Experimental and Numerical Modeling of Lateral Pipeline-Trench Interaction Backfilled with Sand

by

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ABSTRACT

Subsea pipelines are usually buried for physical protection in shallow waters. Pipelines may undergo large lateral displacements duce to ice gouging, ground movement, extreme thermal gradients, fish traps, pulling by anchors, etc. Sand backfills that have a different stiffness relative to the native ground are sometimes used for backfilling of the pipelines. The different stiffness of the sand backfill and the native ground affects the failure mechanism around the laterally moving pipe, and consequently the ultimate laterally mobilized soil resistance. This important effect is not considered by design codes in the lateral design of pipelines due to less explored failure mechanisms in pipeline-backfill-trench interaction process. In the current study, the lateral interaction between trenched pipeline backfilled with loose sand was investigated by performing centrifuge model tests. Soft slurry and lose sand backfills were used to facilitate investigation of the backfill stiffness effect. Transparent observation window was used with digital cameras to conduct Particle Image Velocimetry (PIV) and capture the internal soil deformation mechanisms. State-of-the-art instrumentation was used to collect high-quality data from the pipe, backfill, and trench. Partially drained condition was adopted to allow for full development of interaction mechanisms. Advanced numerical simulation of the conducted the tests was also conducted by using Coupled Eulerian-Lagrangian (CEL) analysis and built-in constitutive soil models in ABAQUS/Explicit. The study showed the significant influence of the relative backfill-trench stiffness on the lateral response of pipeline to large displacements. Comparisons with design codes revealed that the proposed equations by design code underestimate the lateral response inside the backfill, overestimate the lateral response for pipe penetrating into the trench wall, and propose no prediction for the pipe approaching the trench wall.

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Finally, I should thank Negin, my wife, for her support and all-round assistance, patience throughout my research work. I am sincerely grateful to my parents' devotion and unconditional love. This accomplishment would not have been possible without them. Thank you!

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CHAPTER 1: INTRODUCTION

1.1 Background

One of the common methods to protect subsea pipelines against the internal and external loads is to bury the pipelines inside the excavated trenches. The buried pipelines may go under large lateral displacements due to ground movement, ice gouging, accidental loads, etc. The dredged material is usually used as a cost-effective solution for backfilling of the pipeline. However, there are some occasions that cohesionless material such as sand is used for burying the pipelines. Based on construction strategy, dredging/trenching methodology and environmental loads, the degree of remolding and/or densification may vary in different kind of backfilling materials. However, regardless of the nature of backfill, the stiffness between the backfilling material and the cohesive native ground is largely different. consequently, this affects the response of the pipeline to large lateral movement through altering the soil failure mechanisms around the buried pipeline.

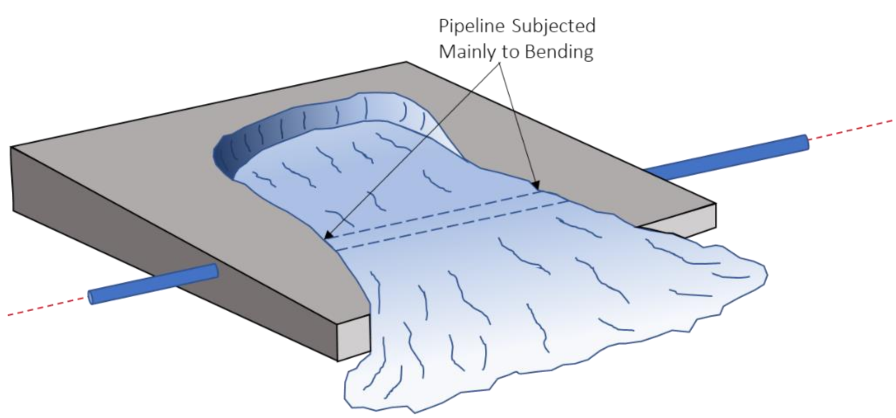

Figure 1-1 Example of subsea geohazard and the pipeline subjected to lateral loading

The relative displacement between surrounding soil and the pipeline applies forces on the pipelines. The vastness of these forces and the pipe force-displacement response which induced by deformations could be controlled by various factors including the submerged weight of native and the mobilized backfilling soil, horizontal shearing resistance presented by interacted soil and the suction behind the pipeline. Successively, these parameters are related to geo-mechanical properties of the native soil, backfill, trench geometry, confining pressure, burial depth, pipeline roughness, loading rate, pipeline size, soil stress history, the degree of backfill consolidation and the native soil's over consolidation ratio (OCR).

In reality, by describing the force-displacement relationship in a set of independent springs the response of the pipeline could be analyzed in a homogeneous soil media which is considered a gross simplification in comparison with a realistic trenched-backfilled pipe (e.g., ALA 2005). In this approach, springs' behavior is indicated by hyperbolic or bilinear functions (PRCI 2009; ALA 2005) that do not account for trench effects. This is due to the lack of sufficient knowledge about the effect of trench and backfill on internal soil deformation mechanism and its impact on lateral soil resistance.

In order to contribute to the filling of this crucial knowledge gap, series of centrifuge tests and numerical simulations were conducted using the centrifuge facilities at C-CORE. The novelty of the currently conducted study was the using of a transparent observation window in the side of test strongbox along with digital cameras and performing PIV analysis. This approach enabled direct observation of the internal soil deformations affected by trenching and backfilling effect and its impact on ultimate soil resistance.

In addition, the existing solutions usually use the undrained shear strength which may not be proper for lower rating loads. The drained or partial drained condition can also be encountered in a various geographical location having silt fractions in the seabed sediments. Therefore, in the current study, partially drained condition was adopted by lowering the interaction rate the pipe moving velocity to allow the full development of pipeline-backfill-trench interaction. The key objectives of the current research work are outline in the next section.

1.2 Objectives

- Observing lateral soil deformations and failure mechanisms in both the backfilling material and the native trench wall.
- Obtaining the lateral force-displacement (p-y) response and the ultimate soil resistance affected by trenching and backfilling.
- Obtaining the pore pressure variation and potential suction force mobilization behind the moving pipe and its potential contribution to lateral load.
- Ascertaining the interaction properties of the pipeline-backfill-trench for loose sand condition.
- Calibration of the numerical model using the test results

1.3 Outline of the thesis

This is a paper-based thesis with three chapters already published, except Chapter 4 which is a submitted journal paper and is currently under review. The thesis is composed of six chapters. Chapter 1 outlines the introduction and the main objectives of the thesis. Chapter 2 reviews the literature and investigates the previous studies conducted in the field. Chapter 3 is a published conference paper, presented in the Offshore Technology Conference (OTC2018, Houston, Texas, USA). The paper described the full details of the conducted testing program and presented a summary of the key results and observations. This paper was co-authored by another PhD student. The candidate contributed to 100% of the testing operation, but only two tests were solely considered for the current thesis. The rest of the tests was for the other PhD project. Chapter 4 was submitted as a journal paper to present the post-processing of the tests results conducted on sand backfills. The paper comprehensively discussed the observed internal failure mechanism affected by trenching and backfilling effects. Chapter 5 presents a published conference paper that was presented in GeoEdmonton2018 (Edmonton, Canada). The paper described the numerical simulation of the tests conducted on sand backfills using the advanced large deformation analysis (CEL) in ABAQUS/Explicit. The main conclusions of the conducted study are presented in Chapter 6 that is accompanied by recommendations for future studies.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

In 2017, Canada safely delivered over 1.4 billion barrels of crude oil and 5.7 trillion cubic feet of natural gas, where the pipelines play a vital role in support of more jobs and drive economic growth across Canada (www.cepa.com). The safety and the integrity of these important elements of the energy field developments is one of the highest priorities of the involved parties. One of the main challenges in buried pipeline design is the effect of natural forces and geohazards on the mechanical response and integrity of pipelines. In certain situations, pipelines can be exposed to potential ground failures, such as surface faulting, liquefaction-induced soil movements, and landslide induced permanent ground deformation (PGD). In the current state-of-practice (e.g., Committee on Gas and Liquid Fuel Lifelines of ALA 2002), the pipeline is generally modeled by a simplified beam in a homogeneous soil media that is represented by simple springs in axial (or longitudinal), transverse horizontal, and transverse vertical directions using Winkler type model (Winkler, 1867) as shown in Figure 2-1.

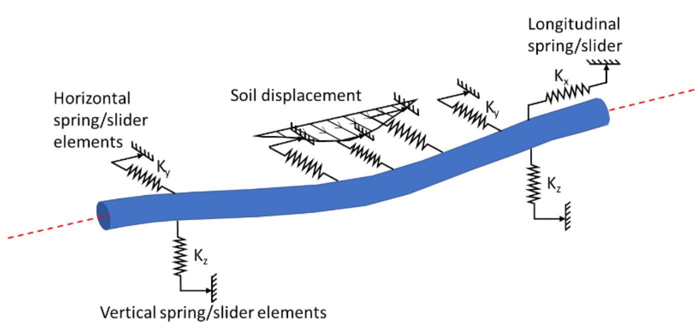

Figure 2-1 Schematic representation of soil reactions rafter (O 'Rourke and Lane. 1989)

The properties of soil springs in three orthogonal directions are independent which means that the deformation of soil in one direction has no effect on pipe/soil interactions in other directions. The general form of the load-displacement relations for these springs can be expressed as:

$$T = f(x); P = f(y); Q = f(z) \tag{2-1}$$

Where T, P and Q are the soil loads applied to unit length of the pipeline and x, y and z are the relative displacements between pipe and soil in longitudinal, lateral and vertical directions, respectively. Neither of this group of approaches considers the trenching and backfilling effects and result in overestimation of the ultimate lateral response.

For the pipeline buried in sand, ALA (2002) provides two models to calculate the horizontal bearing factor, N_{qh}, (Figure 2-2 and Figure 2-3). These models may be used for the sand backfill if the trench width is wide enough to prevent any interaction with trench wall. However, this is rarely happening in real practice, where the trench width is minimized to reduce construction costs. The first model proposed by ALA is based on the work of Audibert and Nyman (1977). They adapted Hansen (1961) model for vertical piles subjected to lateral loading and a good agreement with experimental data was found. The value of N_{qh} increases with soil friction angle and burial depth-diameter ratio, H/D (PRCI, 2005). The second model uses the work conducted by Trautmann (1983) to adopt the N_{qh}. The proposed predictions were in good agreement with the solution proposed by Ovesen and Stromann (1972) for vertical plate anchors subjected to horizontal loading that has a similar fashion to the lateral pipe response. For the same burial depth and soil properties, the factor N_{qh} obtained from the model of Hansen (1961) is 50 to 100% greater than that obtained from the Ovesen and Stromann (1972) based model (PRCI, 2003). Guo and Stolle (2005) reconciled the difference between Hansen (1961) and Ovesen and Stromann (1972) based on the size effect, stress level, and soil weight.

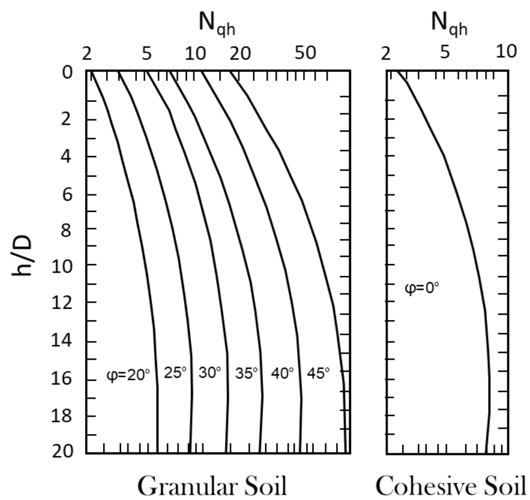

Figure 2. 2 Horizontal bearing capacity factors as a function of depth to diameter ratio for pipelines (after ASCE. 1981).

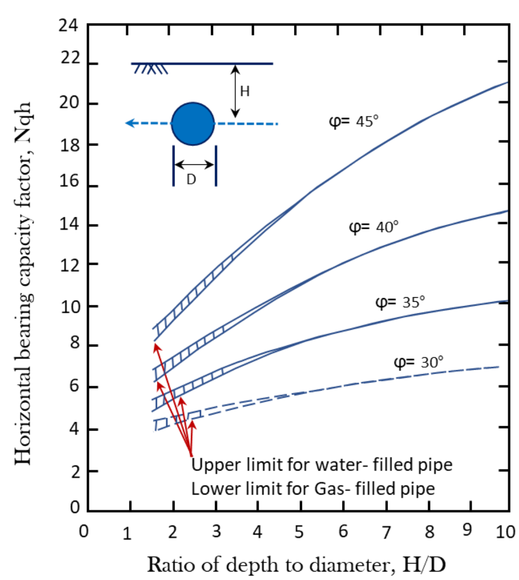

Figure 2-3 ASCE horizontal bearing capacity factor ((after Trautmann and O'Rourke (1983))

There is an only a limited number of experimental and theoretical models in literature to speculate on the force-displacement (p-y) and ultimate lateral resistance curve for pipelines in clay. As the same behavioral fashion with pipelines, anchor plates are the base of most of the presented models (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, Merified et al. 2001). A great number of other solutions are proposed based on the 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 are a few models based on the lateral interaction of pipelines (Oliveira et al. 2010, Poorooshasb et al. 1994, Paulin 1998).

A group of lateral pipeline-soil interaction centrifuge tests was conducted by Paulin (1998) in clay to investigate the impacts of burial depth, trench width, interaction rate, stress history and backfill properties of soil on the curves of force-displacement. This is maybe the only systematic research work that has widely investigated the trenching and backfilling effect on lateral pipe response to large displacements. The author employed four equipped aluminum pipelines which had 250 mm length and 19 mm diameter which were corresponded to prototype pipe with a length of 12.5 m and diameter of 0.95 m (1:50 scale). The test bed was a blend of Sil-Co-Sil silt and kaolin clay (50%-50%) which had an undrained shear strength of 40 kPa after consolidation. The pipeline was dragged horizontally with different velocities by the actuator to get lateral p-y responses. It was discovered by the authors that the trench width had minor or no impact on an undrained interaction, whereas as the burial depth increases the undrained load on the pipeline will increase. The authors concluded that the transferred load from soil to pipeline is significantly affected by the displacement rate of the pipeline. The authors stated that by using the existing analysis procedures ultimate loads and p-y response could be estimated by $\pm 20\%$. Paulin (1998) tried to capture the soil failure mechanism by using threads of printed spaghettis. This method only offers some qualitative information about failure mechanism, and there is no direct visualization data, and it makes this method less reliable. However, this was an indirect observation, and they couldn't sufficiently outline the internal soil deformation. This important knowledge gap was filled in the current research work. The authors stated that the overall normalized interaction between soil and pipeline might be influenced by backfill properties. Although, they could not ascertain if this is caused by a change in the separation condition behind the pipe or a change in failure mechanism. Paulin (1998) spotlighted the necessity for more investigation in order to improve the magnitude of the current database to decrease scatter in experimental data. This could improve the current analytical methods. For further studies, the effects of pipeline end, internal pressure, and backfill properties were also suggested.

In the current study, a full set of monitoring and state-of-art instrumentation were utilized on the backfill, pipeline, actuation system, native soil, and whole test configuration. The author used a digital camera, transparent acrylic sheet and particle image velocimetry (PIV) to attain interactive and progressive failure mechanisms. Altogether, this study boosted the current comprehension of the lateral response of entirely buried pipes to large deformations and offered a complete understanding of this inspiring problem.

Furthermore, the undrained shear strength parameter is regularly used for assessing the pipeline-soil interaction. The rate dependency of pipeline response will be neglected by using this method. In real pipe-soil interaction circumstances, both drained and partially drained states are completely frequent, where in these conditions the rate of relative displacement between soil and the pipeline is very moderate. In such instance, during the displacement, the soil which is surrounding the pipeline reaches some degree of consolidation. Besides, in a lot of geographical locations, silt fragment is found in soft natural offshore clays (e.g., Gulf of Mexico, Schiffman 1982). The consolidation properties of clay tend toward partial drained or even drained if silt presents in clay. Similar effect maybe indicated by further compositional and depositional fragments. In clay, the drained response of the pipeline induced by large deformations has been less investigated (Paulin 1998). In this study, the pipeline response induced by large lateral displacement in both partially drained condition was adopted to incorporate the rate dependency in pipeline response.

CHAPTER 3

LATERAL RESPONSE OF TRENCHED PIPELINES TO LARGE DEFORMATIONS IN CLAY

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This paper was presented at the Offshore Technology Conference held in Houston, Texas, USA, 1–4 May 2018. The contribution of the candidate was to 100% of testing program, but only the tests covering the sand backfills are covered in the current MEng program. The rest of the tests were for the PhD studies of the first author above.

3.1 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 mechanism in surrounding soil and the lateral load-displacement response of the pipeline, consequently. These important considerations are less covered

in 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. Transparent observation window combined with digital cameras were used for Particle Image Velocimetry (PIV) analysis. Full instrumentation was installed on pipeline, backfill, and trench to obtain the key data and the lateral p-y response of the buried pipe. The influence of several key parameters on lateral pipeline response were also investigated including backfilling properties, trench geometry, interaction rate effect, and burial depth. The results showed that the assessment of accurate failure mechanisms affected by various pipeline-backfill-trench interaction parameters has significant impact on 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 on pipeline design to large lateral displacements.

3.2 Introduction

Subsea pipelines may be buried inside the excavated trenches in cohesive soils for protection against the external and internal loads. Trenching and laying the pipeline may take place at the same time or in different period of 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 trenching/dredging methodology, construction strategy, and environmental loads, the backfilling material may experience different degrees of remoulding leading to different geomechanical properties. This, in turn, affects the failure mechanisms and pipeline response to large lateral displacement that may be caused by ground movements, 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 interacted soil, and the suction behind the pipe. These parameters, in turn, depend on 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 springs are 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). 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 are the main sources of observed discrepancies. In addition, the models proposed for prediction of 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 pipeline 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 instrumented test 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.

3.3 Previous experimental studies in clay

Most of the experimental pipeline studies in the literature were conducted in the sand. There is 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 anchors plates because of similar behavioural fashion 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 base 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 are some limited models based on pipelines lateral interaction (Oliveira et al. 2010, Poorooshasb et al. 1994, Paulin 1998).

Paulin (1998) conducted a series of lateral pipeline-soil interaction centrifuge tests 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 maybe the first small-scale comprehensive study on the lateral response of fully buried pipelines in clay incorporating the effect of backfill and trench. They used four instrumented aluminum pipes with a diameter of 19 mm and length of 250 mm corresponding to a prototype pipeline with a diameter of 0.95 m and 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 pipeline increased with 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. Authors tried to monitor the failure mechanisms using strands of painted spaghettis. This technique provided some qualitative information about the failure mechanisms, but lack of direct visualization made it less reliable. Authors noted that 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. 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. The effects of internal pressure, pipeline end conditions, and the backfill properties were also recommended for further investigations.

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 instrumentation and monitoring was applied on the pipeline, backfill, native soil, actuation system, and the whole tests 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 interaction. This approach results in neglecting 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 soil is very slow. In such occasion, the soil surrounding the pipeline is achieving some extent of consolidation 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 affects the consolidation characteristics of clay towards the partial drained and even 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 explored (Paulin 1998). The current study more focused on partially drained and drained response of the pipeline throughout large lateral displacements to investigate the rate dependency of the pipeline response.

3.4 Testing program

The testing program comprised five series of tests involving the lateral pipeline-backfill-trench interaction in clay throughout large lateral displacements (up to 4D) at a centrifuge acceleration of 19.1g. Two similar pieces of pipes with different configuration were pulled in opposite directions and tested in each run resulting ten tests in total. In addition, three series of tests (six pipe tests) were conducted in the dry loose sand. However, the current paper is only discussing the tests conducted in clay. The details of 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 direction with pre-determined moving velocity, while pipes were free to move vertically at least over a large course of displacement.

The main objectives of the testing program were 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 angle), backfilling properties, interaction rate, soil stress history, and suction force mobilization;
- Developing analytical models for lateral p-y curve and ultimate soil resistance
- Evaluation and improvement of the current practice for lateral pipeline-soil interaction

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 going on and the results will be published accordingly. Samples of failure mechanism and corresponding PIV analysis is also provided. The testing schedule was defined to maximize the obtaining of required high-quality data. Table 3-1 gives a summary of the conducted testing program.

Table 3-1 Summary of the conducted testing program

Test	Test bed	Pipe	Test name	Scale	Model cover depth (mm)	Embedment ratio, H/D	Trench backfill type	Trench wall	Model displacement rate (mm/s)	Normalized velocity vD/c_v	Normalized pulling distance
Test 1	Cohesive	Pipe 1	T1P1	19.06	92	3.90	Chunk	Inclined (30°)	0.00896	0.407	2.61
		Pipe 2	T1P2	19.06	92	3.90	Slurry	Vertical	0.00909	0.412	3.03
Test 2	Cohesive	Pipe 1	T2P1	19.06	99	4.12	Loose sand	Vertical	0.00929	0.422	3.60
		Pipe 2	T2P2	19.06	99	4.12	Slurry	Inclined (60°)	0.00916	0.416	3.50
Test 3	Cohesive	Pipe 1	T3P1	19.06	33	2.04	Slurry	Vertical	0.00944	0.428	3.93
		Pipe 2	T3P2	19.06	33	2.04	Chunk	Inclined (30°)	0.00923	0.419	3.82
Test 4	Cohesive	Pipe 1	T4P1	19.06	32	2.01	Slurry	Vertical	0.00300	0.136	3.93
		Pipe 2	T4P2	19.06	32	2.01	Chunk	Inclined (30°)	0.00301	0.136	3.87
Test 5	Cohesive	Pipe 1	T5P1	19.06	98	4.09	Slurry	Vertical	0.00298	0.135	3.71
		Pipe 2	T5P2	19.06	98	4.09	Chunk	Inclined (30°)	0.00301	0.137	3.85

3.5 Experimental setup and testing procedure

3.5.1 Modelling considerations

The main objective of the testing program was to investigate the pipeline-backfill-trench interaction and its impact on the force-displacement response of pipeline within 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 pipeline. Therefore, a plane strain container with Acrylic side window was used to monitor the failure mechanisms for further PIV analyses explicitly. The effects of variation in burial depth, trench geometry, interaction rate, and backfill properties were other objectives of this study to ensure the results could be confidently scaled up to full-scale conditions. shows 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 3-1 illustrated the boundary conditions normalized to pipe diameter using dotted circles.

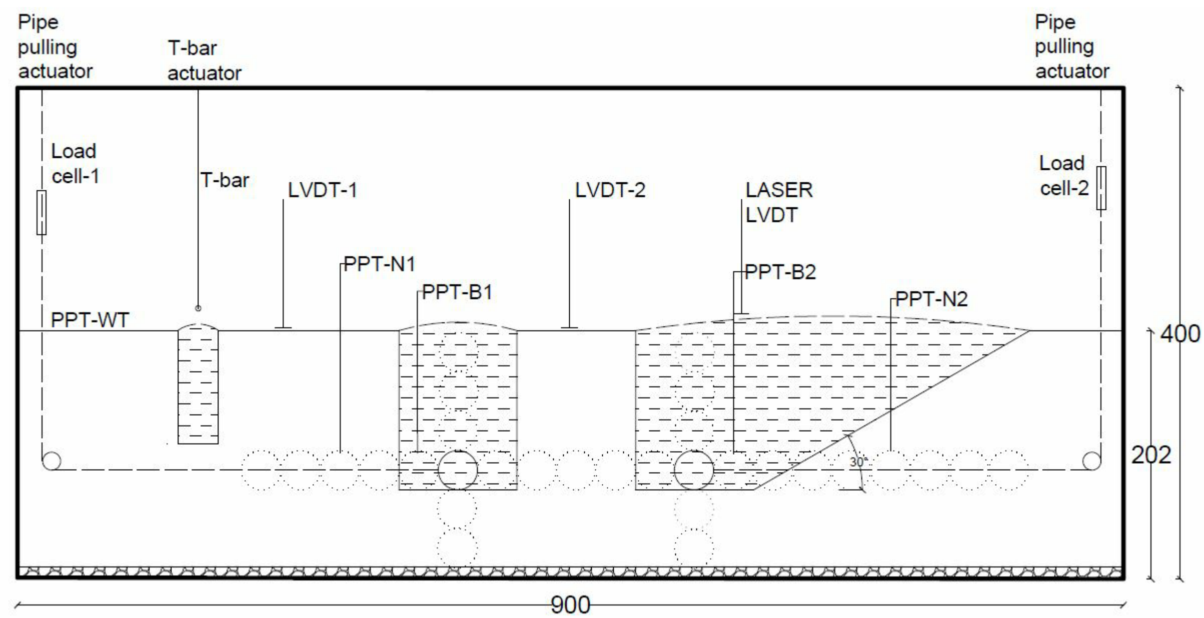

Figure 3-1. Schematic view of test setup (cohesive test bed); Instrumentations are coded; all dimensions are in mm

The soil sample was consolidated to effective stress of 400 kPa and was unloaded sequentially. This level of consolidation yielded soft clay with 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 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 of 610 mm diameter as targeted by the industry sponsor. This pipe size was same the earlier tests conducted in the sand (Burnett 2015) representing size range of export pipelines. Different embedment ratios (H/D) ranging from 2 to 4 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 trench wall (90°, 60°, and 30°). The trench wall behind the pipe was kept vertical assuming minor effect on lateral pipe response moving in opposite direction.

The effect of interaction rate has rarely been considered in developing the existing prediction models (Paulin 1998). In reality, depending on the nature of the interaction, the pipeline displacement rate could be in the order of 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 instrumentations was used for full monitoring of the testing program such as pore pressure transducers (PPTs), strain gages, load cells, linear variable differential transformer (LVDTs), T-bar, actuators and vertical drive motion controller, digital cameras, markers and artificial textures.

3.5.2 Soil preparation

Different procedures were used to prepare the native soil bed and various backfilling materials trying to simulate the realistic field conditions better. 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 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 (Figure 3-1). 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 3-2 and Figure 3-3 and 3-4 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.

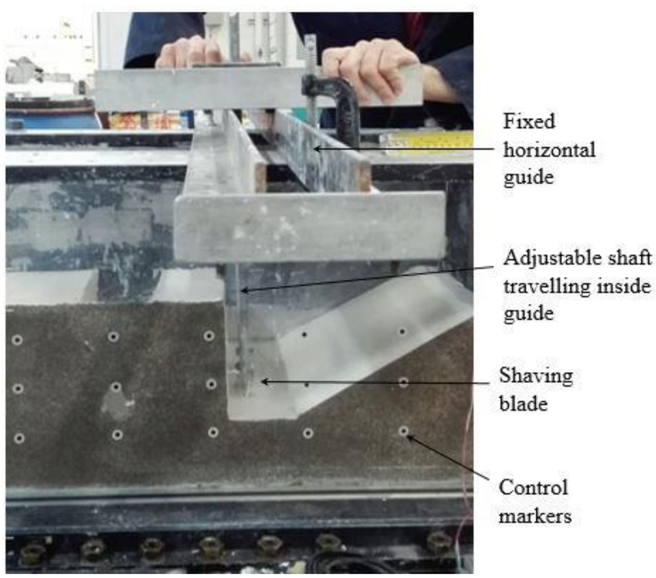

Figure. 3-2 Excavating trench bottom using blade

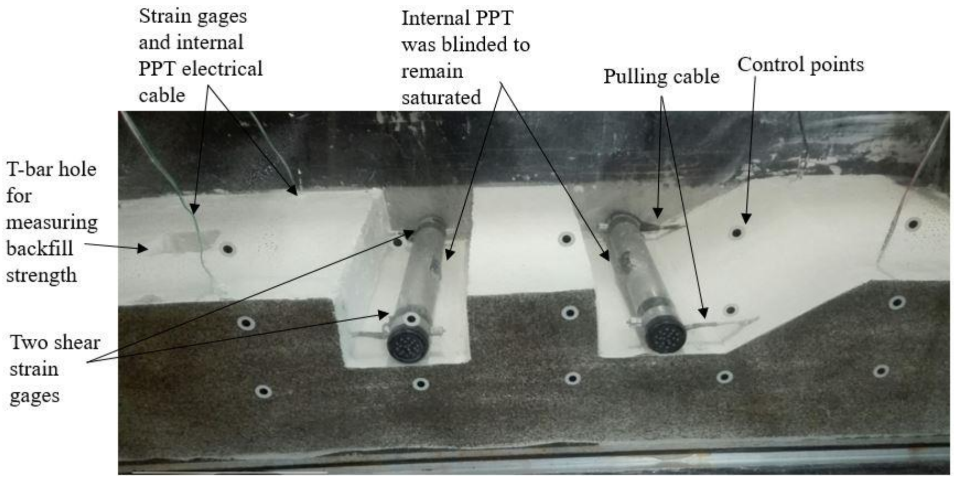

Figure 3-3 Box front view; Pipes installed inside two excavated trenches before backfilling

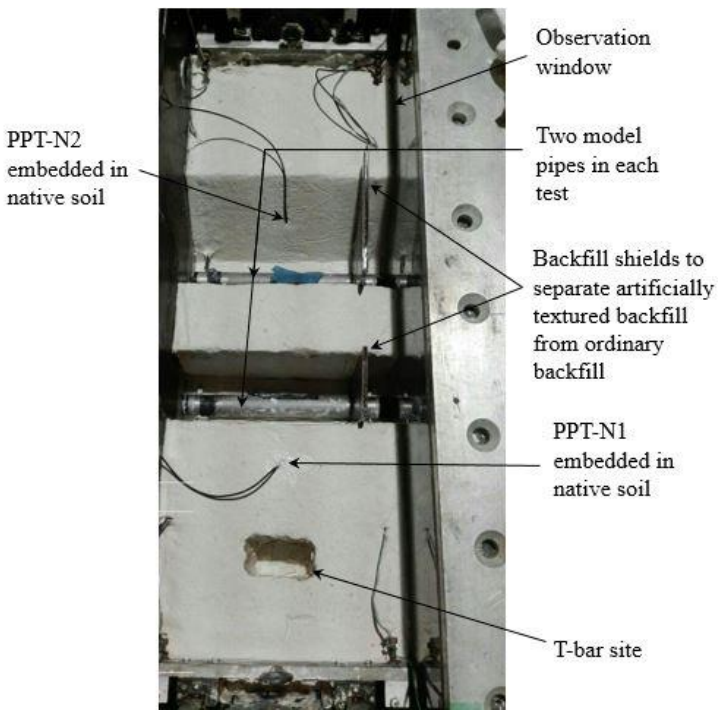

Figure 3-4. Top view of instrumented box before backfilling

3.5.3 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 3-2 shows the summary of the backfilling material prepared and tested in this study.

Table 3-2. Soil properties of cohesive testbed

Test	Pipe	Test name	Trench backfill type	Trench backfill ID	T-bar site backfill	T-bar site backfill S_u (kPa)	Native S_u at pipe depth (kPa)	Native soil water content after cons (%)	Native water content after test at pipe depth (%)	Native soil void ratio	Saturated unit weight γ_{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							

3.5.4 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). Figure 3-5 shows a top view of the backfilled soil sample.

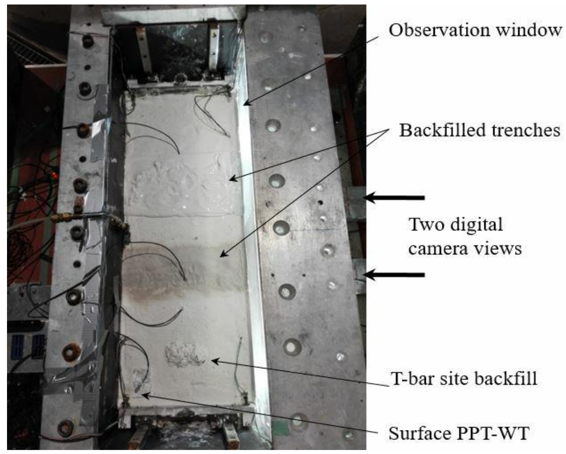

Figure 3-5. Top view of the instrumented box after backfilling

3.5.5 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 ploughing, backhoe and clamshell bucket. Four different chunky material with different stress history were produced and tested in this program.

3.5.6 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.

3.6 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	Non-vented PPT with flexible cable	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 gage	On pipe at the reduced section. 2-half bridge pattern (1 full Wheatstone bridge)	Shear gage 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	Frasier river sand	NA
End caps & O-ring	The end of the pipes	Nylon	2 per pipe

One internal (non-vented PPT with flexible cable) and four external (Druck PDCR81) 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. 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 gages were installed in the reduced cross-section of the pipes to capture the lateral pipe response. The strain gages 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.

Figure.3- 6 Shear strain gage installed at reduced section

3.7 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) where the locations of interest or subsets 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 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 fisheye 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.

3.8 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.

3.9 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, embedment 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.

3.9.1 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 majors 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. Figure 3-7 shows a sample of p-y responses obtained for different backfilling material.

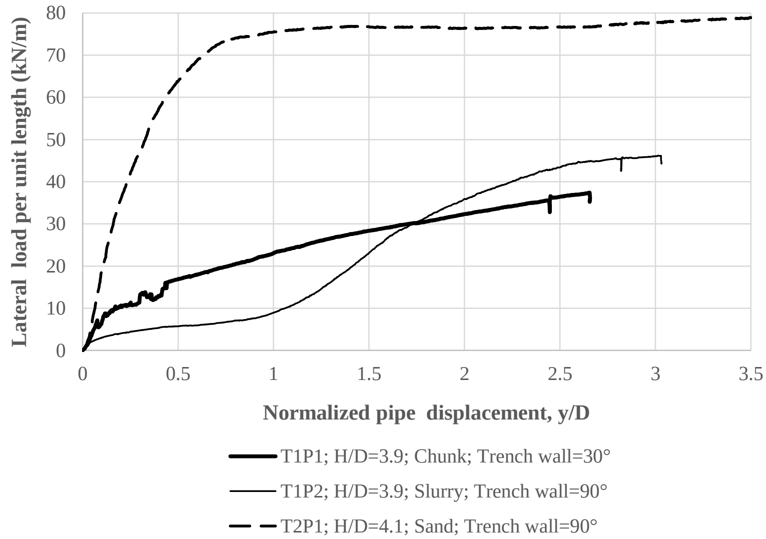

Figure. 3- 7 Effect of backfill type on force-displacement response

As earlier shown in, 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. Figure 3-7 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 Figure 3-7 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.

3.9.2 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 Figure 3-8 and Figure 3-9 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 embedment ratios. 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 embedment, 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).

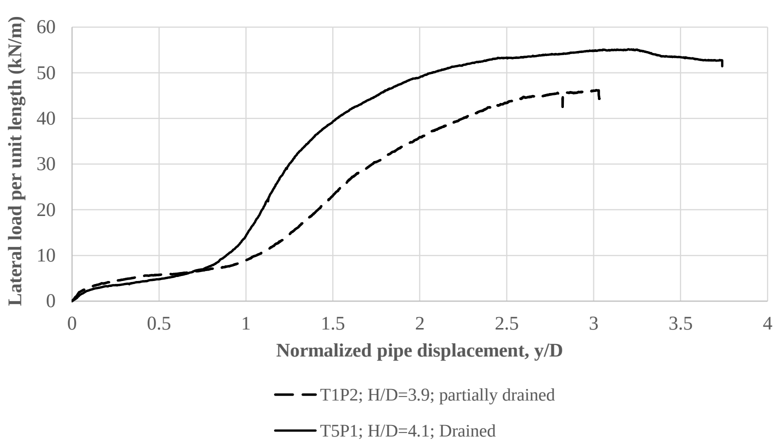

Figure 3- 8. Interaction rate effect on prototype force-displacement response (deep burial**)**

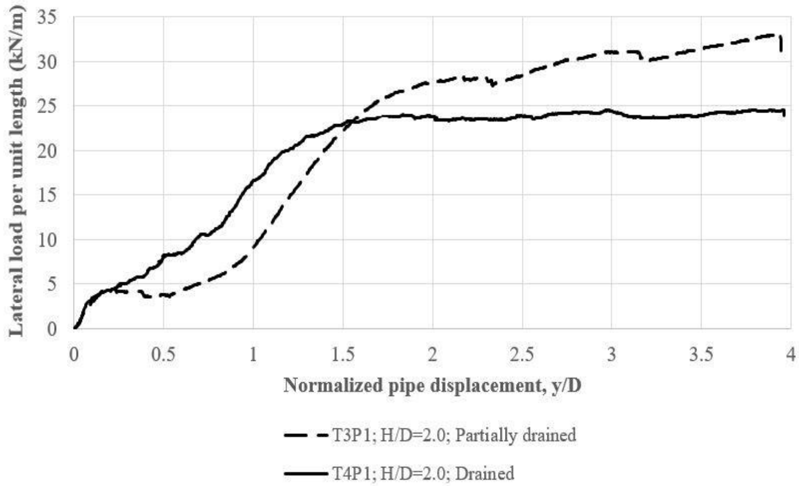

Figure 3-9. Interaction rate effect on prototype force-displacement response (shallow burial)

Figure 3-10 and Figure 3-11 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 embedment ratio and interaction rate. However, it is much different in native soil, where the pore pressure dissipation depends on both embedment ratio and interaction rates. Figure 3-10 shows that the pore pressure increases over the course of 0.25D penetration of the pipe into the trench wall in deeper embedment case. The pore pressure is then continuously decreased in all cases, while the dissipation rate is different depending on embedment ratio and interaction rates. Corresponding to the lateral responses discussed above in Figure 3-11, the ultimate pore pressure in drained deeply buried pipe test is much lower than the shallowly buried pipe. Also, the results show that the pore pressure arrives at a low ultimate state in the shallowly embedded pipe.

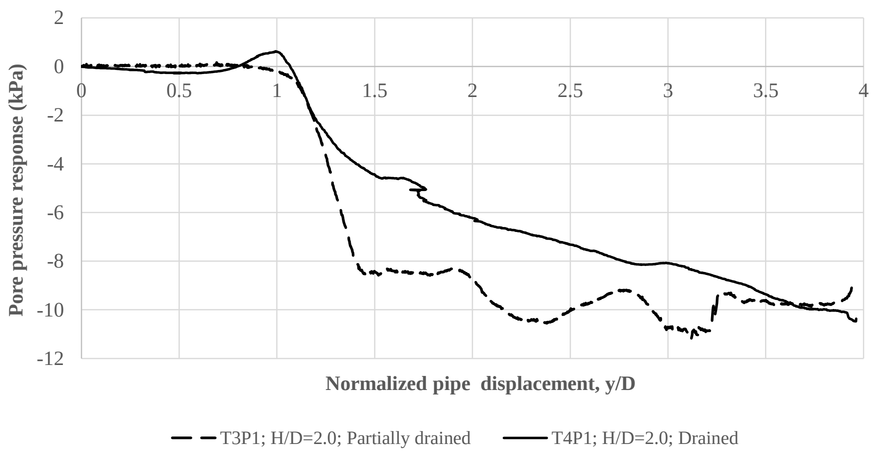

Figure 3-10. PPT-N1 responses to pipe displacement

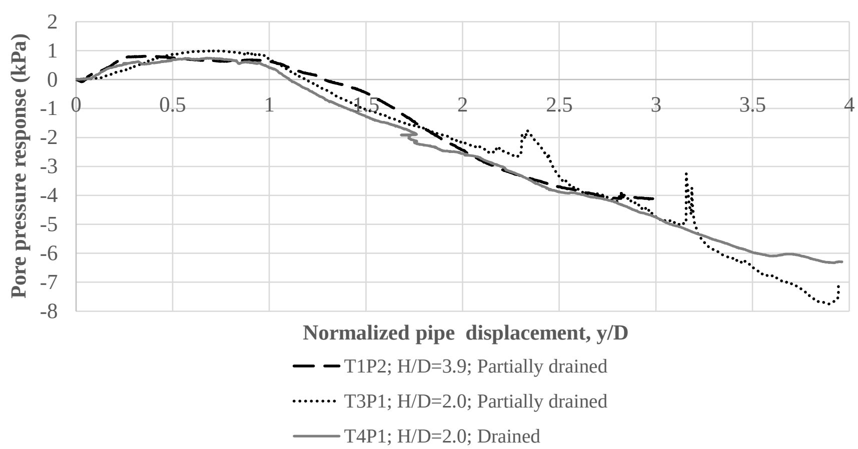

Figure 3-11. Internal PPT responses to pipe displacement

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.

3.10 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 Figure 3-12. 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 Figure 3-12. The slope of the pipeline force-displacement response has achieved its maximum value in the range of 1.0D to 1.5D.

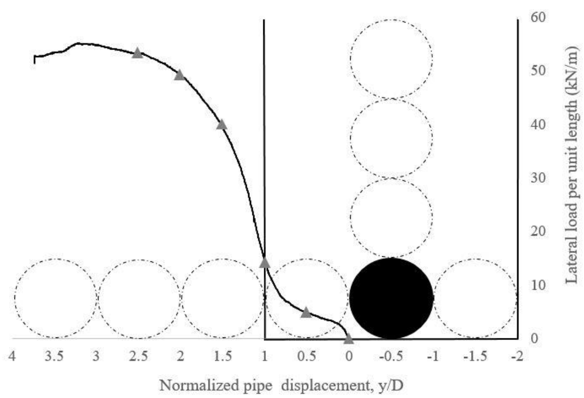

Figure 3-12. Force-displacement of T5P1 in the schematic trench; PIV intervals are marked by triangle

Figure 3-13 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.

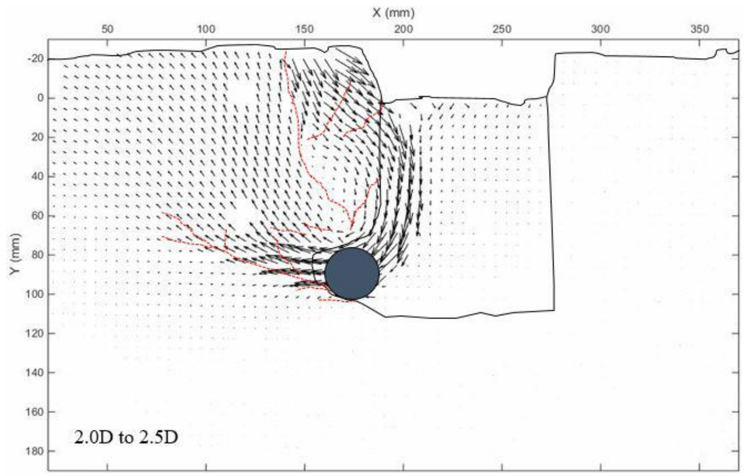

Figure 3-13. Vectorial displacement for pipe movement from 2.0D to 2.5D

The progressive stages of soil deformation by 0.5D intervals are illustrated in Figure 3-14. 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.

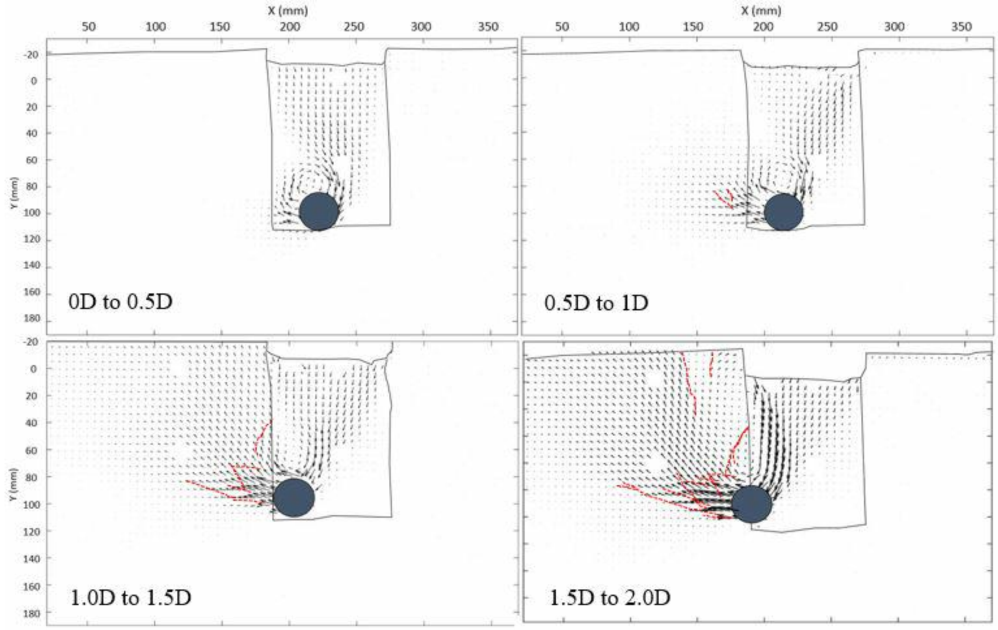

Figure 3-14. Vectorial displacement fields during lateral displacement up to 2.0D pipe movement

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.

3.11 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, embedment depth, and interaction rate effects. Full instrumentation was applied to pipes including strain gages, 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.

3.12 Acknowledgement

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CHAPTER 4

EXPERIMENTAL STUDY OF THE TRENCH EFFECT AND SAND BACKFILL ON LATERAL PIPELINE-BACKFILL-TRENCH INTERACTION AND THE RESULTANT FAILURE MECHANISMS

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This chapter has been submitted as a journal paper. The paper was co-authored by another PhD student that supported the PIV analysis. The contribution of the candidate in this paper is more than 80%.

4.1 Abstract

Subsea pipelines may experience large lateral displacements due to ground movement, landslides, ice scour, operational loads, etc. Pipelines are often buried by subsea trenching and backfilling for physical protection against these kinds of lateral displacements. The sand backfills are sometimes used for burial of the trenched pipelines. This backfilling condition is different from cohesive backfills, where due to environmental, constructional, and operational loads, the backfilling material is significantly remolded and become much softer than native ground. Although, the stiffness of the sand backfill is different both from the cohesive backfill and the native ground. The analytical and empirical solutions currently recommended by design standards do not account for the effect of trenching due to its less explored effect on lateral soil failure mechanisms. In this study, the effects of slurry and sand material backfilling in deep trenching on lateral pipeline-backfill-trench interaction were experimentally investigated by conducting centrifuge model tests. Transparent observation windows equipped with digital cameras and state-of-the-art instrumentation were used to directly monitor the soil deformations and conduct particle image velocimetry (PIV) analysis. Several significantly important mechanisms were observed, and a couple of new research avenues were identified that has never been addressed in the past. The study provided an excellent insight into the trench effect on soil resistance against the lateral pipeline displacements.

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

4.2 Introduction

Subsea pipelines are one of the key components of developing offshore oil and gas fields. These important elements may be also used for transferring the water supply crossing the lake and rivers. Subsea pipelines may experience large lateral displacements under the impact of the ground movement, ice gouging, drag anchors, etc. Pipelines are usually buried by trenching and backfilling to reduce the effect of environmental and operational loads. Depending on the construction process and the environmental loads, the backfill material may be remoulded to different extents and become much softer than the native ground (M. Paulin et al. 2014).

The different stiffness between the backfill and native material significantly affect the total mobilized lateral soil resistance against the moving pipe. However, the interaction mechanisms between the pipeline, backfill, and the native ground (trench walls) have not been sufficiently explored and implemented by design standards (e.g., ASCE-ALA). Sometime the design code recommends to assuming a wide trench to make sure the pipeline lateral response will depend only on the properties of the controlled backfill material (PRCI 2009). Figure 4-1 shows the interaction event that may happen depending on the relative backfill/native soil stiffness.

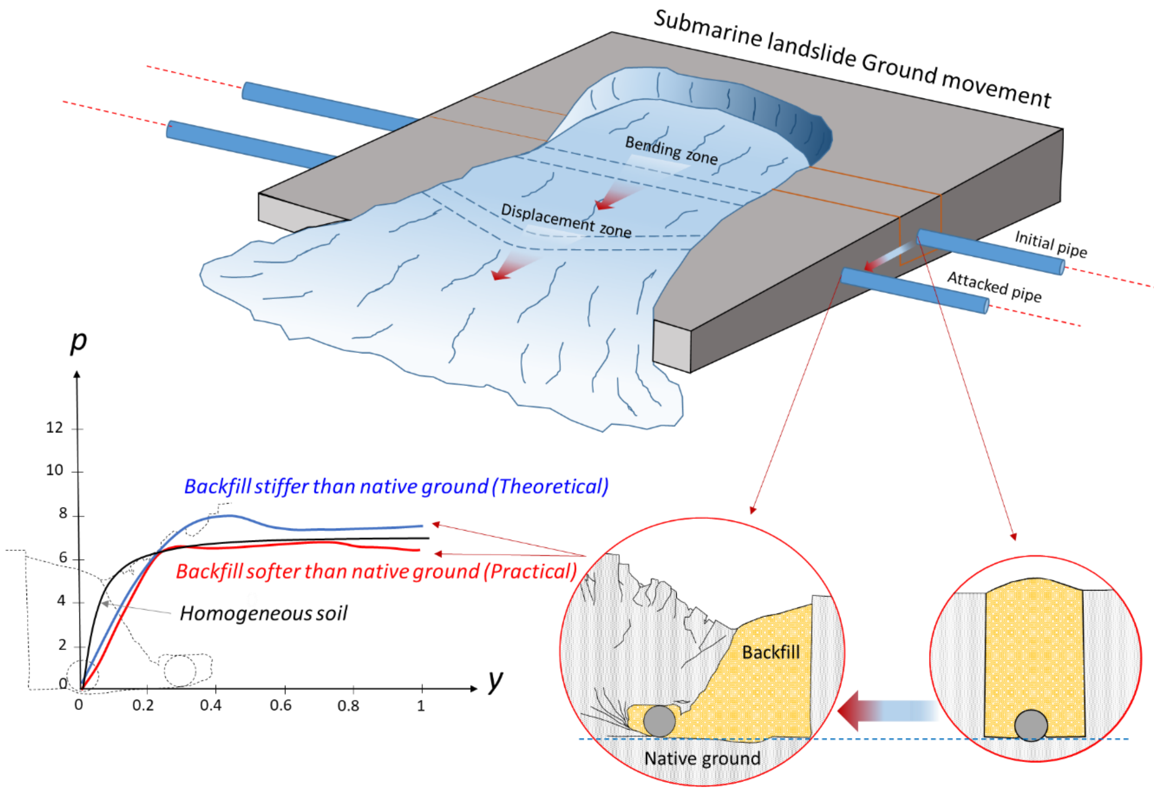


Figure 4-1: Lateral response of trenched and backfill pipeline to subsea geohazards

Paulin (1998) comprehensively investigated the trenching and backfilling effect on large lateral pipe-soil interaction process in clay by performing experimental study. A wide range of parameters were investigated 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 directly observe the lateral pipeline-backfill-trench interaction mechanisms for more accurate assessments.

In this study, the succsessive pipeline-backfill-trench interaction were directly recorded and analyzed by applying particle image velocimetry (PIV) in the centrifuge facilities at C-CORE. A complete set of instrumentations were used to closely monitor the interaction mechanisms. The effect of the trench on lateral pipe response and the corresponding failure mechanisms were investigated both in sand and slurry backfill materials.

The force-displacement (p-y) curves were obtained and compared with the PIV analysis results throughout a large pipeline displacement (about 4D). It was observed that the pipeline-backfill-trench interaction mechanisms completely governs the effect of pure backfill and native soil strengths. The study showed several important mechanisms that has never been investigated in the past. Exploring these new areas is expected to significantly improve the safety and the cost-effectiveness of the current practice in the near future.

4.3 Test setup configuration

The tests were conducted at C-CORE centrifuge facilities located at the St. John’s campus of the Memorial University of Newfoundland. Sand and very soft slurry backfills were used in (T2P1, H/D = 3.60) and (T5P1, H/D = 3.70) rectangle trenches via partially drained condition (Normalized velocity, vD/c_v = 0.14, based on Phillips et al. (2004)).

The test setup was designed similar to Paulin et al. (1996-1998), Popescu et al. (1999), and Konuk et al. (1999) for better comparison with earlier studies. The significant advantage of the current test set up compared to the earlier studies was the using of the transparent observation window and PIV analysis that enabled direct capturing of failure mechanisms and soil displacements beside the lateral p-y responses. A prototype pipe of 24” with an external diameter of 610 mm was selected. This was in continuation to the earlier full-scale studies in sand conducted by Burnet (2015) at Queens University. The spinning acceleration was set on 19.1g to meet the other specifications. The dimensions of the strong box was (900 × 400 × 300 mm, L × H × B). T-bar penetrometer was used to capture the soil strength profile inflight. The full details can be found in Kianian et al. (2018).

Table 4.1 Summary of conducted experiments

Test ID	Embedment 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
T2P1	3.60	Sand	Vertical	9.09	0.42	3.60D

The test apparatus was designed to conduct two separate tests at the same time. Figure 4-2 shows a schematic view through the transparent window. The model pipes were backfilled inside the excavated trenches in a pre-consolidated soil bed.

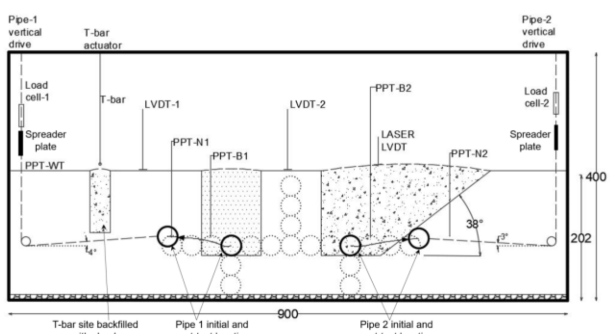


Figure 4-2: Sample schematic view of test setup and instrumentations

To prepare the native ground, Speswhite kaolin clay and Sil-Co-Sil silt were mixed by 50%-50% in weight and sufficient amount of water to form a slurry with a nominal moisture content of about 70%. The native soil bed was consolidated to the effective stress of 400 kPa and then was incrementally unloaded to 100 kPa with an open drainage valve. During the unloading of the soil sample down to 100 kPa, the water flow 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 intermediate undrained shear strength of 15 to 25 kPa which is quite common in Canadian offshore region.

Trenches were excavated using a blade with adjustable side angle that was mounted on a guide beam sitting on the strong box. A trench width of about 3D was considered. The burial 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 there will 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	17.5	32.04 - 32.97	18.33
T2P1	Sand	----	----	16.0	30.81 - 31.11	18.56

The model pipe size was fabricated from stainless steel pipe (31.75 mm) 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) .

Three parallel data acquisition systems (each has 8 individually configurable inputs) were used with various instruments for full monitoring of the testing program such as pore pressure transducers (PPTs), strain gauges, load cells, conventional and riser linear variable differential transformers (LVDTs), T-bar, vertical drive motion controller, digital cameras, markers and artificial textures. 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 purely capture the effect of pipeline-backfill-trench interaction. Further details of the test set up preparation can be found in Kianian et al. (2018).

The soil strength profile was obtained by using an inflight T-bar penetrometer. Figure 4-3 shows the undrained shear strength profile for all of the conducted tests outlined in Table 4-3. 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.

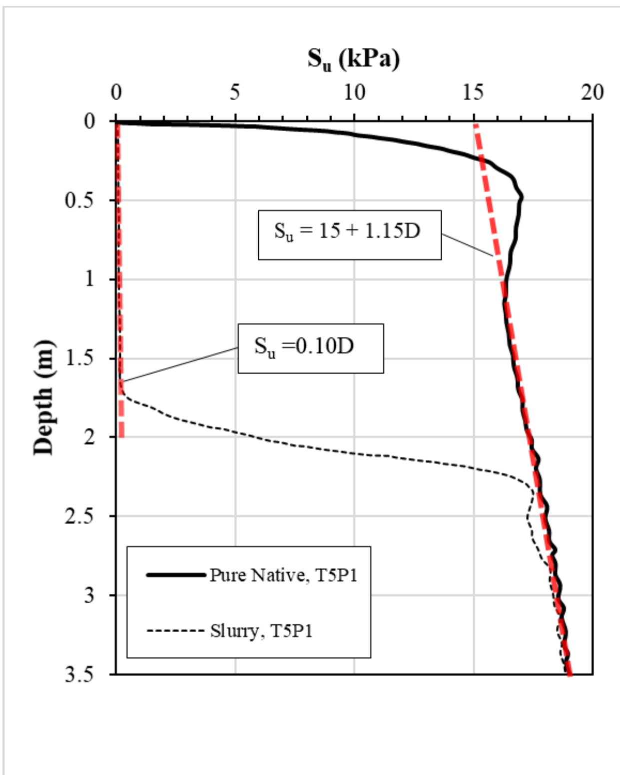


Figure 4-3: Undrained shear strength profiles and linear curve fits

Linear S_u profiles were fitted both for backfill and native soils as shown in Figure 4-3. The undrained shear strength in slurry backfills is almost negligible. The native soil located underneath the backfill material showed a slightly softer response in initial stages of penetration. This is due to slightly water dissipation from 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. 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	0.00	0.10

4.4 Test Results

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 then presented in the next section for different stages of pipeline-backfill-trench interaction to compare the internal soil deformations and failure mechanisms with the obtained responses. Figure 4-4 shows the force-displacement responses against the normalized lateral displacement (y/D) of the conducted tests.

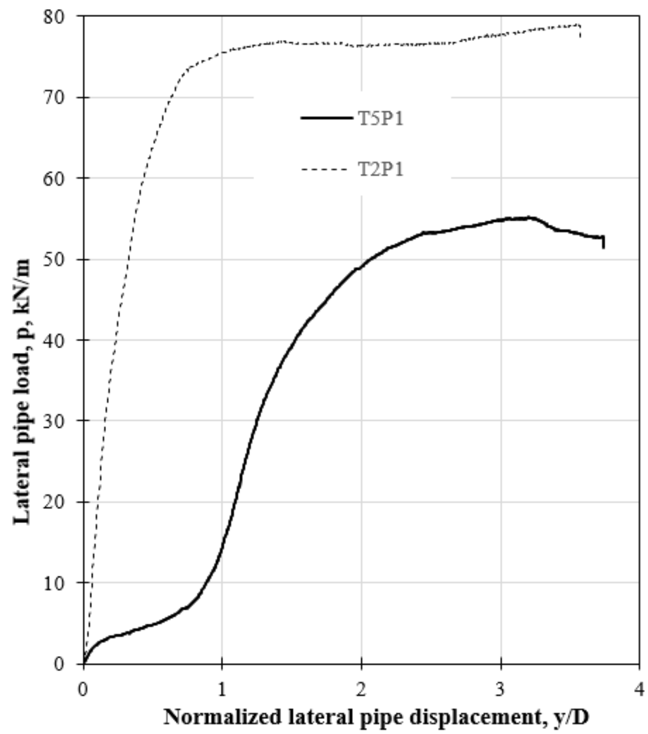
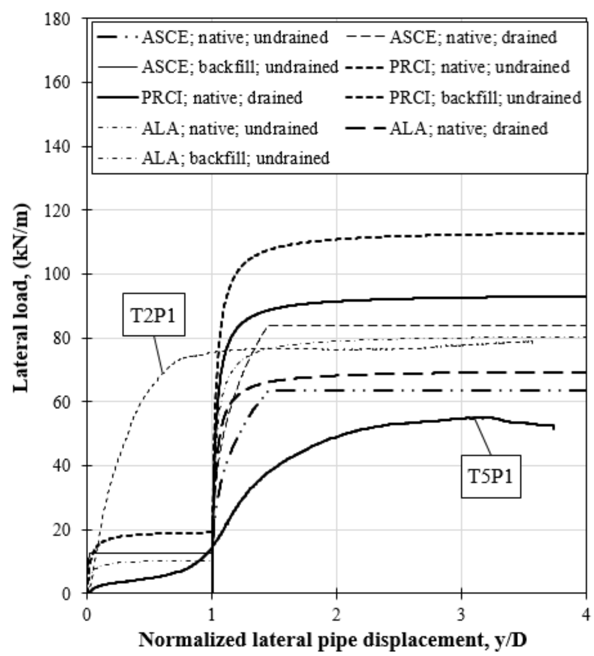


Figure 4-4. The lateral load-displacement response against the displacement

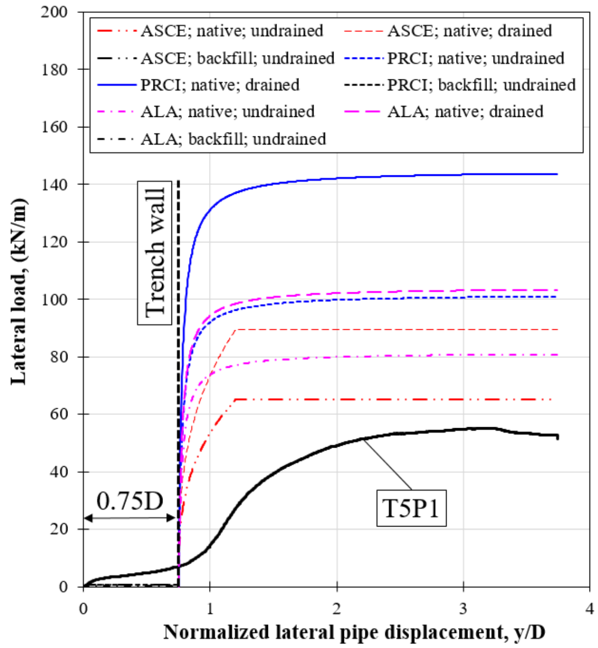
When the pipe starts to move in slurry backfilled test, the load is slightly increased with a relatively high stiffness at the beginning and continued by a softer response. By getting closer to the trench wall (native ground), the response becomes stiffer, and the load is rising up with a steep transition slope, which is getting more inclined with further penetration into the native ground. For the test with sand backfill, the load is steeply increased from the beginning to a high ultimate load. This shows effective transferring the load by sand to 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 embedment, the larger the lateral resistance, as reported by the studies conducted by Paulin (1998), Altaee and Boivin (1995), and Karal et al. (1983).

Also, Figure 4-4 shows a lateral load of about 5 kN/m for the pipe inside the slurry, which is much larger than what is expected. Since the slurry has an extremely low strength and perfect lubrication was applied between the pipe end caps and the test box walls, no considerable load is expected while the pipe is moving inside the backfill. The PIV results showed that the source of this load mobilization is pipe-trench bed interaction, which affects the lateral soil resistance in larger pipe displacements. Further investigations are needed in this area for improvement of the lateral response of trenched/backfilled pipelines.

Figure 4-5 compare the test results with the p-y curves predicted by the existing design codes (i.e., PRCI, ALA, and ASCE). 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 4-3, and the drained parameters were adopted from the triaxial tests (Paulin (1998)).



(a)



(b)

Figure 4-5. The comparison of the p-y responses between the test results and design codes

The results presented in Figure 4-5 show that the design codes overestimate the ultimate load for a pipe penetrating into the trench wall and underestimate the lateral load for the pipe moving inside the trench. This large difference is due to the significant effect of the trench presence that largely releases the passive pressure against the collapsing trench wall and is not considered by design codes because of less explored soil deformation mechanism. Also, the design codes underestimate the lateral load for the pipe approaching the trench wall, which is an important aspect and needs improvements to come up with a more conservative design strategy.

Overall, the design codes and the plasticity solutions that consider homogeneous soil strata and ignore the highly different stiffness between the backfill and the native soil underestimate the lateral load inside the trench and in the transition zone and overestimates the ultimate response.

A deep understanding of the source of these deviations needs an accurate investigation of the soil deformation and failure mechanisms that will be done in the coming sections.

Figure 4-6 show the variation of pore pressure against the pipe displacement in 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 4-2. The variation trends in internal PPTs indicate an initial increasing of the pore pressure followed by dissipation of the excess pore pressure and develop 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.

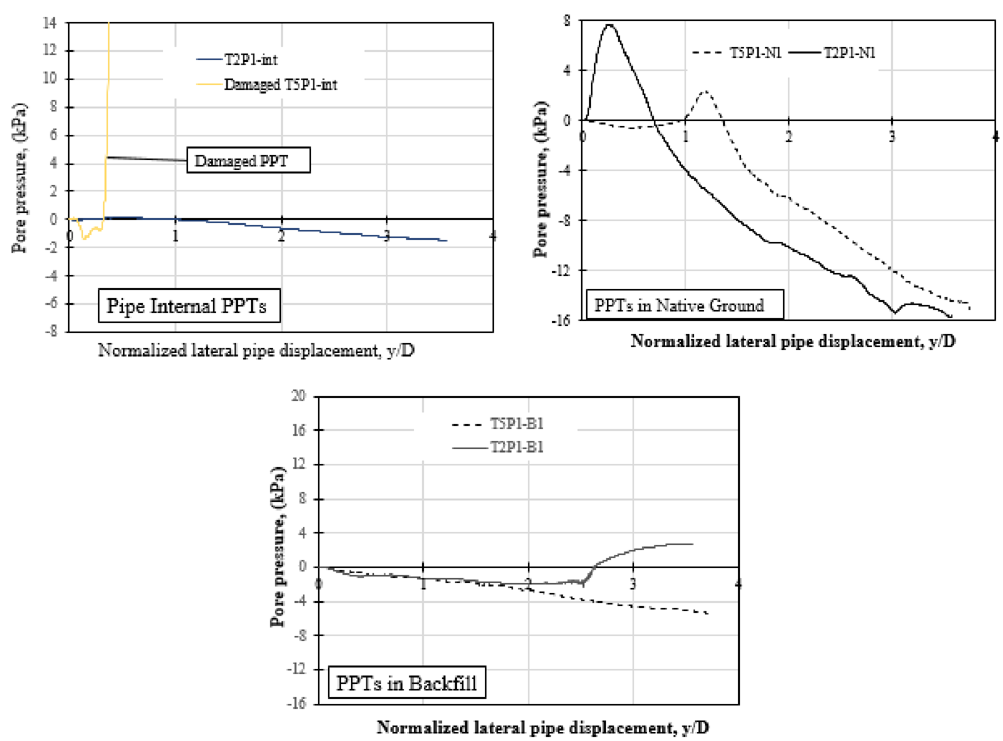


Figure 4-6. Variation of pore pressure in backfill, native ground, and the rear of the pipe

In the native ground in front of the moving pipe, after a slight decrease and then increase, the excess pore pressure continues to dissipate with time and are slightly affected by the pipe interaction with the trench wall.

4.5 PIV Results

The pipeline displacement was divided to three different assessment zones (I, II, and III) shown in Figure 4-7 based on changing the key soil displacement mechanisms that will be explained in coming sections.

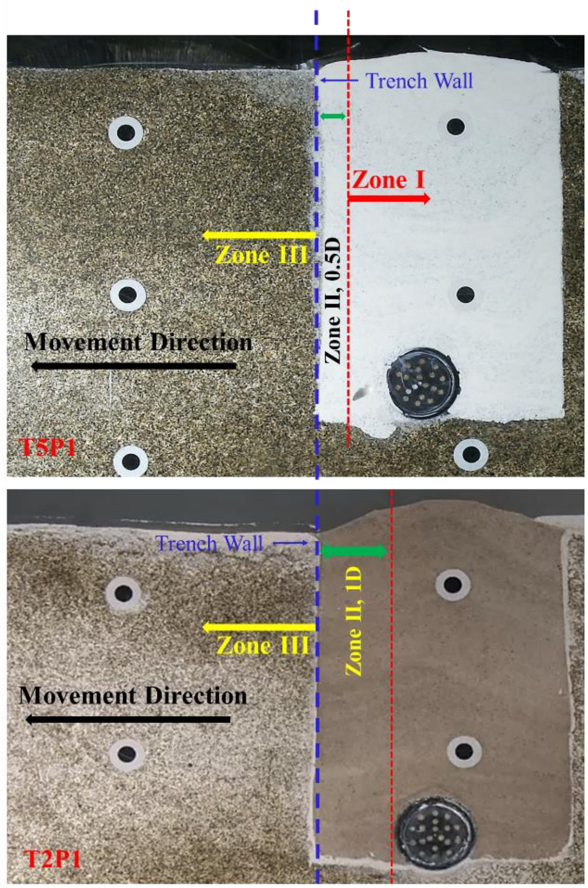


Figure 4-7. Observation zones based on key soil displacement mechanisms

Two main mechanisms were observed in Zone I: i) pipeline-backfill interaction ii) pipeline-bed interaction. Figure 4-8 shows samples of the PIV analysis in Zone I.

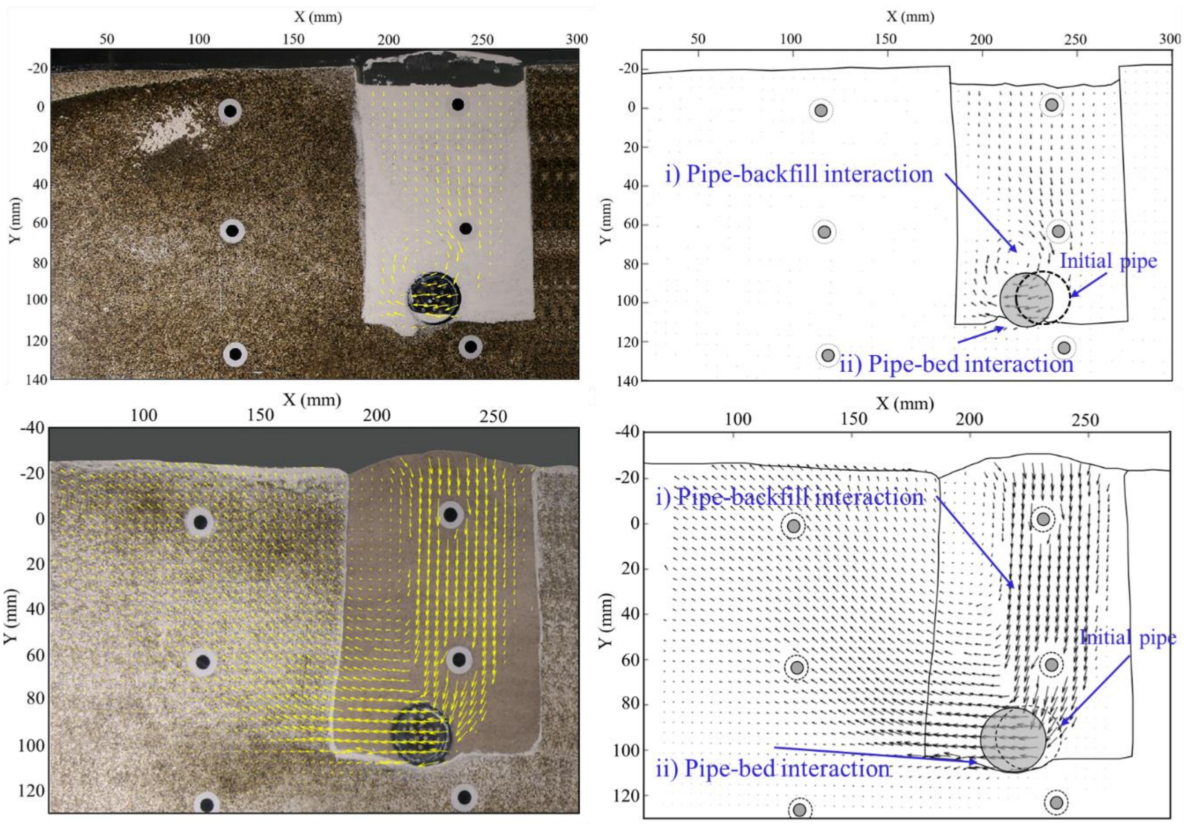


Figure 4-8. Sample PIV analysis results in Zone I, (~ 0.25D pipe displacement)

A close investigation of recorded videos and PIV results shows that the pipeline-backfill interaction (i) comprises loops of eccentric spiral failures with rotational circles around the moving pipe. These spiral failure surfaces emanate from a point above the pipe and horizontally move with a pipe until the failure surface touches the trench wall. From this stage, with further displacement of the pipe towards the trench wall, the spiral failure starts to contract with a varying ratio that depends on its distance to the wall; closer the wall, smaller the failure circle. A closer investigation of the recorded videos and PIV results showed a second mechanism that is significantly important in the assessment of the lateral soil resistance. This mechanism is a result of interaction between the pipeline and the trench bed, simply referred to as pipe-bed interaction. The pipe section slightly penetrates into the trench bed during the inflight consolidation due to pipe weight and the bearing stress. This initial embedment results in the creation of small soil berms in front and rear of the pipe which the size of that vary in each test. Due to a minor penetration of the slurry backfill into the native soil around the internal surface of the trench, these small soil berms are barely seen in the tests, but the recorded videos and PIV analysis confirm their existence and significant contribution as logically expected. As pipeline moves laterally, the front berm is successively developed pushing the pipeline upward into the backfill that has a lower strength, which that's not too tangible in T2P1 test. The upward movement is accelerated as the pipe further approaches the trench wall, where the front berm is stuck between the pipe and trench wall and is compressed to the trench corner. In addition, the squeezed soil berm that is stiffer than the backfill intervenes and stops the rotational failure in front of the pipe, which is considered to be the starting point of the Zone II. Considering the low magnitude of the shear strength in slurry backfill in T5P1, this second mechanism is the main contributor to the p-y curves in the Zone I. The resistance in T2P1 starts earlier and achieves a very higher value compared to T5P1.

Entering into Zone II, two important effects initiated in Zone I influences the soil resistance. First, the developed soil berm squeezed into the trench corner pushes the pipeline upward and results in an oblique penetration into the trench wall. Second, the squeezed soil berm intervenes and stops the rotational soil failure in front of the pipe due to its higher stiffness compared to the backfilling soil. This mechanism converts the pipe diameter to act like a virtual larger pipe section penetrating into the trench wall and affect the embedment ratio and failure mechanism in later stages of lateral pipe movement (see Figure 4-9).

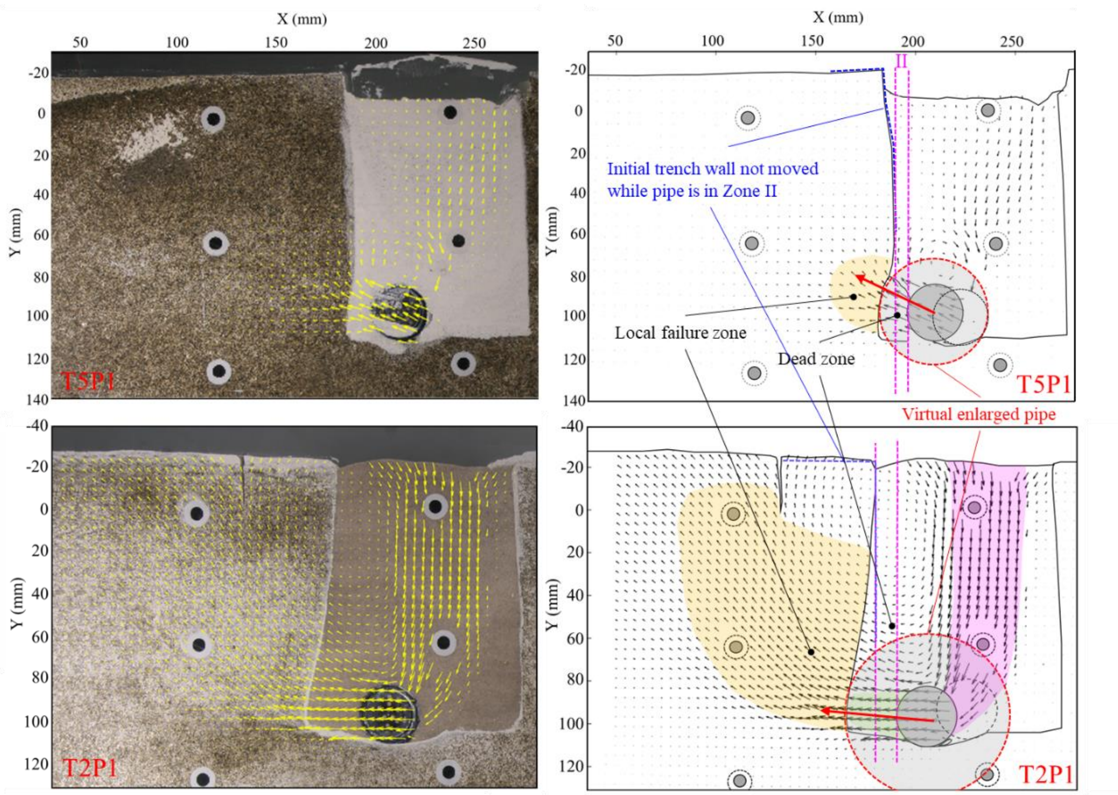


Figure 4-9. Different soil displacements in Zone II

As mentioned earlier, in practice, the probability of pipeline falling into Zone II is higher than Zone III, where pipeline may go under 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 4-20), and the area still needs deep investigations.

By approaching the Zone III, where the pipe front arrives at the initial trench wall location, a small triangular wedge is created in front of the pipe, while the first appeared logarithmic spiral shear band is faster developed under the pipe (see Figure 4-10). The observed isosceles triangle, which is similar to Terzhaghi's active zone under a footing, has different size and direction in trench (T2P1) and trench (T5P1) and follows a different progression scheme as well. The wedge impact region in trench T2P1, which is larger than the trench T5P1, is surrounded by spiral shear band underneath the wedge. In the T5P1 trench, the active wedge is completely separated from the spiral shear band and is smaller compared to the shallow trench.

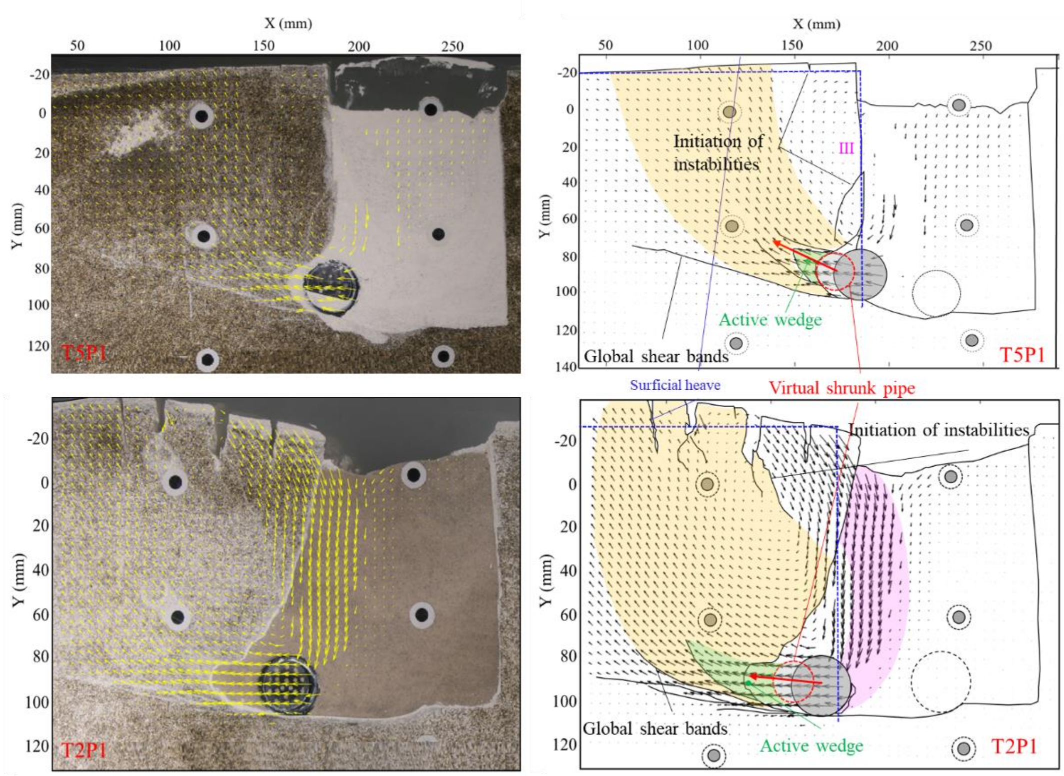


Figure 4-10. Trench deformations at the end of Zone III

A series of total plastic strain variation throughout the Zone I, II, and III along with the observed deformations is presented in Figure 4-11 to have 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.

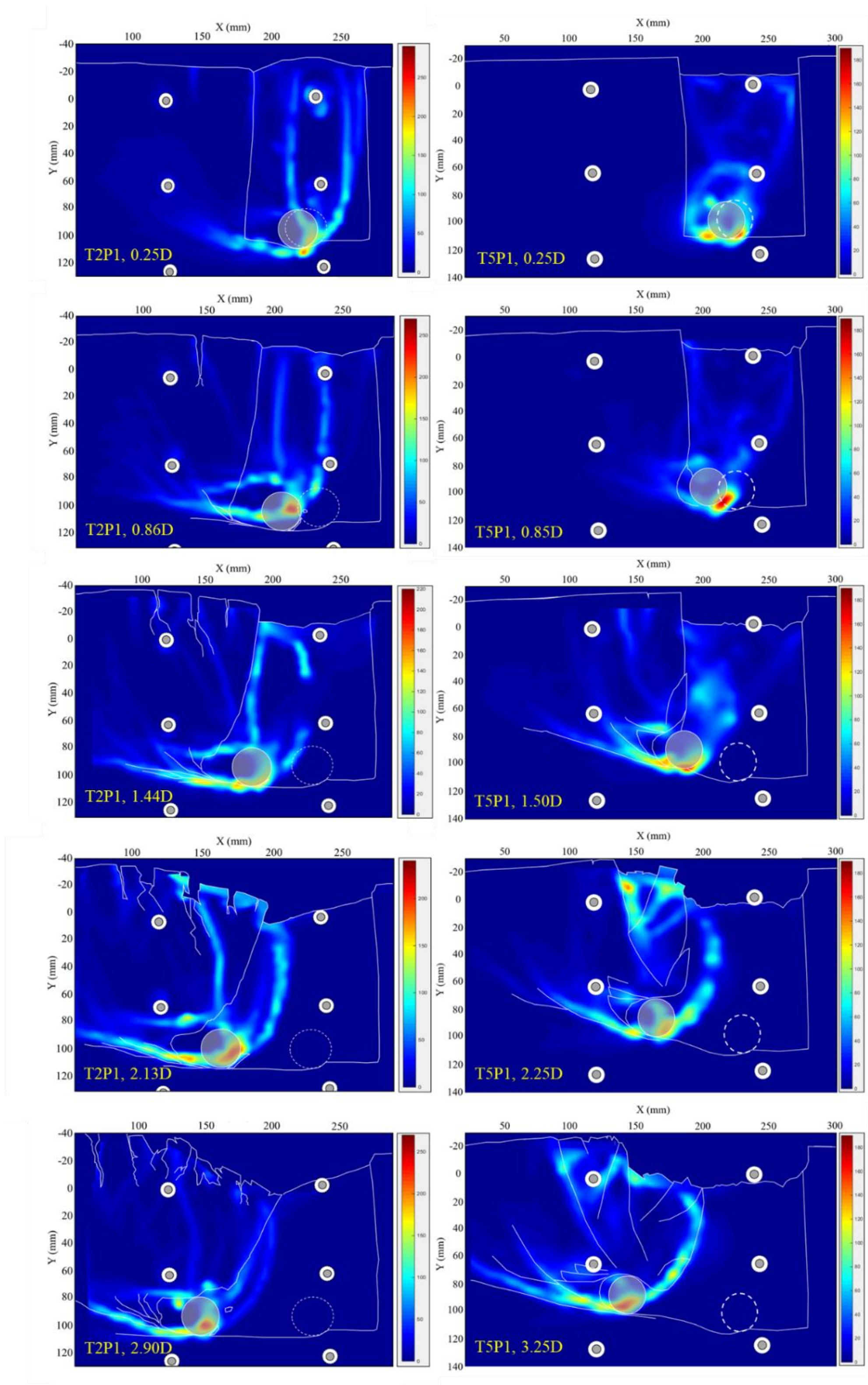


Figure 4-11. Total plastic strains from PIV analysis in the Zone I, II, III

4.6 Conclusions

Experimental study was conducted by using C-CORE centrifuge facilities to investigate the effect of different backfill materials ratio on large lateral soil deformations and failure mechanisms around the trenched/backfilled pipelines. PIV analysis were used to capture high-quality images and analyze the internal soil deformations and failure mechanisms in both backfill and native trench wall. Several significantly important aspects were observed:

- The trenching reduces the ultimate lateral soil resistance against the pipe approaching/penetrating to the trench wall due to the progressive collapse of the trench wall into the backfill. The magnitude of reduction may vary depending on the stiffness of the backfill and the amount of passive lateral pressure that the backfill material mobilizes against the active trench collapse.
- The pipeline-trench bed interaction, including the magnitude of the initial pipe embedment into the trench bed and the lateral failure mode of partially embedded pipe 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 never been addressed or investigated in the past and need comprehensive investigations. This mechanism and squeezing of the trench bed material into the trench corner causes the pipe to move upward and enter the trench wall in an obliqued direction.

These observations shows the influence of several parameters on lateral soil resistance against the largely displaced pipeline that needs further investigations such as the effect of pipe weight, pipe type, deep burial effect, backfill buoyancy, trenching and backfilling methodology, construction procedure, construction season, operational loads, thaw settlement and permafrost, longitudinal seabed profile, etc.

4.7 Acknowledgments

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CHAPTER 5

TRENCH IMPACT ON LATERAL RESPONSE OF PIPELINE BURIED IN SAND

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5.1 Abstract

Subsea pipelines may go under large lateral displacements due to ground movement and ice gouging etc. In practice, the backfilling material is significantly interacting with the pipeline and trench wall affecting the lateral response of the pipeline. The pipeline-backfill-trench interaction is not usually considered in design practice and has not been deeply explored in the literature. This paper presents the numerical modeling of centrifuge tests conducted at C-CORE to investigate the lateral response of a trenched pipeline backfilled with sand. The native soil bed in which the trench had been excavated was over-consolidated clay and also pure loose sand. Coupled-Eulerian-Lagrangian

(CEL) analysis was performed using ABAQUS/Explicit to model the pipeline, trench, and backfill. A parametric study was conducted to investigate the influence of various parameters including the burial depth, and trench geometry on the lateral force-displacement (p-y) response of the pipeline. The results showed that the lateral p-y response of the pipeline is significantly affected by interactive failure mechanisms of the backfilling material and trenched native soil.

RÉSUMÉ

En pratique, le matériau de remblayage interagit de manière significative avec le pipeline et la paroi de la tranchée, ce qui affecte la réponse latérale du pipeline. L'interaction pipeline-remblai-tranchée n'est généralement pas considérée dans la pratique de conception et n'a pas été explorée en profondeur dans la littérature. Cet article présente la modélisation numérique des essais de centrifugation effectués à C-CORE pour étudier la réponse latérale d'une tranchée de tranchée remplie de sable. Le lit de sol indigène dans lequel la tranchée avait été creusée était de l'argile sur-consolidée et aussi du sable meuble pur. L'analyse Coupled-Eulerian-Lagrangian (CEL) a été réalisée en utilisant ABAQUS / Explicit pour modéliser le pipeline, la tranchée et le remblai. Une étude paramétrique a été menée pour étudier l'influence de divers paramètres, y compris la profondeur de l'enfouissement, et la géométrie de la tranchée sur la réponse latérale force-déplacement (p-y) du pipeline. Les résultats ont montré que la réponse p-y latérale du pipeline est significativement affectée par les mécanismes de rupture interactifs du matériau de remblayage et du sol natif de la tranchée.

5.2 Introduction

Trenching is one of the most practical physical protection methods for subsea pipeline transporting oil and gas. Lateral displacement of pipeline can be caused by ground movement, ice gouging etc. and consequently it is necessary to examine the force induced by the trench-backfill-pipeline interaction for the sake of the integrity of the pipeline. Experimental and numerical studies can be found in the literature with focus on the lateral displacement of a buried pipeline and the interaction between pipeline and backfilling material. But effects of backfilling material properties, trench geometry, and interaction rate have not been systematically examined before. Considering various backfilling materials used in practice, current design guidelines such as ALA-ASCE (2001), ASCE (1984), PRCI (2009, 2004) and O'Rourke and Liu (2012, 2010) do not make available specific recommendations with attention to the appropriate trench dimensions. Also, to estimate the ultimate soil reaction pressures, available methods do not take the effects of trench dimensions into accounts (Trautmann & O'Rourke 1985). To fill the knowledge gap and fully examine the trench-backfill-pipeline interaction and the resultant p-y response of the pipeline during large lateral displacement, a series of research work has been done. This paper specifically Focused on the experimental and numerical studies on trench-backfill-pipeline interaction that has been examined and presented with loose sand backfilled in the vertical trench excavated on native ground.

The centrifuge experiments were used to explore the pipeline loading in the mixed soil. To examine the soil interaction and the pipeline strains, the trench is backfilled with loose to medium dense sand in the state of permanent ground displacements and stiff natural soil conditions. An advanced numerical model was also developed for comparison with experimental tests and will be further calibrated using the test results.

5.3 Literature review

Force-displacement response of pipelines in lateral pipe-soil interactions has been widely explored. But studies that specifically focus on trench dimension effects and failure mechanisms during the large displacement of pipelines are very limited. Phillips et al. (2004) examined the trench effects using numerical models (discrete nonlinear springs for cohesive soil around pipeline) and a centrifuge model (under an acceleration of 50 g). The results showed that the existence of a trench and 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. It should be noted that in deep embedment conditions and under large relative displacement, the kinematic mechanism changes from a global-type failure to local shear soil failure (Yimsiri & Soga & Yoshizaki & Dasari & O'Rourke 2004).

Based on this literature review, there is not an adequate number of experimental and theoretical models in the literature to speculate the (p-y) and ultimate lateral resistance curve for pipelines. Most of the present models were based on anchor plates (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; Merified et al. 2001). A large number of other solutions were proposed on the basis of the piles (Hansen 1948, Poulos 1995, Hansen and Christensen 1961, Matlock 1970, ALA 2005, Welch 1975, Reese and

Bhushan et al. 1979, Edgers and Karlsrud 1982, Klar and Randolph 2008). Only a few models were developed on the basis of the lateral interaction of pipelines (Oliveira et al. 2010, Poorooshasb et al. 1994, Paulin 1998). Paulin (1998) conducted a group of lateral pipeline-soil interaction centrifuge tests (under an acceleration of 50 g) to investigate the impacts of trench effects as one of the primaries that thoroughly investigates small-scale studies on the lateral response of completely buried pipelines in clay (Kianian M, Esmaeilzadeh M & Shiri H 2018). It was discovered that trench width had negligible impact on an undrained interaction, whereas as the burial depth increases the undrained load on the pipeline will increase. The authors concluded that the transferred load from soil to pipeline significantly affected by displacement rate of the pipeline. But the failure mechanism was qualitatively explained and there is no direct visualization data. The authors stated that the overall normalized interaction between the soil and pipeline may be influenced by backfill properties. However, they could not ascertain if this is caused by a change in the separation condition behind the pipe or a change in failure mechanism. To better examine the trench effects and present the failure mechanism during the large displacement of the pipeline, the authors developed a series of experimental tests with a full set of monitoring and state-of-the-art equipment utilized on the backfill, pipeline, actuation system, native soil and whole test configuration. The authors used a digital camera, transparent acrylic sheet and particle image velocimetry (PIV) to attain interactive and progressive failure mechanisms. Furthermore, an advanced numerical model was developed and will be further calibrated according to the experimental results. Altogether, this study increased the current comprehension knowledge of the lateral response of entirely buried pipes to large deformations and offered a complete understanding into this important critical problem. Ongoing tests and simulations will further explore the effects of interaction rate. In real pipe-soil interaction circumstances both drained and partially drained states are very frequent. In these conditions the rate of relative displacement between soil and the pipeline is moderate. In such instance, during the displacement the soil surrounding the pipeline reaches some degree of consolidation. Besides, in many geographical locations, silt fragment is found in soft natural offshore clays (e.g., Gulf of Mexico, Schiffman 1982). The consolidation properties of clay tend toward partial drained or fully drained if silt presents in clay. Similar effects may be indicated by further compositional and depositional fragments. In clay, the drained response of the pipeline induced by large deformations has been less investigated (Paulin 1998).

5.4 Centrifuge tests

The testing program contains five series of tests involving in the lateral interaction of pipe-backfill-trench in clay through large lateral movement at a centrifuge with 19.1 g acceleration. In each run, two pipes with different configuration were dragged in opposite directions. Additionally, three series of tests were carried out in the dry loose sand. Although, in this paper, the results of performed tests in clay with sand backfill (rectangular trench) were discussed. The author used the transparent observation window placed on the front side of test box in order to directly monitor the details of interactive failure mechanisms during the lateral displacement of the pipeline. High quality images were captured by digital cameras for particle image velocimetry (PIV) and post-processing. During the tests, the full equipped model sections of pipeline were placed on the bottom of excavated trenches and were buried with backfilling material. The pipes were pulled in opposite direction with fixed moving pace controlled by two vertical actuators which had pulleys and horizontal cables, while pipes were not constrained in the vertical direction.

Principal objectives of the experimental tests are:

- Failure mechanisms in both trench wall and backfill;
- P-y response of pipeline and peak resistance for both drained and partially drained tests;
- Interaction properties of the pipe-back-trench;
- Impact of backfilling properties, trench geometry, interaction rate, suction force mobilization and soil stress history;
- Development of analytical models for both ultimate soil resistance and lateral p-y curve;
- Assessment and development of this study for lateral interaction of pipeline-soil;
- Comparison between experimental results and previous studies without trenches

The primary objective of this paper is a general review of instrumentation, test configuration, observation and the primary results which were acquired from testing procedure in clay. Additional analysis of these data is proceeding, and the outcomes will be released accordingly. Failure mechanisms instances and proportional PIV analysis is produced. Testing program clarified to maximize the achieving high quality data. A summary of performed testing procedure is shown in Table 5-1.

Table 5-1. Sand backfill testing program

Characteristics	DETAILS
Test bed	cohesive
Pipe diameter	31.7 mm
Scale	19.06
Model cover depth	99 mm
Embedment ratio (H/D)	4.12
Trench backfill type	Loose Sand
Trench wall	vertical
Modified displacement rate	0.00929 mm/s
Normalized velocity (vD/c_v)	0.422
Normalized pulling distance	3.60
T-bar site backfill S_u	2-3.7 kPa
Native S_u at pipe depth	16-19.5 kPa
Native soil water content after consolidation (%)	30.81
Native water content after test at pipe depth (%)	31.11
Native soil void ratio	0.815
Saturated unit weight (γ_{sat})	18.56 kN/m^3

In order to derive the profiles of undrained shear strength in both backfilling and native material, a T-bar penetrometer (Stewart and Randolph 1994) was employed. For deep penetrations, 10.5 T-bar bearing factor was selected. On the other hand, for shallow depths, a decreased bearing factor due to buoyancy of the soil and shallow failure mechanism mobilized prior to soil full flowing throughout the bar (White et al. 2010) was employed to convert the calculated bearing resistance to undrained shear strength.

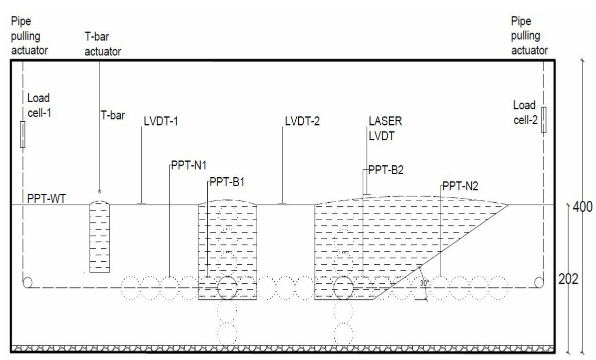


Figure 5-1. Configuration of experimental test

5.5 Numerical modelling

5.5.1 Development of CEL model

A coupled Eulerian-Lagrangian (CEL) model was developed in ABAQUS/Explicit to explore the backfill-trench-pipeline interaction. CEL has advantage in overcoming the mesh distortion problem compared with the conventional Lagrangian mesh. The large deformation of soil caused by the laterally displaced pipeline can be well represented using Eulerian elements. Pipeline has been modelled as a discrete rigid body with Lagrangian mesh. According to the geometry of the experimental tests (see Figure 5-1), the CEL model configuration was set in ABAQUS/Explicit (see Figure 5-2). The whole Eulerian domain has been separated into 4 parts: (1) initial void part (void above the initial soil surface), (2) native clay soil seabed, (3) trench with sand backfilling, (4) initial void part in trench taken by pipeline (no soil particles). Different parts were assigned with multi-material representing different types of soil.

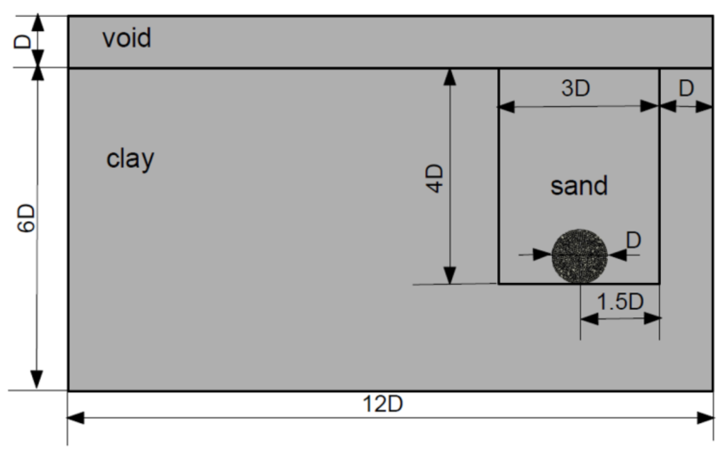


Figure 5-2. configuration of numerical model

To model the native ground clay behavior, the cam clay constitutive model is used, and parameters of clay are selected based on the experimental test (see Table 5-2), Paulin's thesis (1998), and Chen's thesis (2013).

Table 5-2. Characteristics of native clay ground

Characteristics (%)	Vancouver
Density	1800
Stress ratio at critical state	0.8
Peak strength parameter	0.5

Linear hardening rule of Cam-clay model requires the relation between yield stress values and plastic natural volumetric strains (Tekeste et al. 2013) and this needs to be input as tabular mode since this is the only option for ABAQUS/Explicit (ABAQUS 2012a). With tests conducted (oedometer test etc.) for required parameters, the plastic volumetric deformation, elastic natural volumetric strain, and therefore the plastic natural volumetric strain can be calculated according to equations listed as below (Tekeste et al. 2013, ABAQUS 2012b):

$$\bar{\varepsilon}_v = \ln\left(\frac{v_i}{v_o}\right) \tag{1}$$

$$\bar{\varepsilon}_{ve} = \ln\left(\frac{v_i}{v_e}\right) \tag{2}$$

$$\bar{\varepsilon}_{vp} = \bar{\varepsilon}_v - \bar{\varepsilon}_{ve} \tag{3}$$

where

$\bar{\varepsilon}_v$ is the total natural volumetric strain

v_i is the specific volume at the maximum stress value

v_o is the specific value at the preload stress

$\bar{\varepsilon}_{ve}$ is the elastic natural volumetric strain

v_e is the specific value at lowest rebound stress

$\bar{\varepsilon}_{vp}$ is the plastic natural volumetric strain.

To model the backfill sand behavior, the Mohr-Coulomb model is used, and sand parameters are selected according to the loose sand backfill properties in Paulin's thesis (1998). Therefore; the sand unit weight was set to γ=14.8 kN/m3 for the loose sand and other properties are listed in Table 5-3.

Table 5-3. Characteristics of backfill sand

Characteristics (%)	Value	Unit
Density	1480	kg/m^3
Poisson's ratio	0.33	-
Young's modulus	5	MPa
Friction angle	31	degree

5.6 Simulation steps

5.6.1 First step for geostatic stress and multi-material assignment

Set geostatic stress for soil models via predefining conditions. To specify different types of soil in native ground and trench backfill (consider the room taken by buried pipeline), trench geometry and seabed ground geometry were created as reference regions and EVF tool was adopted to assign different materials into different reference regions (see Figure 5-3). With gravity load executed on whole model, the stress (S33) in the soil can be observed in Figure 5-3.

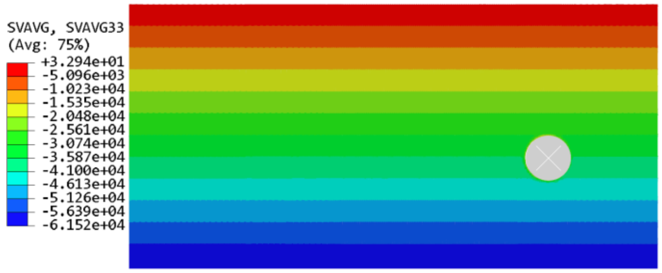


Figure 5-3. Stress levels in soil

5.6.2 Second step for lateral displacement of pipeline

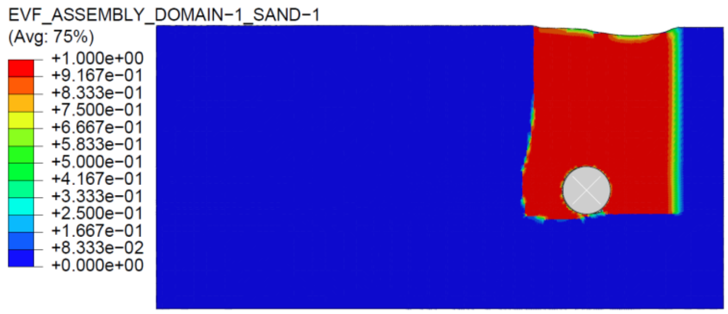
Velocities normal to all surfaces of the whole Eulerian domain were set as zero to prevent the flow out and flow in of materials during the analysis. The pipeline was displaced laterally by a distance of 4D with constraint in vertical direction. During the large lateral displacement of pipeline, the failure of trench wall was observed, and this will be discussed in next section.

5.7 Results

5.7.1 Failure mechanism

During the lateral displacement of pipeline, different flow trends of soil occurred in different locations. As shown in Figure 5-4, before the pipeline enters into the native soil (see Figure 5-4 (b)), load has been transferred to native ground by the backfilling sand and the clay soil in the front side of pipeline was forced to start moving (see Figure 5-4 (a)). Also, it was observed that the backfilling sand began to fall downward especially sand in approximate a curved band on the rear side while the pipeline moved forward.

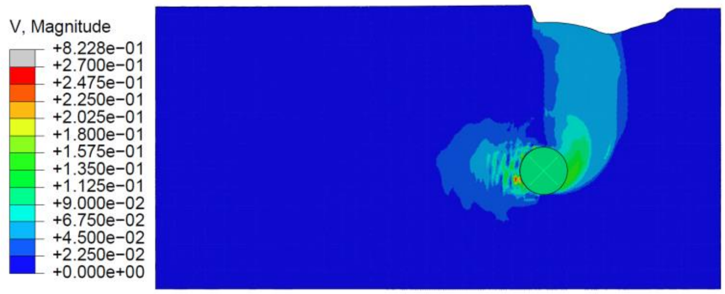
(a) Velocity of soil



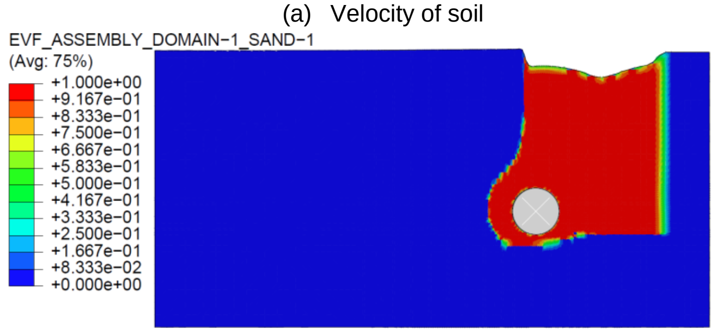
(b) Distribution of soil

Figure 5-4. Pipeline laterally displaced by 0D-0.5D.

While the pipeline further displaced and arrived at the trench wall (see Figure 5-5 (b)), a similar curved band of falling sand can be observed in Figure 5-5 (a) and this time, left part of backfilling sand showed larger velocity in flowing. It can be observed that backfilling sand in front of the pipeline has been somewhat pushed into the native ground and in that region soil particles have higher magnitude of velocity compared with shown in Figure 5-4 (a).



(a) Velocity of soil



(b) Distribution of soil

Figure 5-5. Pipeline laterally displaced by 1D-1.5D.

Failure of trench wall showed while the pipeline further entered into the native ground as shown in Figure 5-6. Instability of the trench wall caused by the interaction can be directly observed in Figure 5-6 (a) since the velocity of the native ground soil near to the trench wall increased significantly compared with figure 5-4 and figure 5-5. Indications of cracks in clay can also be observed at the surface of native ground (see Figure 5-6 (b), vertically above the pipeline) between the actively moving clay part and the relatively stationary clay part (see Figure 5-6 (a)).

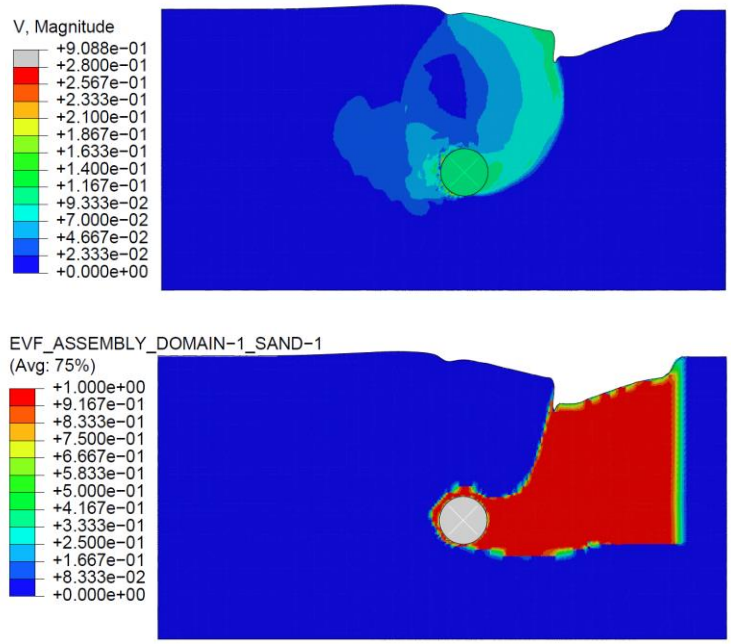


Figure 5-6. Pipeline laterally displaced by 2D-3D.

As shown in Figure 5-7, with the vectors plotted for the soil materials, the backfill-trench-pipeline interaction can be better observed. The location of most active region of soil with high velocity moved laterally with the displacement of pipeline. Also, clear difference in moving trends of native ground can be found in Figure 5-7 (c) and Figure 5-7 (d) and indications of crack showed right in that area (see Figure 5-7 (d)).

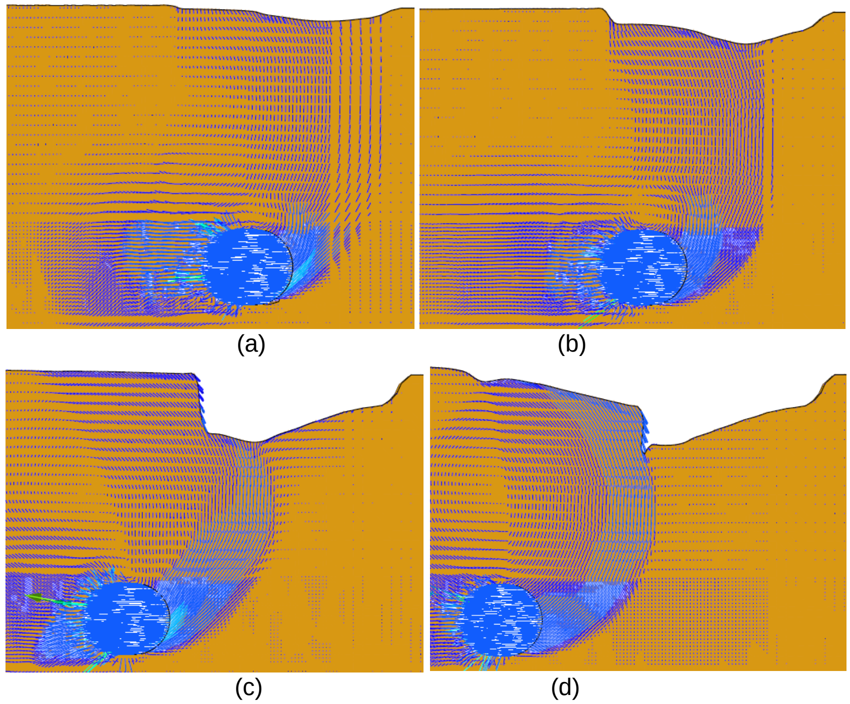


Figure 5-7. Backfill-trench-pipeline interaction

5.8 Comparison with experimental test

As shown in Figure 5-8 and figure 5-9, the results from experimental test and numerical model meet well. The ultimate lateral load per unit length is around 80 kN/m and the normalized lateral load is around 13-14. Slight differences showed in the 0D-0.5D on the magnitude of responses where the numerical model produced higher magnitude of p-y response. Also, the ultimate response magnitude in experimental test was arrived at 1D-1.5D while in the numerical model it was arrived later at round 2D-3D. Further enhancement can be made to overcome this defect by using finer mesh in the trench wall region to get more accurate material assignment (more accurate value of material volume fractions in boundary elements) and calibrating the numerical model parameters with the experimental results.

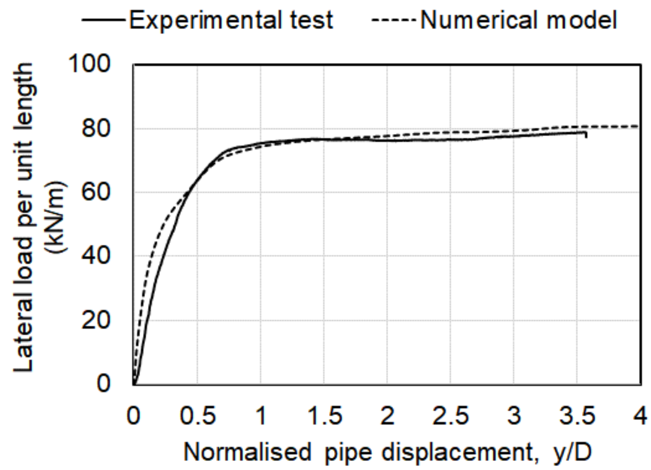


Figure 5-8. p-y responses of pipeline in numerical model and experimental test.

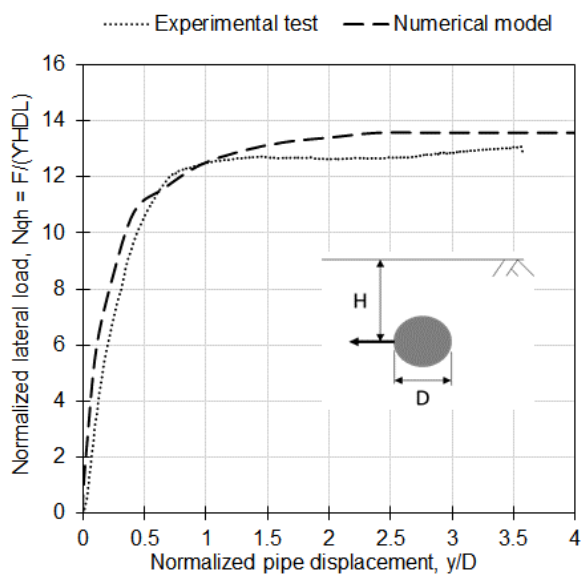
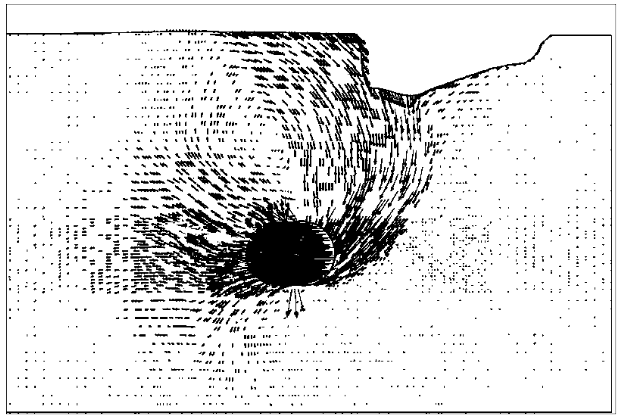
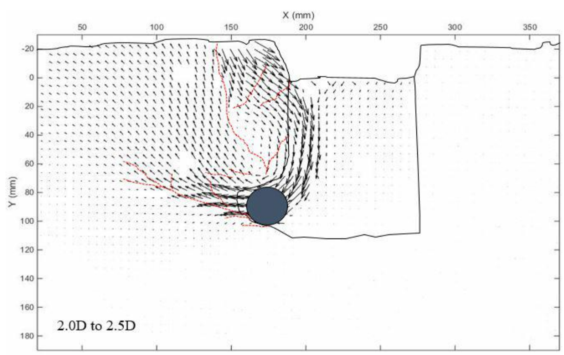


Figure 5-9. Normalised lateral load

While the pipeline entered into the native ground and the trench wall was about to fail towards the trench, the displacement trends of soil in native ground and backfilling sand showed good agreement in the numerical model (see figure 5-10 (a)) and experimental test (see figure 5-10 (b)). The trench wall began to lean towards the backfill and in following period cracks tended to show on the surface of native ground as we discussed in former section.



(a) Numerical model



(b) Experimental test

Figure 5-10. Vectorial displacement for pipe movement from 2.0D to 2.5D

In current testing procedure, it was noticed that various essential factors could control the lateral response of the pipe these parameters mostly including type and the strength of the backfilling material, geometry of trench, embedment depth and interaction rate (see Figure 5-11). Consequently, pipeline response and failure mechanism will be influenced by all of these crucial factors. Authors are now working on the postprocessing of the tests and calibration of current numerical model based on the conducted tests. Numerical modelling work will also be extended to conduct the parametric study of the key factors of backfill-trench-pipeline interaction.

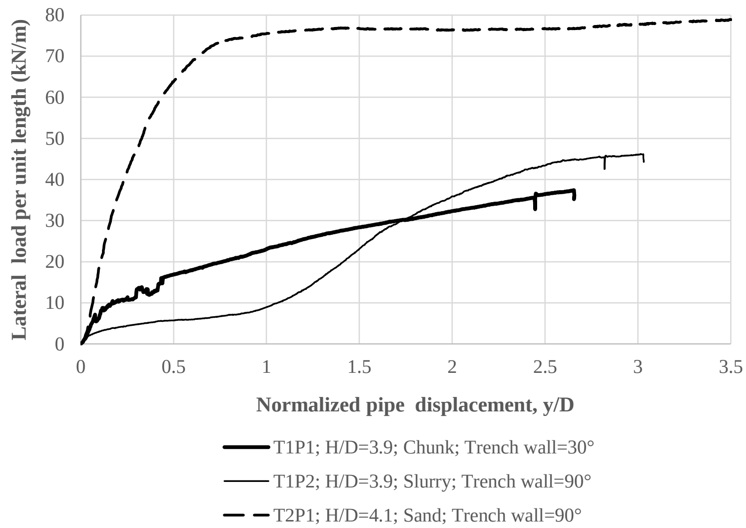


Figure 5-11. Effect of backfill type on force-displacement response (Kianian et al., 2018).

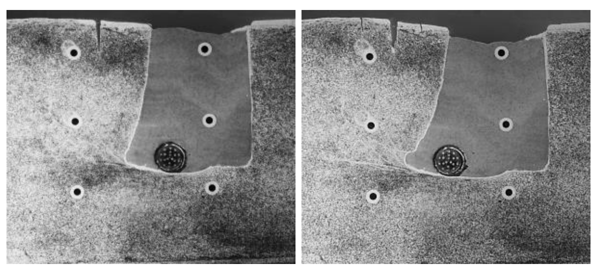


Figure 5-12. Crack shown in native ground

During the testing, cracks on the native clay ground surface can be observed with further penetration of the pipeline towards the trench wall (see Figure 5-12). Similar phenomenon can be observed in numerical modelling as shown in Figure 5-6 and Figure 5-7. Some differences could be found, and this further proved the importance of experimental tests, that is to say, experimental data will provide better assistance in setting parameters for numerical model. Then the calibrated numerical model will be adopted to conduct a series of simulations representing various backfill-trench-pipeline interaction cases to generate results for developing analytical design equations, which is one of the objectives of the whole research project.

5.9 Summary and conclusion

In order to define the shape and mechanism of failure in loose sand backfill, the present study uses experimentally verified numerical analyses. The analyses results can be summarized as follows:

- The advanced CEL model gives direct view of the interaction between backfill material, native soil and the laterally displaced pipeline by generating the moving trends of soil during the analysis.
- Curved band of moving soil showed on the rear side of the pipeline and moved forward with the pipeline displacement.
- Experimental tests have shown the influence of type and the strength of the backfilling material, geometry of trench, embedment depth and interaction rate on the ultimate pipeline response. Numerical models are now under development for further exploration with systematic parametric study to providing strong basis for proposing analytical equations for backfill-trench-pipeline interaction.
- In view of above finding, to drive an approximate formula in order to the maximum horizontal force estimation on shallow pipelines installed in dry loose-to-medium sand, we can use the failure of backfill prism geometry and maximum forces developing on the pipeline.

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CHAPTER 6: CONCLUSIONS AND RECOMMENDATION FOR FUTURE RESEARCH

6.1 Conclusions

The lateral interaction between pipeline, sand backfill, and cohesive trench wall was investigated throughout centrifuge models tests and advanced numerical studies and the results were compared against the soft slurry backfills. Transparent observation windows and digital cameras were used on the side wall of the testing box to record the failure mechanisms in the trench wall and backfill. The pipes were completely equipped by strain gages, laser and conventional LVDTs, load cells, miniature T-bar, markers and patterns, interior and exterior pore pressure transducers, etc. A set of full high-quality data was acquired, and the post-processing investigation was conducted through PIV analysis. The numerical results were calibrated and compared with the conducted tests.

The subsequent interpretation was attained as the main outcomes of the preformed study:

- The soil deformation mechanism in the backfill and the native ground was found to be completely interactive, where earlier deformations affect the later stages of interaction.
- Current design practices overestimate the ultimate lateral soil resistance for pipeline penetrating to the trench wall, underestimate the lateral resistance for

pipeline moving inside the backfill, and provide no solution for pipeline approaching the trench wall.

- The lateral response of the pipe under large displacements is significantly affected by backfilling characteristics which are controlled by various constructional factors.
- The failure mechanisms which are influenced by the relative strength of both backfill and native soil can govern the lateral response of the pipe.
- The pipeline-trench bottom interaction was observed to have a significant contribution to the lateral soil resistance. This area needs further research works to investigate the details of mechanisms.
- Lower lateral peak soil resistance was produced by softer backfilling materials.
- Pipeline may have vertical upward movement throughout the failure of the trench. As the strength of backfilling materials decreases the displacement size in the vertical direction will increase. This has a significant impact on lateral pipe response.
- The interaction rate seriously affects the lateral response of the pipe. Examining the interaction between pipe-backfill-trench reveals that as the velocity of displacement increases, the lateral resistance may decrease.
- The deviations of pore pressure have a straight relation with the lateral resistance of pipe. A suction force generation was observed behind the moving pipe but dissipated with further pipe displacement.
- In order to achieve an accurate assessment of the lateral soil resistance against the pipeline displacement, it is necessary to incorporate the trench effects.

- The advanced CEL model can provide a good view of the interaction between backfill material, native soil and the laterally displaced pipeline by generating the moving trends of soil during the analysis. However, the further analysis needs to be conducted to calibrate the model for wider conditions and configurations.

6.2 Recommendations for future research

- Expand numerical analysis for more accurate results through a wider range of soil properties and trench configurations by incorporation of more advanced user-defined subroutines for modeling the material response.
- Develop advanced implicit methods such as RITSS to investigate the consolidation effect and coupled response of soil matrix and pore pressure in partially drained conditions.
- Conduct comprehensive LDFE analysis and propose analytical solutions to incorporate the trench effect on lateral soil resistance.
- Conduct a wider range of experimental studies in undrained conditions to study the effect of other influential parameters such as trench wall angle, trench width, stress history, confining pressure, etc. and their impact on internal soil deformation mechanisms and ultimate lateral response.