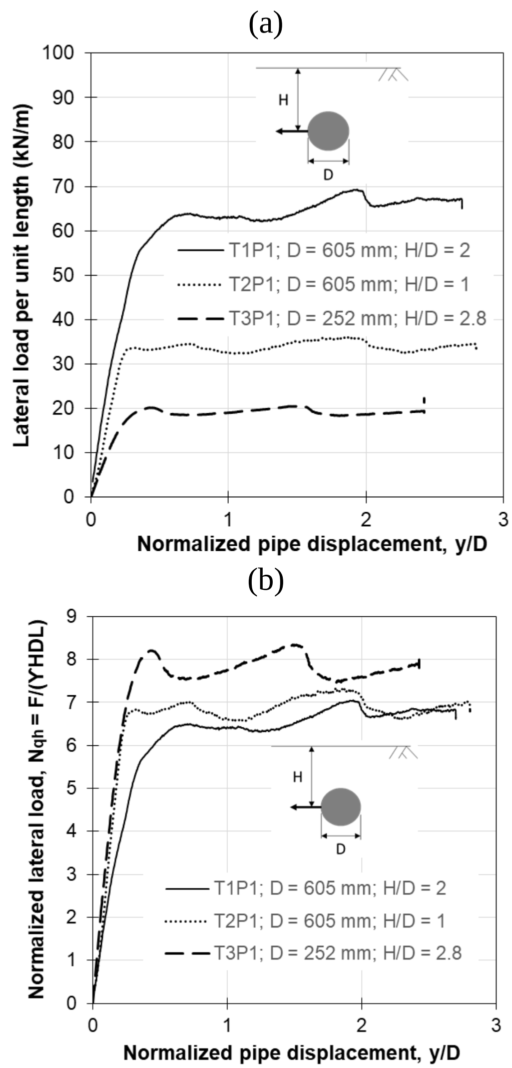

Figure 2. Force-displacement responses before and after normalization (a) prototype-scale, (b) normalized lateral resistance

expressed with the dimensionless load, $N_{qh} = F/(\gamma HDL)$, and dimensionless lateral displacement y/D in which F is the force acting on the test pipe, γ is the test soil density, H is the distance from the soil surface to the pipe centerline, D is the pipe diameter, L is the pipe length (0.3 m in current test), and y is the lateral pipe displacement. Figure 2 illustrates the lateral load-displacement curves. It was observed that increasing burial ratio or pipe diameter leads to increases in the lateral resistance applied to the pipeline. For a given pipe diameter at T1P1 and T2P1, pipelines tested at larger burial depth ratio experience larger lateral soil resistances and require a large displacement to mobilize peak lateral soil force.

Pipelines with a larger diameter experience larger lateral soil forces and require a large displacement to reach the mobilization distance. As shown in Figure 3, for the same pipe diameter in T1P1 and T2P1, the higher burial depth ratio (T1P1) experiences less upward movement.

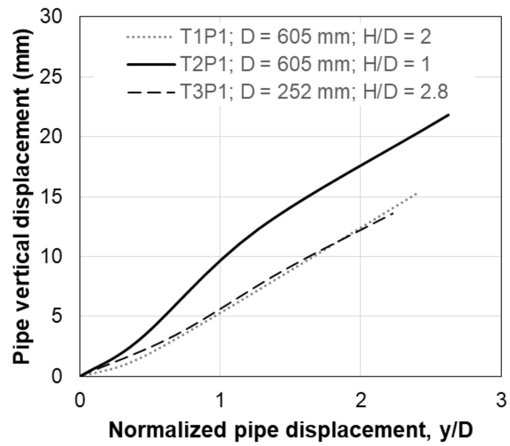

Figure 3. Pipe trajectories

Comparison with published studies

Figure 4 compares the test data with some other experimental studies and guidelines including, ALA (2005), PRCI (2009) and Rajah et al. (2014). All the data presented in this figure are selected from the previous experiments executed in loose sand testbeds. ALA (2005) predicts closer results to the present experiments. There are many sources of discrepancies, including friction angles, sand types, sand densities, and experimental procedures associated with test setups and their side effects on the produced results. Table B.2 describes some of the differences in the sand type, friction angle, scaling, pipe diameter, burial ratio, and relative density conditions.

The results of the current study are comparable with the results of the full-scale experiments conducted by Burnett (2015). There were several differences between the current study and Burnett (2015) including small scale modeling using centrifuge, as well as the sand type and relative density. Figure 5 shows the force-displacement curves of the current study in comparison with the corresponding full-scale experiments performed in olivine loose sand (Burnett 2015). All results show a favorable agreement. There are no major deviations in the trends seen in the load-displacement behavior, mobilization distances, or the maximum lateral soil forces. The load-displacement curves of both experiments show that increases in the burial ratio or pipe diameter lead to increases in the lateral resistance applied to the pipeline. The other conclusion is that for a given pipe diameter, pipeline tested at larger burial depth ratios, showed larger lateral soil resistance, which means that a larger displacement is required to mobilize ultimate lateral soil resistance. The displacement associated with the maximum resistance is defined as $0.04\left(H+\frac{D}{2}\right)$ which should not be taken more than 0.01D to 0.15D (ALA 2005 and PRCI 2009). This guideline is in accordance with the fact that both the depth and diameter of the pipe have a direct relationship with mobilization distance.

There are three main discrepancies in the results of full-scale (Burnett 2015) and centrifuge small-scale tests (current study) including (a) Full-scale experiments due to the higher level of relative density and, consequently, the friction angle, led to higher levels of ultimate resistance. (b) Full-scale tests due to higher relative density showed greater initial stiffness. (c) Mobilization distance seems to be shorter in centrifuge small-scale tests with respect to the full-scale tests. This might be because of the scale effect in centrifuge.

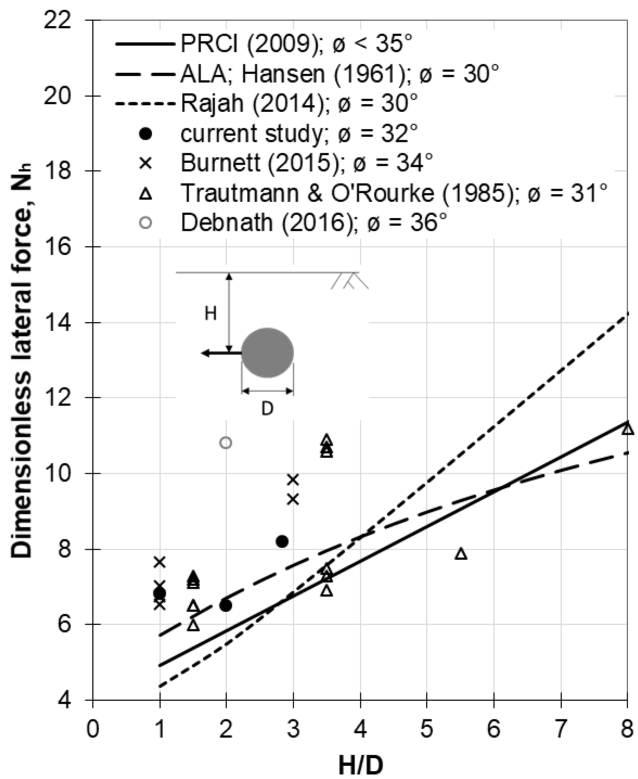

Figure 4. Comparison of the normalized resistance with several published test results and guidelines for loose sand

Table B.2. Lateral pipe–soil interaction experimental studies in sand

Experimental study	Sand type	Relative density condition	H/D	Pipe diameter (mm)	scale	Friction angle
Current study	Dry silica sand	Loose	1, 2, 3	252 & 605 prototype	7.95 & 19.06	32
Debnath (2016)	Dry silica sand	Dense and loose	2	609 prototype	13.25	32
Burnett (2015)	Dry synthetic olivine sand	Loose and dense	1, 3, 7	254 & 610	1	32.7 - 35.4
Karimian et al. (2006)	Moist & dry Fraser River	Medium dense	2.75, 1.92	324 & 457	1	32 - 34
Trautmann and O'Rourke (1985)	Dry Cornell filter sand	Loose to dense	1.5, 3.5, 5.5, 8, 11	102 & 325	1	31, 36, 44
Audibert and Nyman (1977)	Carver sand	Loose to dense	1 to 24	25, 63.5, 114.3, 228.6	1	35

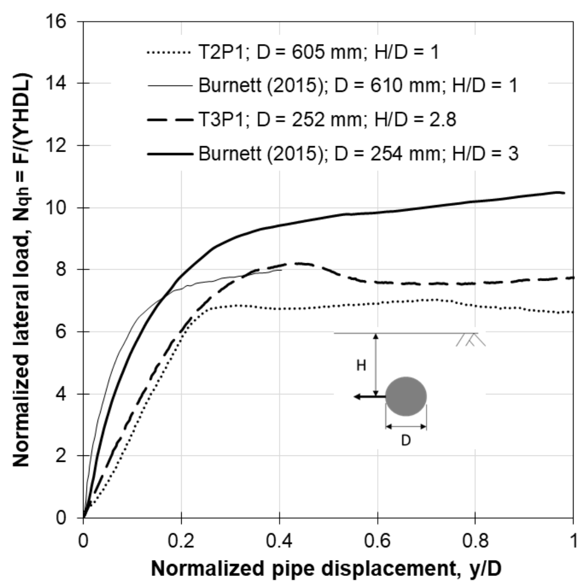

Figure 5. Comparison of the present study (very loose sand) with Burnett (2015) (full-scale loose sand)

Conclusion

A comprehensive understanding of pipe-soil interaction is necessary for the design of pipelines to minimize the risk from environmental, geophysical, and operational events. The response to large lateral displacement of the pipeline, regardless of the cause of the event, is crucial to the current understanding of pipeline-soil interaction.

This paper presents the results of a centrifuge experimental study of lateral pipeline-soil interaction induced by relative large horizontal movement of buried pipeline in silica sand under a plane strain condition. The present experimental program has focused primarily on the ultimate resistance against pulling the pipeline horizontally and the force-displacement relationships associated with the progressive mechanisms of failures that were observed through the window. The current results of the buried pipeline in pure granular soil without trench are then comparable with the results of the cohesive testbed and granular backfill (Kianian et al. 2018).

The results of the current small-scale centrifuge study are comparable with the results of the full-scale experiments conducted by Burnett (2015). There are three main discrepancies in the results, including (a) the full-scale experiments due to a higher level of relative density and consequently, the friction angle showed a higher level of ultimate resistance. (b) Full-scale tests due to higher relative density showed greater initial stiffness. (c) Mobilization distance seems to be shorter in centrifuge tests with respect to the full-scale tests. This might be because of the lower relative density in centrifuge tests. Initial observations of the conducted testing program can be summarized as follows:

- ALA (2005) predict closer to the results of the present experiments.
- Between the assessed guidelines, ALA (2005) showed closer prediction for the lateral bearing factor (normalized lateral force)
- All test results agree with previously published literature. There is no major deviation in the trends. Overall, centrifuge tests underestimate the ultimate resistance of soil in comparison with associated full-scale tests.
- There is no specific criteria to select the mobilization distance in loose sand therefore the choice of the distance required to develop maximum load in loose sand from the experimental data is somewhat subjective. And there is high variability in this parameter.
- An increase in pipe diameter leads to higher lateral soil resistance and, therefore, greater ultimate values. Larger pipe diameter results in more upward movement during pure lateral actuation.
- Deeper burial depths result in greater required displacement to develop ultimate resistance (mobilization distance)

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Appendix C

Constructional and operational considerations in assessing the lateral response of buried subsea pipelines

This paper has been accepted for presentation in the 4th International Symposium on Frontiers in Offshore Geotechnics (ISFOG) to be held in Austin, Texas on 16-19 August 2020.

CONSTRUCTIONAL AND OPERATIONAL CONSIDERATIONS IN ASSESSING THE LATERAL RESPONSE OF BURIED SUBSEA PIPELINES

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ABSTRACT

Subsea pipelines passing through the shallow waters are usually buried inside the subsea trenches that are backfilled with pre-excavated material as a cost-effective protection solution. Buried pipelines may undergo large lateral displacements due to ground movement, landslides, ice scour, etc. Environmental, constructional, and operational loads significantly remold the backfilling material and make it much softer than the trench wall or native ground. This aspect that significantly affects the lateral pipe-soil interaction is not considered by current design standards due to the less explored interactive soil failure mechanisms around the trenched-backfilled pipeline. In this study, the lateral pipeline-backfill-trench interaction and the resultant soil failure mechanisms were investigated by centrifuge tests and advanced numerical simulations. Transparent observation windows equipped with digital cameras and state-of-the-art instrumentation were used to monitor the soil deformations directly and run particle image velocimetry (PIV) analysis. Using the experimental and numerical observations the influence of range of constructional and operational considerations on lateral response of the pipeline were discussed such as trenching and backfilling methodology, construction procedure, construction season, operational loads, environmental and operational thermal effects, longitudinal seabed profile, pipe weight, pipe type, backfill properties, backfill buoyancy, etc. Several new research avenues were identified, and their significance in assessing the ultimate pipeline response to lateral loads was discussed.

Keywords: Lateral pipe-soil interaction; p-y response; large deformation; centrifuge testing; trenching and backfilling

1. INTRODUCTION

Subsea pipelines that are widely used for the development of offshore fields can be subjected to large lateral displacements due to environmental, operational, and accidental loads (e.g., ground movement, ice gouging, drag anchors, etc.). Subsea trenching and backfilling by re-using the excavated material or purchased material is usually a cost-effective solution to protect the pipeline against lateral displacements. Depending on the constructional, operational, and environmental conditions, the backfill material may undergo different degrees of remolding and disturbance. This process causes the backfill material to be usually much softer than the native ground, with a wide range of shear strengths ranging from negligible to almost native soil strength values (Paulin et al. 2014). The difference between the stiffness of the backfill and native material can significantly affect the failure mechanisms around the pipe and the resultant lateral soil resistance. Fig. 1 schematically shows the potential scenarios that may happen depending on the relative backfill/native soil stiffness.

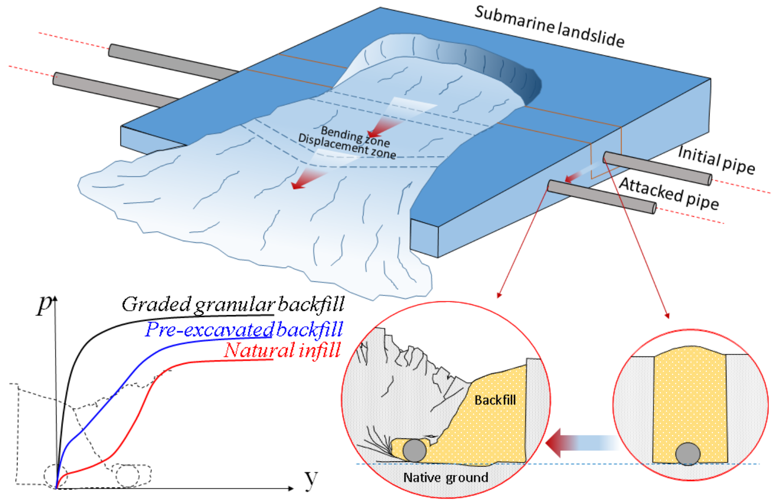

Fig. 1. The lateral response of trenched and backfill pipeline to subsea geohazards

However, the less-explored interaction between the pipeline, backfill, and the native ground (trench walls) has caused the design standards to simplify the buried pipe configuration to a uniform soil. The practical incorporation of this simplification needs excavation of an adequately wide trench that results in a high construction cost. This is only to ensure the pipeline response will depend solely on the properties of the controlled backfill material, and not on the stiffer native ground (Kouretzis et al. 2013).

Most of the experimental studies in the literature have been conducted in the sand, and there is a very limited number of studies in clay. Many of the proposed or potential models are based on anchor plates and piles because of relatively similar behavioral fashion with pipelines. There are some limited models based on pipelines lateral interaction (Oliveira et al. 2010, Poorooshasb et al. 1994, Paulin 1998). Wantland et al. (1982) conducted experimental studies in an estuary in an unconsolidated highly plastic montmorillonitic clay with an undrained shear strength increasing by l-2 kPa with depth. The main objective of the study was to investigate the effects of pipe weight, diameter, embedment depth, loading rate, and soil properties on lateral resistance. Ng (1994) evaluated the results of the field lateral loading tests in clay against the existing empirical equations. The author incorporated the interaction effects between the pipeline and two distinct materials as backfill and the native ground by defining a factor to modify the bearing capacity factor affected by a backfilled trench. Ng (1994) fitted the p-y curves based on the field tests using the hyperbolic p-y equations provided by Audibert and Nyman (1977) and Trautmann and O'Rourke (1985). Eventually, the author proposed a modified hyperbolic formulation using the Nc values suggested by the Committee on Gas and Liquid Fuel Lifelines (ASCE, 1984). Paulin et al. (1998) conducted a series of lateral pipeline-soil interaction tests in clay. The key objectives of the program were to study the effects of trench width, burial depth, interaction rate, backfill properties, and stress history of the soil on p-y curves. The program was maybe the first comprehensive study in its kind, incorporating the effect of backfill and trench on the lateral response. The authors observed that the trench width had little or no effect on an undrained interaction, while the undrained load on pipeline increased with increasing the burial depth. The pipeline displacement rate (drainage conditions) showed a significant effect on the loads transferred to the pipeline. They concluded that the undrained p-y response and ultimate loads could be predicted within ±20% using existing methods of analysis. The authors observed that the backfill properties could affect the overall normalized interaction between the pipeline and the soil. However, they couldn't determine if this is due to a change in failure mechanism or a change in the separation condition behind the pipeline. Kianian et al. (2018), and Kianian and Shiri (2018, 2019a,b,c, and 2020a) conducted a series of centrifuge tests (titled S1P1 to S2P2, and T1P1 to T5P2) to investigate the internal soil displacements and failure mechanisms around the trenched-backfilled pipeline. The authors adopted partially-drained to drained conditions to magnify the trenching and backfilling effect on lateral soil resistance (Paulin 1998). Using Particle Image Velocimetry (PIV) analysis, the authors identified different failure mechanisms in three different zones of the pipe displacements, i.e., i) small displacements inside the trench (no pipe-trench wall interaction yet), ii) pipeline approaching to trench wall, (initiation of pipe-trench wall interaction) or, transition zone, and iii) pipeline penetrating into the trench wall, (full development of pipe-trench wall interaction). The authors observed that the pipeline-backfill-trench interaction and the resultant lateral soil resistance is significantly affected by relative stiffness of the backfill and native ground, burial depth ratio, and pipeline-trenchbed interaction intensity.

In this paper, the influence of constructional and operational conditions on the lateral soil resistance of the trench-backfilled pipelines have been discussed based on some of the recent findings of the study conducted by Kianian et al. (2018), and Kianian and Shiri (2018, 2019a,b,c, and 2020a). As will be discussed throughout the paper, there are several factors most of them ignored by the current design codes, that may significantly affect the lateral p-y response of the trenched-backfilled pipeline. Some of these factors include trenching and backfilling methodology, construction procedure, construction season, operational loads, environmental and operational thermal effects, longitudinal seabed profile, pipe weight, pipe type, relative stiffness of backfill to native ground, rate effects, backfill buoyancy, etc. The study showed that proposing a simple p-y curve for a uniform soil condition, as currently done by design codes, is a gross simplification and may result in significant underestimation or overestimation of the lateral soil resistance in different zones of displacements. The study revealed several new research avenues for future studies to improve the integrity and cost-effectiveness of pipeline design against lateral displacements.

2. CENTRIFUGE TEST SETUP

Centrifuge tests were conducted at C-CORE located at the St. John's campus of the Memorial University of Newfoundland (Newfoundland and Labrador, Canada) to investigate the lateral soil response to the large displacements of the shallowly and deeply trenched-backfilled pipelines. A 50%-50% mixture of Speswhite kaolin clay and Sil-Co-Sil silt was used to prepare a slurry with a nominal moisture content of 70%. The native soil bed was incrementally consolidated up to the effective stress of 400 kPa and then was unloaded to 100 kPa with an open drainage valve. This resulted in a soft native ground (undrained shear strength less than 25 kPa), which has been observed in shallow waters (e.g., water depth less than 100 m) over different geographical locations (e.g., Bohai Sea (Liu et al. 2013), Mackenzie Delta (Solomon 2003), and Changi Bay (Bo et al.

2015)). Trenches were excavated in prepared native ground and backfilled with three different types of material, including very soft slurry (representing natural infill), chunky clay (representing pre-excavated seabed soil), and loose sand (representing purchased granular material). Two pieces of 32 mm model pipe corresponding to the 24” prototype were horizontally pulled in the opposite direction at a spinning acceleration of 19.1g that was a technical requirement of the project’s industrial sponsor. Several instruments were used along with 3 parallel strong data acquisition systems (each has 8 individually configurable inputs) to monitor the testing program. The instruments included pore pressure transducers (PPTs), strain gauges, load cells, conventional and laser linear variable differential transformers (LVDTs), T-bar, vertical drive motion controller, digital cameras, markers, and artificial textures. This enabled the full recording of progressive failure mechanisms and the development of shear bands in backfill and native soil, the lateral force-displacement response of the pipeline, the suction force variation behind the moving pipe, and the pore pressure variation both in the backfill and native ground. Fig. 2. shows a schematic sample of the test setup.

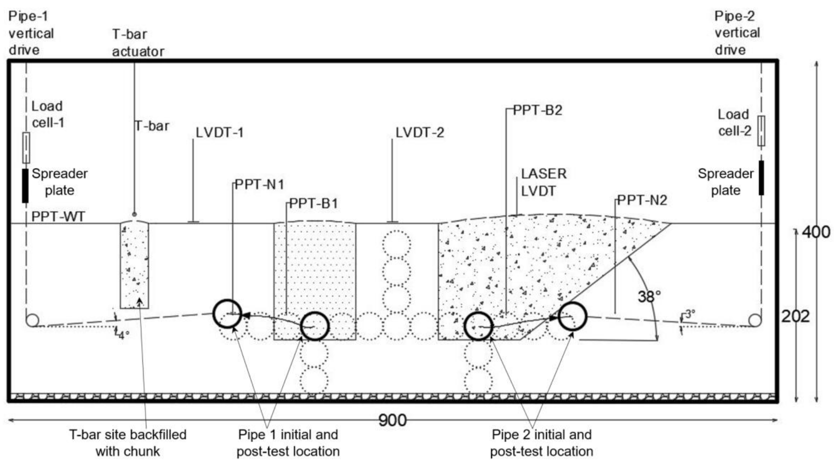

Fig. 2. Test setup (Kianian and Shiri (2019a))

The pipeline displacement rate was set sufficiently low (vD/cv about 0.2, partially drained based on Phillips et al. (2004)) to consolidate the surrounding soil, eliminate the effect of excess pore pressure and magnify the effect of pipeline-backfill-trench interaction. This was motivated by reviewing the observation made by Paulin (1998), where the higher pipeline displacement velocities (undrained condition) caused the backfill to become less of a factor in interaction. In addition, in many geographical locations, silt fractions are found in natural offshore soft clays (e.g., the Gulf of Mexico, Schiffman (1982) that tend the consolidation characteristics of clay towards partially drained and even fully drained conditions. Other compositional and depositional fractions may also show a similar effect. Fourteen tests were conducted including the initialization tests. Further details of the testing program can be found in Kianian and Shiri (2018a).

3. KEY OBSERVATIONS

3.1 Effect of backfilling stiffness

In these series of studies, three tests (T1P2, T2P1, and T5P2) were conducted on trenched pipelines deeply buried in three different kinds of backfills, including loose sand, slurry, and chunky clay to qualitatively investigate the internal soil deformations and failure mechanisms affected by different backfilling strength. These three backfilling material represented the purchased (granular), natural infill (slurry), and pre-excavated material (remolded soft clay) that are the most probable scenarios. Fig. 3 shows the p-y responses and samples of PIV analysis results for these three tests.

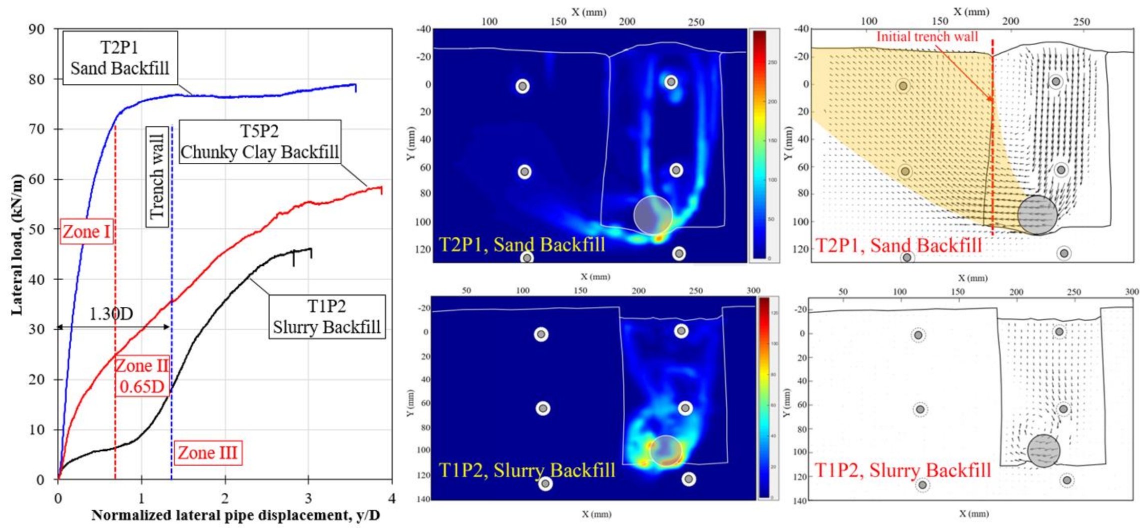

Fig. 3. The p-y curves and sample of PIV results in the zone I for the tests with three different backfills (Kianian and Shiri (2020a))

As shown in Fig. 3, there is no zone I in the test with sand backfill since the pipeline interacts with the trench wall immediately after starting the pulling. Inversely, the pipeline has no interaction with the trench wall in case of slurry backfill, while moving in zone I. The test with chunky clay backfill is located between the slurry and sand backfill tests. Assuming almost a nil shear strength for slurry, the 5 to 7 kN/m soil resistance in the zone I of T1P2 is the result of pipeline-trenchbed interaction that will be discussed in the next section. Fig. 4 reveals the large difference between the ultimate soil resistance in the cases with slurry and sand backfills. The passive pressure in the sand backfill (T2P1) does not allow the trench wall to collapse into the backfill. This prevents the formation of global failure surfaces in the native ground and accumulation of soil resistance resulting in a higher magnitude of soil resistance. In the case of slurry backfill, due to extremely low shear strength, the backfill does not provide any passive pressure. Therefore, the global collapse of the trench wall occurs through the formation of failure surface and the lateral soil resistance is significantly reduced due to the dissipation of energy in shear bands.

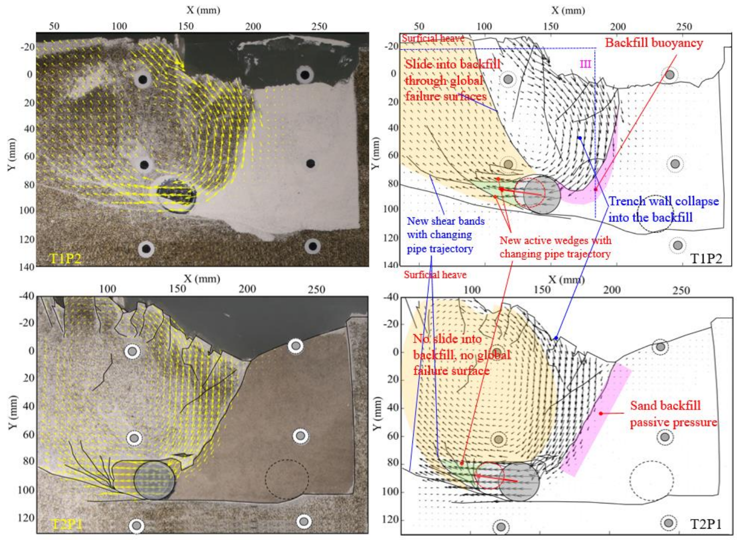

Fig. 4. Sample of PIV results in zone III for the tests with slurry and sand backfills (Kianian and Shiri (2020a))

Further details of the PIV analysis in zone II and chunky backfill can be found in Kianian and Shiri (2020a).

3.2 Effect of pipeline-trenchbed interaction

Paulin (1998) reported the pipeline embedment into the trench bed. However, the authors did not investigate its impact on the lateral p-y response. Kianian and Shiri (2019b) investigated the effect of pipeline-trenchbed interaction intensity on the lateral soil resistance by performing two centrifuge tests, i.e., T3P1 (Horizontally pulled) and T4P1 (downward inclined pulled) both by slurry backfills. The authors applied an initial inflight embedment of 0.1 D into the trench bed. Then the elevation of the pulley and the pipe springline was adjusted to perform horizontal and downward inclined pulling. The downward inclined pulling (by 2 to 5 degrees) represented a more intense pipe-trenchbed interaction. It was observed that the intense pipe-bed interaction enlarges the soil berm in front of the pipe and squeeze it into the bottom corner of the trench. This, in turn, expedites the pipeline-trench wall interaction and pushing the pipe upward to inter into the trench wall in an oblique direction. Fig. 5 schematically shows the different soil berm deformation in horizontal and downward pulling.

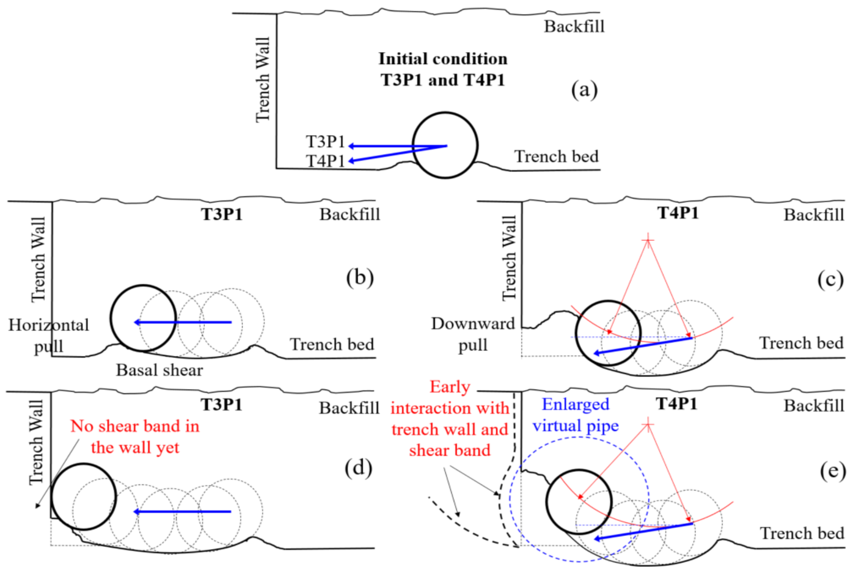

Fig. 5. Schematic illustration of the bed interaction process in horizontal and downward pulls (Kianian and Shiri (2019b))

Fig. 6 shows the p-y response and sample of PIV results in zone II and III for these tests. It was observed that more severe pipeline-trenchbed interaction (T4P1) resulted in an earlier shear band formation in the trench wall, arriving the failure surfaces to the ground surface, dissipating the energy through the shear band, and consequently less magnitude of ultimate soil resistance. The shear band initiation in the trench wall was delayed by reducing the intensity of pipeline-trenchbed interaction (T3P1). This, in turn, results in a series of premature failure surfaces that never arrived at the ground surface and accumulated a larger amount of lateral soil resistance against the pipe penetrating into the trench wall.

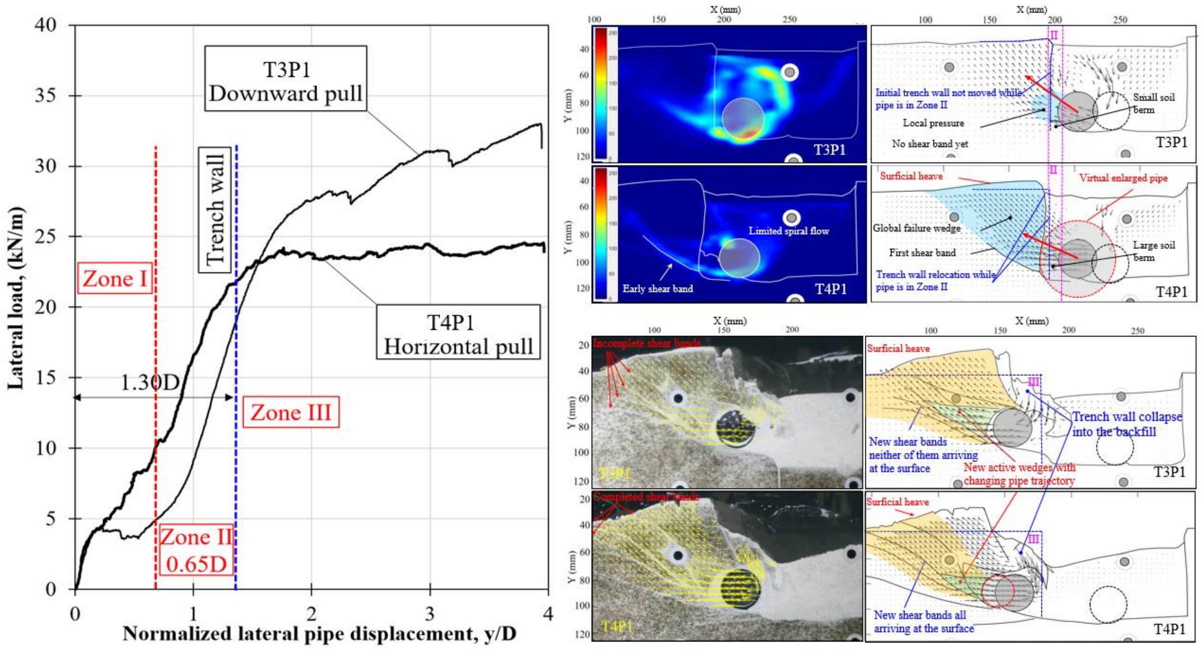

Fig. 6. The p-y response and sample of PIV results in zone II and III (Kianian and Shiri (2019b))

The p-y response in Fig. 6 shows that less interaction with the trenchbed can result in a higher magnitude of ultimate lateral soil resistance. Fig. 7 schematically shows the performance of the design codes (e.g., PRCI) compared to experimental observations. The simplifications by design codes do not necessarily result in conservative design. Depending on the magnitude of pipe displacements and the combination of bed interaction intensity and backfill stiffness, the design codes may overestimate or underestimate the lateral soil resistance.

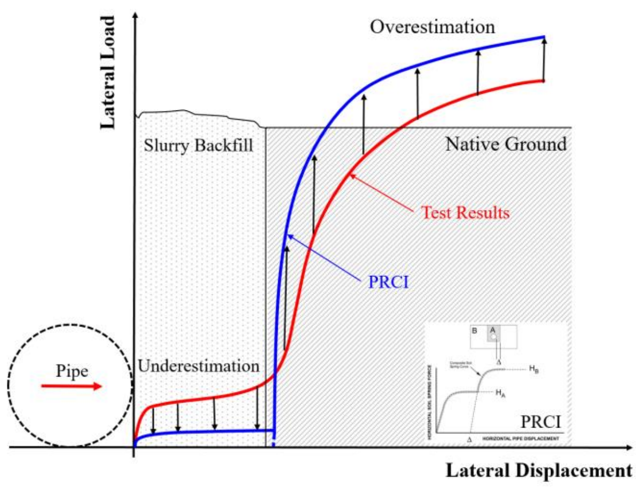

Fig. 7. Schematic illustration of the p-y curves by design code and experimental observations

Several project-specific parameters affect the stiffness of the backfill and the bed interaction but are neglected in the design process. This will be further discussed in the next section.

4. Effect of Constructional and Operational Conditions

Several project-specific constructional and operational parameters such as trenching and backfilling methodology, construction procedure, construction season, operational loads, environmental and operational thermal effects, longitudinal seabed profile, pipe weight, pipe type, backfill properties, backfill buoyancy, etc., may affect the backfill stiffness and pipeline-bed interaction intensity, and consequently the lateral soil resistance. These parameters that some of them are summarized in Table C.1 are ignored by current design codes. Table C.1 qualitatively shows the combined effect of pipeline-trenchbed interaction intensity and the backfilling material stiffness on the lateral soil resistance. The signs ↑↑, ↑, ↓, ↓↓, and ↕ are referring to highly increased, increased, decreased, highlight decreased, and dual-way effect, respectively.

Table C.1. The combined effect of pipeline-trenchbed interaction intensity and backfill stiffness on lateral soil resistance

Parameter	Options	Effect on Bed interaction intensity	Lateral soil resistance			
			Granular backfill	Predredged backfill	Jetting backfill	Natural infill
			H	M	L	VL
Trenching geometry	Rectangle	H	↕	↕	↓	↓↓
	V-shape	L	↑↑	↑	↕	↕
	Trapezoidal-wide bottom	L	↑↑	↑	↕	↕
	Trapezoidal-Narrow bottom	H	↕	↕	↓	↓↓
Trenching method	Back hoe excavator	L	↑↑	↑	↕	↕
	Cutter suction dredger	H	↕	↕	↓	↓↓
	Hopper suction dredger	L	↑↑	↑	↕	↕
	Plough	L	↑↑	↑	↕	↕
	Jetting	H	↕	↕	↓	↓↓
Pipe weight and type	Steel	H	↕	↕	↓	↓↓
	HDPE	L	↑↑	↑	↕	↕
Installation method	Pulling on the trench bed	H	↕	↕	↓	↓↓
	Lifting, shifting, submergence	L	↑↑	↑	↕	↕
Construction season	Warm season	H	↕	↕	↓	↓↓
	Cold season, permafrost	L	↑↑	↑	↕	↕
Ground longitudinal profile	Crest	H	↕	↕	↓	↓↓
	Trough	L	↑↑	↑	↕	↕
Operational load	Heavy content	H	↕	↕	↓	↓↓
	Light weight content	L	↑↑	↑	↕	↕
	High content temperature	H	↕	↕	↓	↓↓
	Low content temperature	L	↑↑	↑	↕	↕
Construction method	Trench bedding	L	↑↑	↑	↕	↕
	No trench bedding	H	↕	↕	↓	↓↓

The recent findings discussed throughout the paper suggest that the least bed interaction intensity combined with the highest backfilling stiffness may result in the highest lateral soil resistance. Inversely, the most intense bed interaction combined with the lowest backfilling stiffness may result in the lowest lateral soil resistance. The combination of low bed interaction intensity and low backfilling stiffness may have a dual way effect on the lateral soil resistance. The same trend may happen with high bed interaction intensity combined with high backfilling stiffness. Table C.2

describes the mechanism that each parameter affects bed interaction intensity. The brief results presented throughout the paper show that the soil resistance against the large lateral displacements of the trenched-backfilled pipelines may significantly be affected by several project-specific parameters that are ignored by design codes. The important point is that these gross simplifications do not result in conservative design in many cases. The current design codes may overestimate or underestimate the lateral soil resistance depending on the magnitude of pipe displacements and the combination of bed interaction intensity and backfill stiffness.

Table C.2. Influence mechanism of construction parameters on bed interaction intensity

Parameter	Options	Effect on Bed interaction intensity	Description
Trenching geometry	Rectangle	H	* Sharp corners, trapping the soil berm and backfill, high bed interaction intensity
	V-shape	L	* No sharp corners, low bed interaction intensity
	Trapezoidal-wide bottom	L	* Pipe pass over the soil berm, less soil berm accumulation, less bed interaction
	Trapezoidal-Narrow bottom	H	* Close to rectangle trench, high bed interaction intensity
Trenching method	Back hoe excavator	L	* Bed is compacted to some extent, less initial embedment, less interaction intensity
	Cutter suction dredger	H	* Uneven trench bed, more initial embedment, higher bed interaction intensity
	Hopper suction dredger	L	* Even bed and compacted to some extent, less embedment, less interaction
	Plough	L	* Bed is compacted to some extent, less initial embedment, less interaction intensity
	Jetting	H	* High remoulding, more chance for initial embedment, higher interaction intensity
Pipe weight and type	Steel	H	* Heavy weight, more initial embedment, higher bed interaction intensity
	HDPE	L	* Light weight, less initial embedment, lower bed interaction intensity
Installation method	Pulling on the trench bed	H	* Increases the initial embedment, higher bed interaction intensity
	Lifting, shifting, submergence	L	* Does not increase the initial embedment, less interaction intensity
Construction season	Warm season	H	* Softer bed, more initial embedment, higher bed interaction intensity
	Cold season, permafrost	L	* Stiffer bed, less or no initial embedment, less bed interaction intensity
Ground longitudinal profile	Crest	H	* Pipe's natural tendency towards the bed, downward lateral move, higher interaction
	Trough	L	* Pipe's natural upward tendency, upward lateral move, less bed interaction
Operational load	Heavy content	H	* Heavy weight, more initial embedment, higher bed interaction intensity
	Light weight content	L	* Light weight, less initial embedment, lower bed interaction intensity
	High content temperature	H	* Thaw settlement in permafrost, more downward embedment, higher bed interaction
	Low content temperature	L	* Less downward embedment, lower bed interaction
Construction method	Trench bedding	L	* Horizontal shear zone underneath the pipe, less bed interaction intensity
	No trench bedding	H	* Higher chance for bed interaction intensity

Further research works on most of the aforementioned constructional parameters seem mandatory for a more accurate and reliable design of lateral pipe-soil interaction.

CONCLUSION

The trenching and backfilling may have a significant effect on lateral soil resistance against the largely displaced pipelines. These effects are not considered in the current design practice that simplifies the soil as a uniform layer. Recently conducted centrifuge tests showed the significant impact of relative backfill/native ground stiffness and the pipeline-trenchbed interaction on lateral p-y response. Backfill with higher shear strength provide passive pressure against the trench wall collapse and prevent the formation of the global failure surface. This results in increasing the ultimate soil resistance. More intense bed interaction accelerates the development of failure surfaces in the trench wall, where the stored energy is dissipated and prevents the accumulation of high lateral soil resistance. The backfilling stiffness and bed interaction can be widely affected by several constructional and operational parameters that are not considered in design such as trenching and backfilling methodology, construction procedure, construction season, operational loads, environmental and operational thermal effects, longitudinal seabed profile, pipe weight, pipe type, backfill properties, backfill buoyancy, etc. The study showed that ignoring these effects results in a level of inaccuracies in the estimation of lateral soil resistance. An accurate assessment of the lateral p-y response needs the incorporation of these project-specific parameters.

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