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Soil Restraint on Steel Buried Pipelines under Reverse Fault Displacement

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ABSTRACT

The design of reverse fault crossings requires pipe-soil restraint properties to estimate the performance of pipeline segments. Experimental data to obtain these properties are seldom available in current published technical literature, and therefore, they are usually inferred in practice on the basis of horizontal and vertical soil restraints. This paper presents the results from a series of 35 and 45 degrees soil restraint tests conducted to improve understanding of soil-pipe behaviour in reverse fault crossings for onshore pipelines. Tests were carried out using a NPS16 diameter steel pipe buried in uniformly graded crushed limestone and in moist Fraser River sand. The pipes had about 450 mm of soil cover above the crown of pipe. Results from horizontal and vertical soil restraint tests are also included.

Introduction

The performance of buried pipelines to earthquake fault displacement depends on mobilized levels of soil restraint. Soil restraints are a function of the relative movement between the pipeline and the surrounding soil, the orientation of the pipeline with respect to the fault trace, the direction and amount ground displacement imposed by the fault, and the specific backfill soil conditions along the pipeline. For example, for pipelines closely aligned with the direction of the strike of a reverse fault, the pipeline is subjected not only to different levels of vertical oblique soil restraint due to the upward and lateral movement of the pipeline, but also to longitudinal-axial soil restraint. While there is a good understanding of pipeline behavior buried in sand and subjected to horizontal ground displacements (Trautmann and O'Rourke 1983, 1985; Yimsiri et al. 2004; O'Rourke et al. 2008) that can arise from strike-slip faults, the practice still has some shortcomings for cases involving the selection of representative vertical oblique pipe-soil restraint values for the design of reverse fault crossings. Because vertical oblique soil restraint values are seldom available in current published technical literature and are usually based on small-scale testing (e.g. Vanden Berghe et al. 2005 in loose sand), they are usually inferred in practice on the basis of horizontal and vertical soil restraints following recommendations from PRCI (2004, 2009) design guidelines. This may involve a large degree of extrapolation and conservative engineering judgment.

An experimental research program was conducted at the University of British Columbia (UBC) to investigate pipe-soil restraint properties using full-scale physical modeling. Vertical oblique displacements at angles of 35 and 45 degrees from the horizontal were applied to a pipe

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specimen to simulate the oblique angle breakout of buried pipelines from their soil embedment on the footwall side of reverse faults. Two inclinometers and a set of 8 string potentiometers (four per loading cable) were utilized in order to record and also to verify that loads were applied along the required inclinations during the testing process.

Geometric changes in the soil mass, shear rupture surfaces and levels of vertical oblique soil restraint are described and discussed with the aim of characterizing the soil-pipe interaction behaviour observed from the large-scale tests. Results from horizontal and vertical soil restraint tests are also included to record the variation of soil restraint with the inclination of the vector displacement and for completeness. The results from this work can provide valuable understanding of soil-pipe behaviour subjected to different directions of ground movement, can improved the design practices of pipeline segments crossing active reverse fault and can be used as reference for numerical modeling.

Test Equipment and Materials

Test Equipment

The tests were performed using an improved version of the Advanced Soil Pipe Interaction Research (ASPIRe) soil chamber that exists at the University of British Columbia (Monroy 2013). The internal plan dimensions of the soil chamber are approximately 2.5 m x 3.8 m and provide for up to 2 m of soil cover above the test pipe. The chamber includes large Plexiglas panels that allow direct visual observation of development of soil failure surfaces, and relative movement between soil and pipe specimen during the tests. A general arrangement of the chamber is shown in Figure 1.

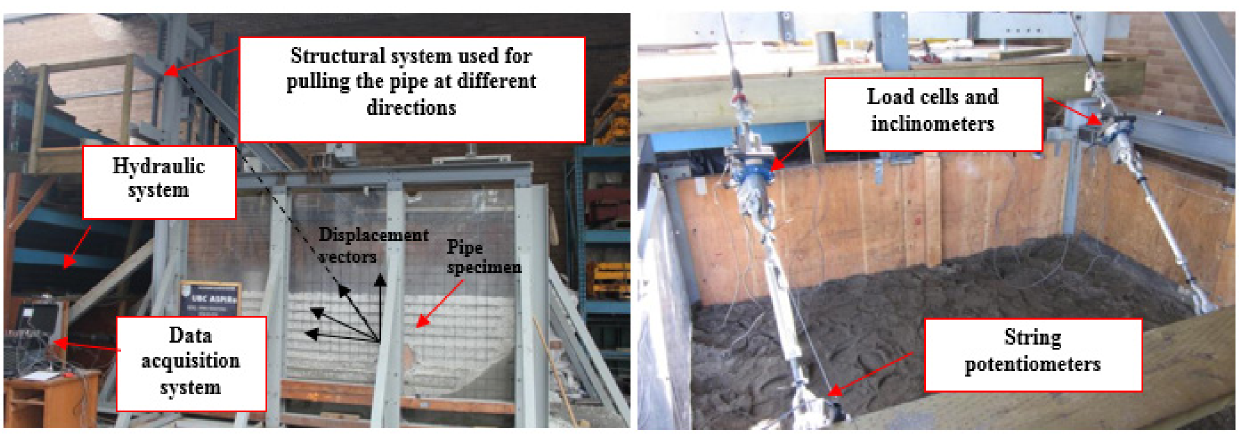


Figure 1. Perspective view of the testing chamber and general arrangements for tests

The development of soil restraints and therefore soil loads on a buried pipeline depends on the amount of relative displacement between the pipeline and the surrounding soil. Soil restraints arise due to either the restriction imposed by the soil to the free movement of the pipeline or to the restraint given by the pipeline to the movement of the surrounding soil. The testing apparatus simulates the former: the soil restraint mobilization is achieved by a set of cables that pull a buried pipe in a predefined direction, rather than pushing the soil from the back or below.

The boundary effects during testing were studied in detail (Karimian, 2004) during the original design. The size of the chamber and location of pipe were selected to allow unhindered formation of displacement zones during soil restraint testing. Displacement zones estimated using analytical and numerical models confirmed the suitability of the selected chamber dimensions. Reduction of the interface friction between soil and vertical sidewalls of the box during lateral pipe pullout was minimized by having the back wall lined with stainless steel sheeting and the front wall with Plexiglas material.

The coupling system consisted of diametric rods passing through each pipe end and attached to steel cables. Bending of the pipe at this scale is assessed to be negligible due to the high section modulus of the steel pipe used in the testing. In all cases, the loading system did not interfere with the movement of the pipe. Each cable was then connected to a load cell mounted on double-acting hydraulic actuators with a digital hydraulic control system. The capacity of the actuators is 418 kN at 21 MPa working pressure. The hydraulic actuators, manufactured by Royal Cylinders Inc., had a 200 mm bore diameter with a full stroke of ±305 mm and a 90 mm rod diameter. The load cells were MTS model 661.22, with a maximum load capacity of 225 kN.

Pipe displacements relative to the soil test chamber were measured using string potentiometers, (SP). The pulling angle was measured by a set of two inclinometers attached to the pulling cables and a set of eight SPs. All measurements from the instrumentation array monitoring the pipe specimens were recorded at 20 Hz (20 samples per second).

Soil and Pipe Materials

Sand-blasted steel pipe specimens (steel Grade 524A) with 2.4 m length and 406 mm (NPS16) diameter were used for the tests. The thickness of all pipe specimens was 1.27 mm. With respect to backfill materials, two scenarios were reproduced. One scenario represents a medium-dense (average dry density of about 1600 kg/m^3) condition that was simulated by using uniformly-graded, moist Fraser River sand with a moisture content of 3 to 5%. The second scenario represents a condition in which sand material is not available at the project site, and “real” soil materials must be used. A 19 mm minus crushed limestone with an average dry density of about 1700 kg/m^3 was selected for this purpose. The grain size curves for the backfill soils are presented in Figure 2.

Fraser River sand was extensively used and documented during numerous element laboratory research programs performed at UBC in the past. The results of those investigations indicate an average grain size, D50, of 0.3 mm, a minimum particle size of 0.074 mm, and a specific gravity (Gs) of 2.70. The constant volume of internal friction angles range from 32° to 34°. The peak friction angle at a dry density of 1600 kg/m^3 is 43°. Crushed limestone showed average peak friction angles between 46º to 51º.

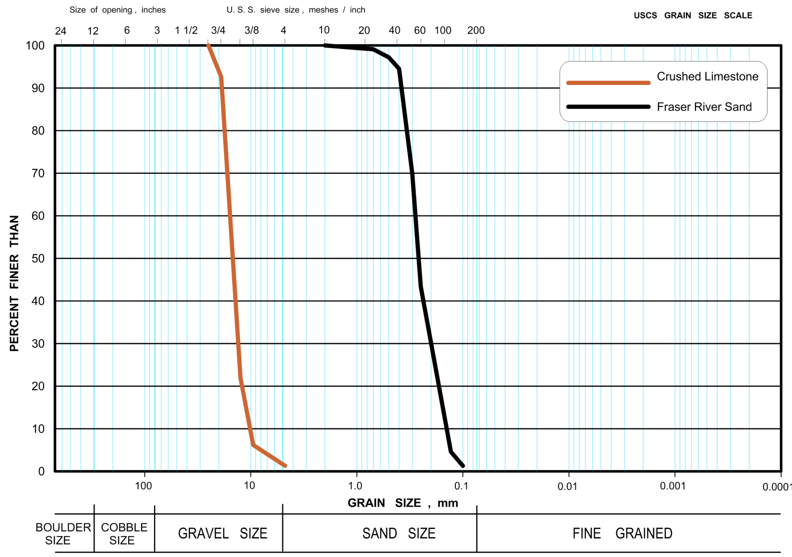


Figure 2. Grain size distribution of backfill materials used in the present study

Experimental Work and Results

Test Program

A total of four (4) tests were conducted to characterize the mechanical soil-pipe behavior due to ground displacements like those induced by reverse faults. In addition, four tests were conducted to determine soil-pipe interaction behaviour under horizontal and vertical upward displacements. Details and characteristics of the tests are shown in Table 1.

The backfill soil was placed in the chamber in approximately 300 mm lifts and mechanically compacted using a static roller to achieve the desired target average soil density. After initial placement of soil to a thickness of about 450 mm, the pipe specimen was placed on the soil bed; the filling of the box was continued up to the level corresponding to the desired overburden ratio, H/D (where H is the vertical distance from the pipe centerline to the ground surface, and D is the pipe diameter). The mass density of the backfill material was measured at random locations during the filling process using a calibrated nuclear densometer. In addition, the density of the compacted backfill was verified using mass-volume measurements taken from aluminium bowls placed within the fill prior to compaction. Upon completion of a given test, the material was removed through an opening located at the rear of the soil test chamber.

The test pipes were loaded in a displacement-controlled manner at a rate of 2.5 mm/s. The loading rates have no noticeable effect on the results (Karimian 2006). For all tests, the total load per unit length on the pipe was determined by adding the load measured from each load cell and dividing it by the length of the pipe specimen (2.4 m). Symmetry of the pulling system was verified by controlling the difference in recorded readings from each load cell to be less than 5%.

Table 1. List of soil restraint tests.

No.	**Test ID**	**Pulling angle**[1]	**Backfill**	**dry density (kg/m^3)**	**Purpose / Comments**
1	T1-α0-Sand	0°	Moist Sand	1,600	Determine horizontal soil restraint
2	T2-α45-Sand	45°	Moist Sand	1,600	Determine soil restraint
3	T3-α45-Sand	45°	Moist Sand	1,600	Repeatability
4	T4-α90-Sand	90°	Moist Sand	1,600	Determine vertical soil restraint
5	T5-α0-CLimestone	0°	Crushed Limestone	1,700	Determine horizontal soil restraint
6	T6-α35-CLimestone	35°	Crushed Limestone	1,700	Determine soil restraint
7	T7-α45-CLimestone	45°	Crushed Limestone	1,700	Determine soil restraint
8	T8-α90-CLimestone	90°	Crushed Limestone	1,700	Determine vertical soil restraint

[1] Angle with respect to horizontal

Test Results

The normalized soil restraint (N_α), and normalized displacement (Y') are presented in the form:

$$N\alpha = \frac{P\alpha}{\gamma \cdot D \cdot H} \tag{1}$$

$$Y' = \frac{Y}{D} \tag{2}$$

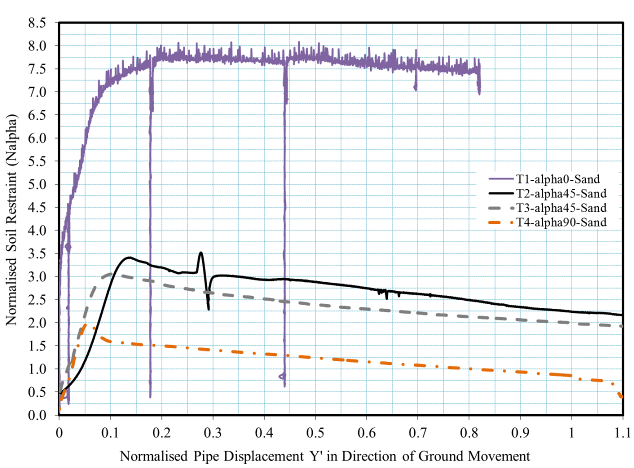
where P_α is the measured total load per unit length, γ is the unit weight of the backfill, D is the pipe diameter, H is the height of soil over the pipe springline and Y is the recorded pipe displacement. The form of the normalized load and displacement, shown above, follows the relationships presented in previous research about lateral soil restraint (Audibert and Nyman 1977, Trautman and O'Rourke 1985, O'Rourke et al. 2008 and PRCI 2004, 2009).

Soil Restraint on Pipes Buried in Moist Sand

Variations of normalized soil restraint, N_α vs. normalized pipe displacement, Y'= Y/D, for tests on a NPS16 (406-mm diameter) pipe specimen buried in moist sand with an overburden ratio H/D of 1.6 are shown in Figure 3a. Pulling displacement of about 0.8D to 1.1D were applied to the pipe specimens with inclinations of 0° (horizontal), 45° and 90° (vertical) to simulate different directions of ground displacement. Figure 3a shows that as expected the largest peak soil restraint on the pipe specimen arose when the pipe was displaced horizontally (Test T1-α0-Sand). The peak soil restraint values diminished as the inclination of the angle of breakout of buried pipelines increased with respect to the horizontal. A similar behaviour was found by Vanden Berghe et al. (2005) for loose sand, Yimsiri et al. (2004) and Guo (2005) for pipe specimens buried in clay.

The soil restraint-displacement relationships indicated a generally continuous rise of soil restraint during the test reaching a peak value at relatively small pipe displacements (i.e., 0.05D to 0.18D). After the peak soil restraint was reached, a fairly constant-rate decrease in soil resistance with increasing pipe displacements was noted for tests that simulated ground displacement at 45° and 90° to the horizontal. For Test T1-α0-Sand, the soil-pipe interaction showed a nonlinear relationship between the mobilised lateral soil restraint and pipe displacement; until the lateral restraint imposed by the soil on the pipe reached its maximum value (it is fully mobilized and overcome). In the lateral pulling test, the condition of maximum normalised lateral restraint, N_α reached a value of about 7.8 at an early stage during the test (Y' = 0.25D). This result agrees with that presented by O'Rourke (2008).

The soil-pipe interaction for both tests T2-α45-Sand and T3-α45-Sand showed a continuous increase of soil restraint during the tests until an average peak normalized vertical oblique resistance, N_α, of about 3.2 was mobilized at an average normalized displacement, Y', of about 0.12D. After the peak soil restraint was reached, a decrease of loading with a fairly constant rate was observed during the rest of the tests. A minimum average normalized soil restraint, N_α, of 2.2 was measured for Test T3-α45-Sand at a 45° vertical oblique normalized pipe displacement, Y', of 1D.



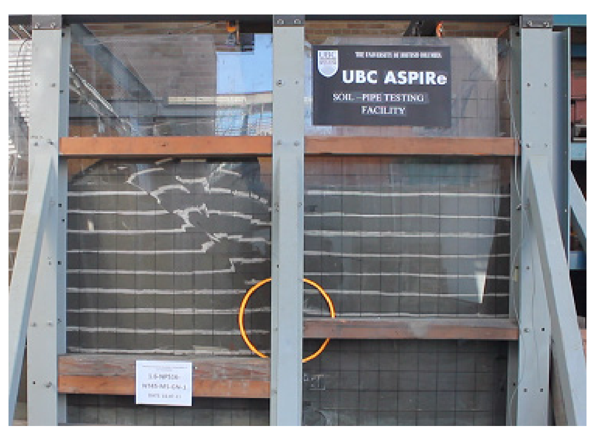


Figure 3(a). Normalized load-displacement relationships for NPS16 pipe specimen with H/D=1.6

buried in moist sand – (b). Failure surface at end of test Y'=0.73D.

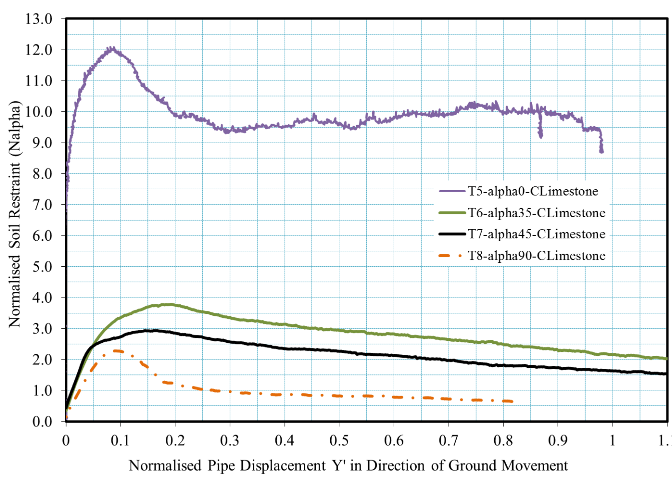
The patterns of soil movements and the failure surface developed during Test T3-alpha-45-Sand Y'=0.73D is illustrated in Figure 3b. This figure shows a planar failure surface oriented at roughly 45° from the horizontal and located in front of the pipe. The failure surface was initially noted at a Y' of 0.12D. After this failure condition, large changes in the soil mass were observed. As test progressed, other failure surfaces developed in the soil mass. However, these failure surfaces were related to active conditions imposed by movements of the soil mass towards void zones. The decrease in vertical oblique soil restraint observed in Figure 3a appears to be directly related to the overburden reduction as the pipe moved towards the surface.

Soil Restraint on Pipes Buried in Crushed Limestone

Variations of soil restraint, N_α, vs. normalized pipe displacement, Y'= Y/D, for tests on a NPS16 (406-mm diameter) pipe specimen buried in uniformly crushed limestone with an overburden ratio H/D of 1.6 are shown in Figure 4a. Pulling displacement of about 0.8D to 1.1D were applied to the pipe specimens with inclinations of 0° (horizontal), 35°, 45° and 90° (vertical) to simulate different directions of ground displacement. Tests T6-α35-CLimestone and T7-α35-CLimestone included a hard trench wall as these tests were used for evaluating the benefit of using geosynthetic-lined trench wall as a means to reduce levels of soil restraint.

As seen in Figure 4a, the soil restraint-displacement relationships for pipe specimens buried in crushed limestone shared a behavior similar to that observed for pipes buried in moist sand. Test T6-α35-CLimestone showed a continuous rise of soil restraint that occurred during the initial part of the test until a normalized vertical oblique soil restraint, N_α, of about 3.8 was reached; then a relatively softer behaviour was observed for the rest of the test.

The peak normalized soil restraint, N_α, for Test T7-α45-CLimestone was about 2.9 and occurred at a normalized vertical oblique displacement, Y', of 0.15D. The lowest recorded N_α value was 1.5 at a normalized vertical oblique displacement, Y', of 1.1D.



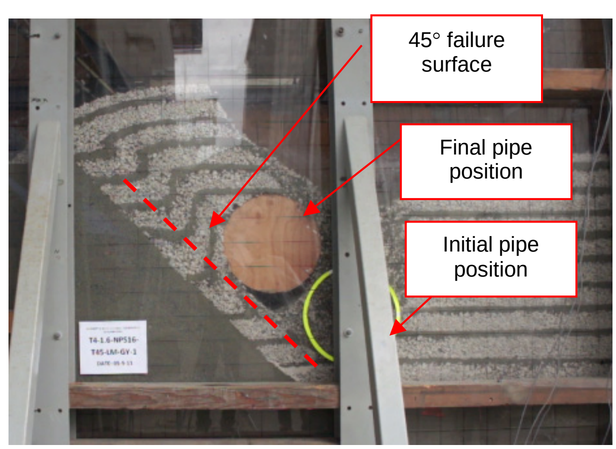


Figure 4(a). Normalized load-displacement relationships for NPS16 buried in crushed limestone – (b). Failure surface at end of test Y'=1.1D

Similar to the case of sand backfill, a planar failure surface oriented at roughly 45° from the horizontal and located between the front of the pipe and the trench wall (inclined at 45°) appeared to be developed during the failure condition (Figure 4b). Large movements in the soil mass behind and above the pipe specimen were observed after this failure condition as crushed limestone particles flowed towards void zones. The nearly vertical wall formed behind the pipe during the test evidenced a high shear resistance associated with the compacted crushed limestone. As evidenced in Figure 4b, a wood cap was placed on one end of the pipe. This was done to observe the change in pipe position as the test progressed. The wood cap was not in contact with the Plexiglas to avoid frictional resistance or pipe end effects. While a few gravel particles got trapped between the end cap and the Plexiglas, this condition occurred after the peak soil restraint was reached. The trapped gravel particles produced no appreciable additional soil resistance to pipe movement as most of the resistance was produced by the material located in front of the pipe and not by some frictional resistance developed at the end of pipe.

Conclusions

A series of tests were conducted on a pipe specimen buried in moist Fraser River sand and uniformly graded crushed limestone to improve understanding of soil-pipe behaviour in reverse fault crossings. Soil restraint-displacement relationships for pipe specimens subjected to displacements with angles of 35 and 45 degrees from the horizontal indicated a generally continuous increase in soil restraint during the test, reaching a peak value at relatively small pipe displacements. After the peak soil restraint was reached, a fairly constant rate of decrease in soil resistance with increasing pipe displacements was noted. The testing work conducted under displacement-controlled loading provided a unique opportunity to observe and quantify this post-peak response – which is a valuable piece of information in the development of more realistic "soil springs" for numerical simulations for the design of pipelines crossing reverse faults.

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