Effectiveness of Geotextile-Lined Pipeline Trenches Subjected To Relative Lateral Seismic Fault Ground Displacements

M. Monroy & D. Wijewickreme
University of British Columbia, Vancouver, BC, Canada

D. Honegger
D.G. Consulting, Arroyo Grande, CA, USA

SUMMARY:
A series of tests was carried out to evaluate the effectiveness of using geosynthetic-lined pipeline trenches to reduce soil loads on pipelines subject to relative lateral ground displacements. Full-scale tests of geotextile-lined pipeline trenches were performed in a large soil chamber where horizontal soil restraint was simulated in a two-dimensional manner by applying relative horizontal displacements. Results indicate that lateral soil restraint increases with increasing displacement and could even be higher than the restraint without the geotextile lining. The results also indicate that the shear resistance is not controlled by the geotextile interface, but by the frictional response of geotextile-soil interface and of the backfill. This observation suggests that there is only a marginal benefit in using geotextile-lined trenches on buried pipelines subjected to large lateral displacements. However, the use may have value if pipelines are subject to small displacements that may arise from thermal expansion, subsidence, or slope creep.

Keywords: pipeline, trenches, ground movements, geotextiles, soil restraints

1. INTRODUCTION

One of the considerations related to the earthquake performance of buried pipelines is the levels of soil restraint developed during potential permanent ground movements crossing the pipeline alignment. The level of soil restraint depends on the relative displacement between the buried pipeline and its surrounding soil and can induce bending, shear, tension or compression demands on segments of buried pipelines. Lateral soil loads arising due to abrupt ground movement, like those imposed by landslides and lateral strike-slip faults, are among the most commonly studied cases of soil-structure interaction for pipelines. The research findings on this subject have been based upon a relative large number of laboratory, numerical and field experimental investigations on pipe response in buried soils and also studies on related structures (piles, anchor plates, strip footings). Charts and theoretical procedures to estimate maximum loads on pipelines during relative lateral soil movements are available and often based on frictional properties of the soil and assumed failure mechanisms (Hansen, 1961; Trautmann and O'Rourke, 1983; O'Rourke et al, 2008).

For pipelines located in tectonically active regions, sources of large ground displacements include surface faulting, triggered landslides, and lateral spreading. These large ground displacements can impose large levels of demand that may greatly exceed established pipeline acceptance criteria. When confronted with this technical challenge, one of the first mitigation options is to reduce soil restraint on the buried pipeline and therefore increase the displacement capacity. For example, the installation of pipelines in a trapezoidal trench with loose to moderately dense sand backfill is one of the common mitigation measures undertaken to reduce soil loads in situations of abrupt ground movement such as pipeline fault crossings. However, the use of a trapezoidal trench with sand backfill may not be a feasible option in locations where suitable low-cost sand backfill is not readily available or drainage and erosion issues preclude the use of sand. Other recommended mitigation option given by design guidelines (e.g. PRCI, 2004) is the use of geosynthetic fabric on sloped trench walls. This recommendation is based upon the premise that slippage in the form of a contiguous soil blocks would be promoted due to the low frictional properties prevalent at the geosynthetic fabric interfaces.

However, the effectiveness of this technique is currently poorly understood, and little guidance is available to permit more complete specifications on geosynthetic installation requirements and define methods for quantifying reductions in soil restraint.

The work previously undertaken at UBC suggests that the behavior of geotextile-lined pipeline trenches subjected to permanent lateral ground deformation does not seem to reduce soil loads on pipeline to the extent it was intended (Karimian et al. 2006). Load-displacement relationships obtained from full-scale testing of buried pipelines in geotextile-lined trenches suggest that the level of lateral soil restraint does not reach a limiting value; instead, it increases with pipe displacement once the pipe approaches the sloped trench wall. This behaviour occurred in spite of the slippage observed between the geotextile layers.

A comprehensive full-scale physical modeling research program was undertaken at UBC to investigate the lateral soil restraint on pipelines buried in geotextile-lined trenches. Permanent relative lateral horizontal displacements were applied to steel pipe specimens, buried in a geosynthetic-lined trench constructed in a large soil chamber. The effect of varying trench slope angle on soil restraints was also investigated together with the influence of the separation distance between pipe and geotextile interface on geotextile contribution to reducing soil restraint. The findings from this work are presented and discussed in this paper.

2. EXPERIMENTAL SET UP

Typical real-life pipeline configurations were physically modeled using the UBC Advanced Soil Pipe Interaction Research (ASPIRe™) soil chamber (2.4 m wide x 4.0 m long x 2.5 m high). This is an improved version of the soil chamber initially developed by Anderson et al. (2004). The internal plan dimensions of the soil chamber measures approximately 2.5 m x 4.0 m and provides for up to 2 m of soil cover above the test pipe. The chamber includes large plexiglass panels that allow direct visual observation of sectional views of different test configurations, development of soil failure surfaces and relative movement between soil and pipe specimen during tests. A general arrangement of the chamber is shown in Figure 2.1.

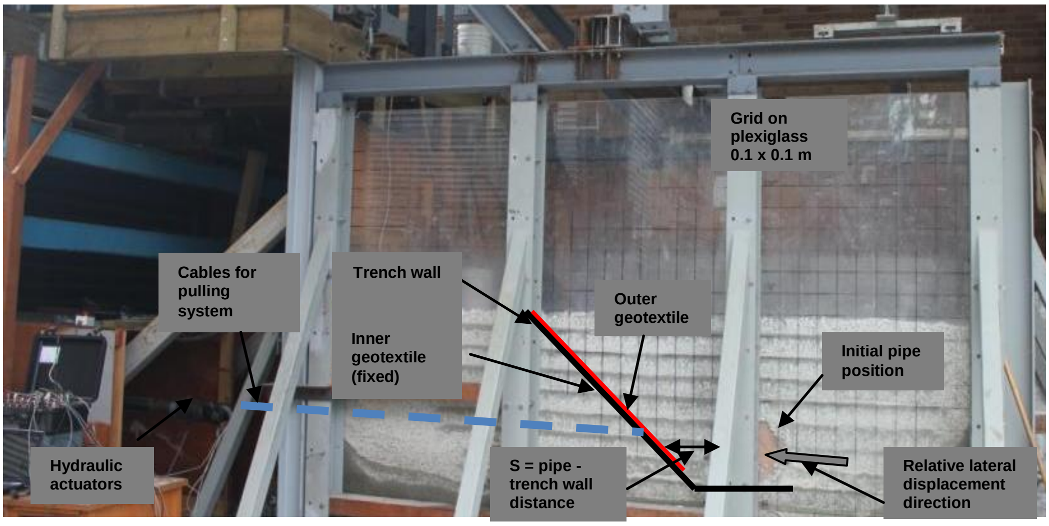

Figure 2.1. Perspective view of the testing chamber and general arrangements for tests.

The investigation of the role of geotextiles as a mitigation measurement requires the reproduction of field-specific geometrical, material and boundary conditions. Therefore, a full-scale trapezoidal trench was carefully constructed inside the soil chamber. Two trench side-wall slope angles were investigated, 35° and 45° to the horizontal. The trench side-wall was lined with dual layers of TC Mirafi Filterweave 700, woven geotextile fabric. The 45° side-wall slope angle was considered to represent a limiting value for measurable benefit. Side-wall slope angles less than 35° were not considered based upon what was believed practical limit on the size of the trench in practice. Only the outer geotextile layer (in contact with backfill material) was allowed to move. Direct shear tests on the dual layer of geotextile fabric show a peak friction angle of 20±1°. A trench wall having a relatively hard native soil condition was simulated with a trench wall built using compacted 19-mm minus well-graded sand and gravel. Sand-blasted steel pipe specimen with a length of 2.4 m, thickness of 1.27 mm and 457 mm (NPS18) diameter was used for the tests.

Testing was undertaken using two types of trench backfill materials. A medium-dense backfill (average dry density of about 1600 kg/m^3) condition was simulated using uniformly graded Fraser River sand with moisture content of 3 to 5%. Sand was selected in order to provide a baseline case and allow our test results to be compared with other published research (which has largely been performed in sand). Fraser River sand has been 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, D_{50} of 0.3 mm, minimum particle size of 0.074 mm and a specific gravity (G_s) of 2.70. Constant volume internal friction angle for Fraser River sand ranges from 32° to 34°. The peak friction angle at a dry density of 1600 kg/m^3 is 43°.

The other trench backfill material used was sand and crushed gravel mixture used for road construction. The sand and gravel used in the tests was placed with an average dry density of about 1800 kg/m^3. Large direct shear tests (0.3 m x 0.3 m) on the sand and gravel indicated an average and peak friction angle of 49º and 51°, respectively. Direct shear tests on the geotextile-moist sand and geotextile-sand and gravel interfaces suggest a peak friction angle of 31±1°.

Approximately 23 m^3 of soil material was typically required to fill the soil test chamber for a test. The test chamber was filled by dumping the backfill material using a tipping hopper car attached to a forklift. Once the soil material was deposited, it was spread within the soil test chamber forming lifts of 300 mm that were later compacted with a hand-pushed static roller. 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 coupling system for the tests consisted of end clamps at each end of the pipe combined with 29-mm steel cables and shackles. The 29-mm cables were directly attached to the clamps and passed through PVC pipes in the soil behind the formed trench wall and vertical slots in one of the ends of the test chamber wall. The pipes and vertical slots were provided to allow free vertical movement of the cables in the event of vertical pipe uplift during the tests. 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 a double-acting hydraulic actuator with a digital hydraulic control system. The capacity of the actuators is 418 kN at 21 MPa working pressure and the actuators have a full stroke of ±305 mm.

Pipe displacements relative to the soil test chamber were measured using string potentiometers mounted outside and at the back of the test chamber. Two 1.6-mm diameter steel cables that were attached to both back ends of the pipe were passed through small-diameter PVC pipes embedded in the soil and connected to string potentiometers. The displacements of geosynthetic fabric layers were measured through an additional set of potentiometers. Very thin extension cables were attached to the top of the geosynthetic fabric layer and to string potentiometers that were mounted on the chamber wall. All measurements from the instrumentation array monitoring the pipe specimens were recorded at 20 samples per second.

3. EXPERIMENTAL WORK

A total of 11 tests were conducted to assess the performance and effectiveness of geofabrics in reducing levels of soil restraint on buried pipelines. Geotextile-lined pipeline trenches and plain cases (no trench) were constructed in full-scale dimensions. Lateral soil restraints were simulated by providing relative soil-pipe displacements in the horizontal direction. Details and characteristics of the tests are shown in Table 3.1.

The test pipes were loaded in a displacement-controlled manner at a rate of 2.5 mm/s. During full-scale testing, loads and displacements applied on the pipe were measured and their corresponding failure mechanisms were investigated. Geotextile displacements were also recorded and evaluated in the context of soil-pipe interaction.

Table 3.1. Testing Matrix for the experimental work

ID	Pipe Diameter	H/D	Backfill Material		Trench wall angle	Distance pipe to trench	Geotextile
			Sand	Sand and gravel			
T1	NPS18	1.9	♦		45	0.5D	No
T2	NPS18	1.9	♦		45	0.5D	Yes
T3	NPS18	1.9	♦		45	1.0D	Yes
T4	NPS18	1.9	♦		45	2.0D	Yes
T5	NPS18	1.9	♦		No Trench	No Trench	No
T6	NPS18	1.9		♦	45	0.5D	Yes
T7	NPS18	1.9		♦	45	0.5D	Yes
T8	NPS18	1.9		♦	35	0.5D	Yes
T9	NPS18	1.9		♦	35	0.5D	Yes
T10	NPS18	1.9		♦	No Trench	No Trench	No
T11	NPS18	1.9		♦	No Trench	No Trench	No

4. TEST RESULTS

For all tests, the total load per unit length on the pipe was determined by adding the load measured from each load cell and then 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%.

Test results are presented in terms of normalized values of lateral soil restraint (N_{qh}) and normalized displacement (Y') determined from the equations below:

$$N_{qh} = P / (\gamma \cdot D \cdot H \cdot L) \tag{4.1}$$
$$Y' = Y / D \tag{4.2}$$

Where P is the measured load, γ is the dry unit weight of the backfill, D is the pipe diameter, H is the height of soil over the pipe springline, L is the pipe length, 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 (Hansen 1961, Audibert and Nyman 1977, Rowe and Davis 1982, Trautman and O'Rourke 1983, Paulin et al 1998, and PRCI 2004).

4.1 Tests with Sand Backfill

The development of soil restraint as relative lateral displacement of the pipe specimen progresses for test configurations with and without geofabric lining, T1 and T2, respectively, is shown in Figure 4.1. The lateral soil restraint increases with pipe displacement until a constant lateral soil restraint is observed (a plateau) at normalized pipe displacements that range from 0.1D to about 0.3D to 0.4D.

Further pipe displacement beyond 0.3D produces a significantly nonlinear increase of lateral soil restraint on the pipe and suggests the development of additional soil deformation mechanisms.

From the data plotted in Figure 4.1, it can be seen that the lateral soil restraint at the plateau level has only been reduced by about 15% due to the application of the geotextile-lining. Similar limited benefit observed for mitigation configurations based on geotextile-lined trenches and the development of an increase in lateral soil restraint at large pipe displacements were also reported by Karimian et al. (2006). The load-displacement relationships from Karimian et al. (2006) are shown in Figure 4.1 for reference and comparison purposes.

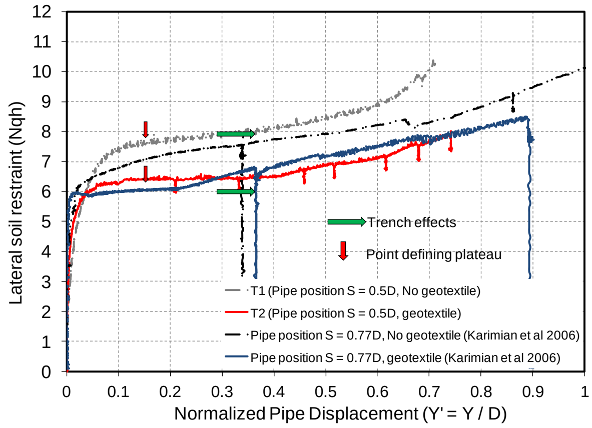

Figure 4.1. Lateral soil restraints in sloped trench walls with moist sand backfill.

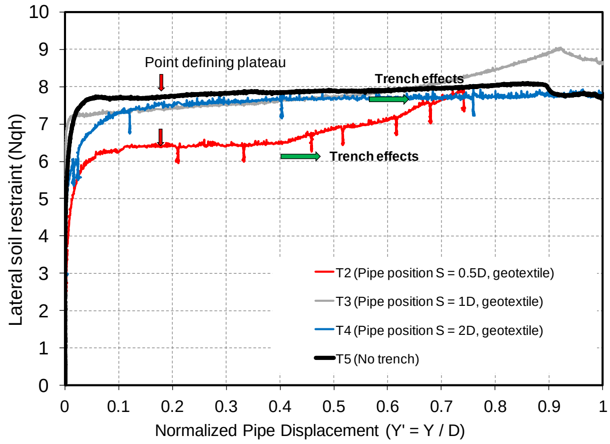

Figure 4.2. Effect of trench wall-pipe distance (S) on levels of soil restraints.

Further tests were conducted to investigate the effect of the trench wall on the mechanical behavior during soil-pipe interaction. Tests T3 and T4 were carried out with a trench wall-pipe distance (S) of 1D and 2D, respectively. In addition, a test with moist sand backfill (with no geotextile) was conducted (i.e., Test T5) with the aim of quantifying the level and development of lateral soil restraint and therefore isolate the effect of trench wall on the load-displacement relationships. The results of these tests are presented in Figure 4.2 together with the results from Test T2. It can be observed that soil-pipe response from tests T3, T4 and T5 are remarkably similar and only the results from test T3 produces the nonlinear load-displacement increase after pipe displacements of about 0.6D (i.e., this is about twice the pipe displacement required for the onset of load increase after the plateau observed in Test T2).

From the results plotted in Figure 4.2, there is clearly a relationship between the variation of soil restraint with displacement and the proximity of the pipe to the trench wall (i.e., trench wall-pipe distance, S). Furthermore, effect of the distance between the pipe and trench wall seems to be limited to distances less than 1D. For the tests conducted with S greater than 1D, the soil-pipe interaction is controlled entirely by the frictional properties of the backfill soil and the soil restraint can be quantified by limit-based equilibrium approaches and a log-spiral failure surface like that proposed by O'Rourke et al (2008). Figure 4.3 illustrates the position of the log-spiral failure surface developed during test T4 (S = 2D) with respect to the geotextile-lined trench wall. The fact that the log-spiral failure surface does not intersect the trench wall indicates that the lateral restraint is governed only by the backfill soil. The log-spiral model (O'Rourke et al 2008) predicts an N_{qh} of 8.1 for the moist sand backfill (friction angle ϕ = 43°, dilation angle ψ = 12°) which is in close agreement with the UBC test values for no trench and a separation distance of 2D.

Based on Figures 4.2 and 4.3, it can be concluded that for the geotextile to reduce the soil restraint, the distance between the trench wall and the pipe has to be no greater than about 0.5D, and at that proximity to the trench wall, the pipe response behavior would be controlled by the frictional properties along the sloped trench wall.

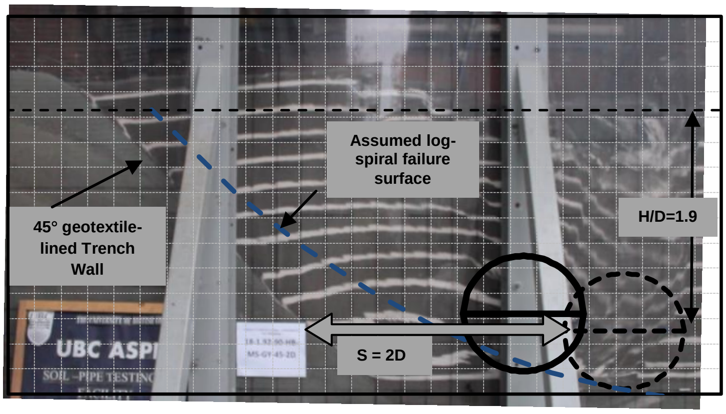

Figure 4.3 Log-spiral failure surface developed for trench wall-pipe distance S = 2D (grid 0.1 x 0.1 m).

Soil deformations for test T2 after Y'=0.05D and Y'=0.45D are illustrated in Figure 4.4.a and 4.4.b, respectively. Corresponding levels of lateral soil restraint for those pipe displacements can be obtained from Figure 4.2. As seen in Figure 4.4, clearly two modes of instability occurred for this test. As expected, the first instability mode occurred after the front passive wedge mobilizes the frictional resistance along the trench wall. After the frictional resistance has been overcome, the pipe starts to move upward with an inclination of about 30° to 35° from the horizontal. This vector appears to result from the interaction between the imposed lateral displacements and the 45° boundary condition of the sloped trench wall.

The maximum lateral soil restraint from the first failure mode is represented by the plateau observed in the load-displacement relationship of Figure 4.2 and continues until the lateral soil restraint increases again due to the proximity of the pipe to the trench wall. This is the second mode of instability and represents a soil response predominantly and likely governed by shearing of a compressed soil mass between the pipe and trench wall. This second mode also shows the development of multiple shear planes and several distinct zones of soil failure (Figure 4.4.b).

PRCI (2004) guidelines identify the use of two layers of geosynthetic fabric as a mean to reduce lateral soil restraint under the premise that a low-friction failure surface will be developed along the sloped trench wall and a passive wedge of soil, as indicated in Figure 4.4 will slide up the trench wall. The validity of this premise can be evaluated from the data presented in Figure 4.5 that shows the development of N_{qh} and the measured displacement of the outer geotextile as a function of test duration (time).

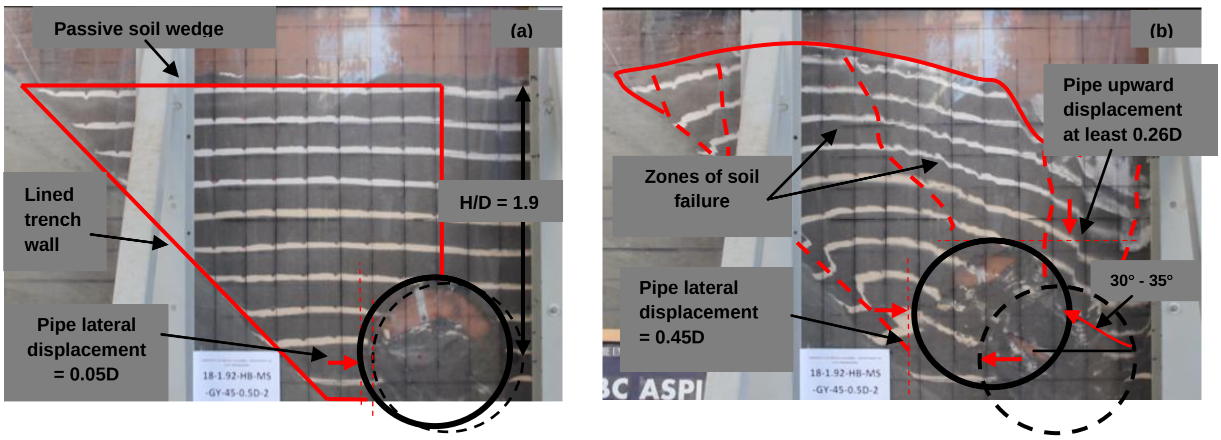

Figure 4.4 Observed soil deformations for tests with moist sand backfill: (a) after Y'=0.05D, and (b) after Y'=0.45D. Note that grid on plexiglass is 0.1 x 0.1 m.

The N_{qh} level required to overcome the geotextile friction can be obtained by identifying the time above which the outer geotextile slides. From Figure 4.5, this time is 40 s. However, the response of the soil-pipe system has not reached the plateau in terms of N_{qh}, but rather, it is still in a process of additional soil restraint development. The data from Figure 4.5 indicates that the load is not associated with simply an intact passive wedge sliding along the low-friction geotextile fabrics interface, but rather along the soil-geotextile interface, the next resistance mechanism along the trench wall surface. Further evidence of this mechanical process is presented in Figure 4.6 for tests T8 and T9.

It is reasonable to state that the horizontal resistance beyond what is necessary to induce slippage at the geotextile-geotextile interface results from the need to overcome additional energy within the deforming soil mass. Information from our tests results is not sufficient to identify the specific sources of increased work energy, but the following are reasonable candidates:

1. Energy required to overcome friction at the geotextile-geotextile interface.
2. Energy required to overcome friction at the soil-geotextile interface.
3. Energy dissipated due to deformations internally occurring within the soil (in overcoming both internal soil friction as well as shear-induced dilative response of soil).
4. Energy dissipated during relative movements between soil and pipe.
5. Energy required for movements against gravity.
6. Energy required to overcome the limitations in the test chamber in simulating ideal two-dimensional conditions.

The degree of contribution of the above factors to the observed increase in soil restraint over that is necessary to initiate sliding at the geotextile-geotextile interface is likely affected by the test configuration, the amount of pipe movement, and other factors. For example, with increasing pipe displacements, it is possible that the energy mechanisms noted in items 2 through 4 above would contribute more to the pipe-soil restraint than that corresponding to the energy required to overcome friction at the geotextile-geotextile interface. In recognition of these factors, the authors are currently pursuing further research including numerical modelling to investigate in depth the above mechanisms.

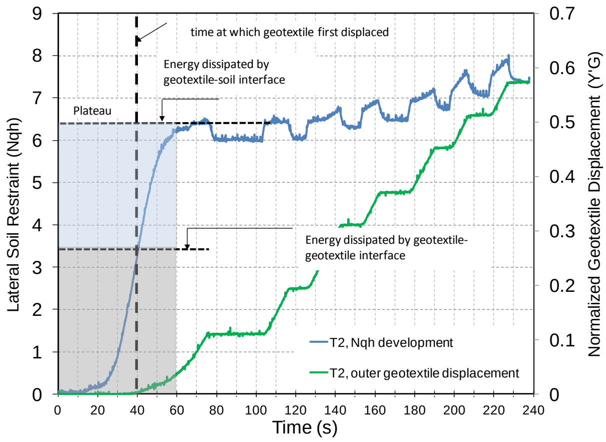

Figure 4.5. Evolution of lateral soil restraint and geotextile displacement as a function of time for T2.

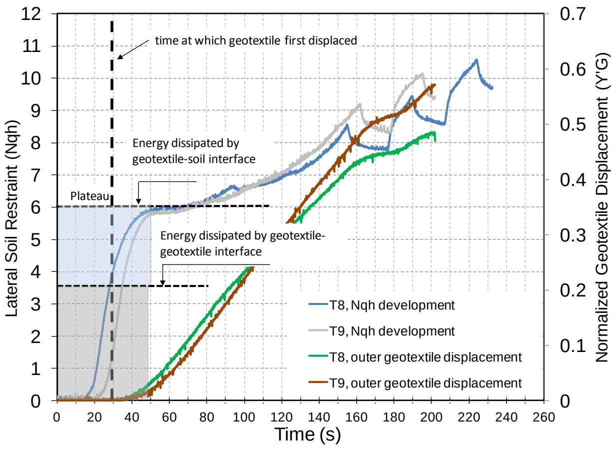

Figure 4.6. Evolution of lateral soil restraint and geotextile displacement for T8 and T9.

4.2 Tests with Sand and Gravel Backfill

The results from test on a stronger backfill material (mixture of sand and gravel) are presented in Figure 4.7. Tests T6 and T7 were carried out with a lined trench wall with a slope of 45 degrees from the horizontal; while tests T8 and T9 were conducted on a lined trench wall with a slope of 35 degrees. Tests T10 and T11 were tests without trench and their soil deformation and inferred log-spiral failure surface are illustrated in Figure 4.8. Test T7, T9 and T11 were done for repeatability purposes and quality control of the soil-pipe interaction response.

The results from Figure 4.7 display a response similar to tests with moist sand in that there is an increase of lateral soil restraint after reaching a plateau. However, the effectiveness of geofabrics in reducing the lateral load imposed on the pipe is more noticeable. A reduction of about 33% and 45% of lateral soil restraint was reached for the cases of geotextile-line trench walls of 45° and 35° at normalized pipe displacements (Y') of less than 0.2D, respectively. As soil-pipe interaction continues the reduction vanishes and as the shape of the load-displacement suggests, the lateral soil restraint can even be higher than those obtained from the plain case (N_{qh} greater than 11).

From the results of the tests with sand and gravel backfill, even though slippage at the geotextile interface occurred during the tests, the use of dual-geotextile lined trench configurations did not provide the benefit that was expected from this mitigation option. Even though the restraint was significantly reduced, indicating greater benefit than observed with the moist sand tests, this reduction begins to be disappear as the pipe approaches the trench wall (Y' ≥ 0.3D).

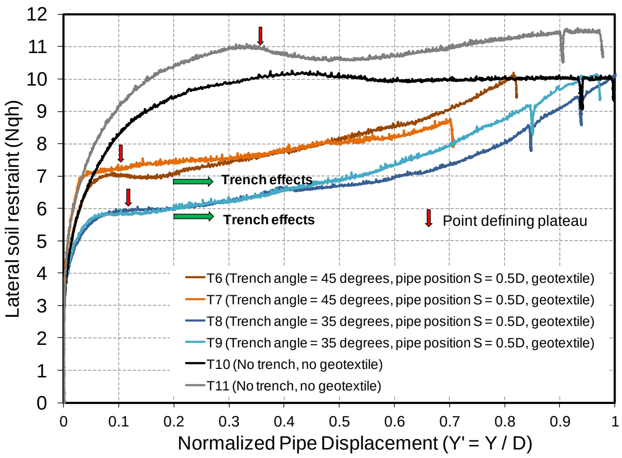

Figure 4.7. Lateral soil restraints in sloped trench walls with sand and gravel backfill.

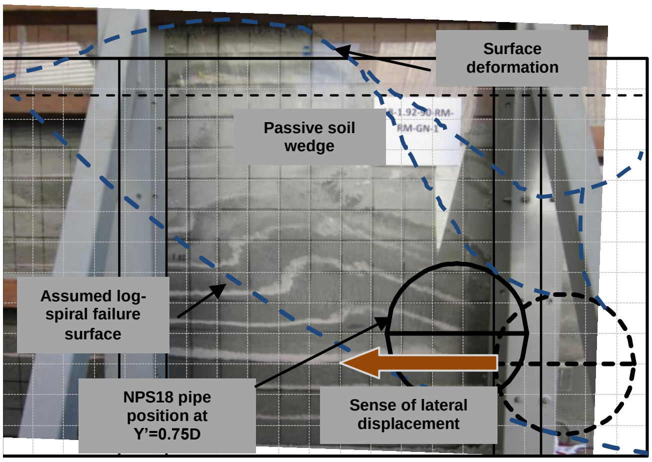

Figure 4.8. Log-spiral failure surface developed during Tests T10 and T11.

5. ANALYTICAL STUDY OF DATA TRENDS

Given the similarity in the shape of the load-displacement response observed in the tests, the development of a simple predictive tool was investigated. This led to a simple mechanical model, based on limit equilibrium considerations and trends from the data. The development is shown in equations 5.1 and 5.2 and predicts levels of lateral soil restraint (N_{qh}) for geotextile-lined pipeline trenches with Y'≤0.3D and Y'>0.3D (post-plateau), respectively.

$$N_{qh} = N_{qhI} = \frac{Y'}{\frac{0.05 \cdot Yp'}{F_y} + \frac{0.95 \cdot Y'}{F_y}} \; if \; Y' \leq 0.3D \quad (5.1)$$

$$N_{qh} = N_{qhI} + K(Y' - 0.3D)^n \; if \; Y' > 0.3D \quad (5.2)$$

where:

$$F_y = (W) * \frac{(sin\theta + tan\phi_i cos\theta)}{(cos\theta - tan\phi_i sin\theta)} \quad (5.3)$$

W is the weight of the front passive wedge, ϕ_i is the geotextile-soil interface friction angle and θ is the inclination of trench wall from the horizontal plane, as illustrated in Figure 5.1.

Y_p' is the normalized displacement at F_y (assumed Y_p'=0.1D), D is the pipe outside diameter, and Y' is the normalized pipe displacement. *K* and *n* are constants equal to 7.0 and 1.65, respectively, and were determining from fitting the data of the tests to a power law. The constants 0.05 and 0.95 in Equation 5.1 were obtained from fitting the data of the tests with Y'< 0.1D to a rectangular hyperbola.

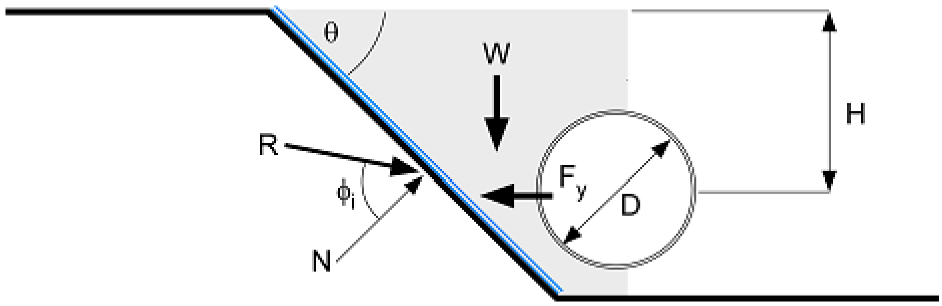

Figure 5.1 Sliding block mechanism for quantifying F_y for pipeline systems with geotextile-lined trench walls. ϕ_i is the geotextile-soil interface friction angle.

A comparison between levels of lateral soil restraint from Equation 5.1 and 5.2 and the test data is shown in Figure 5.2. As can be observed in Figure 5.2, the computed load from Equation 5.1 using the interface friction angle between the geotextile fabric layers substantially under predicts the plateau load observed during the tests. Comparison with data from Karimian et al (2006) is shown in Figure 5.3.

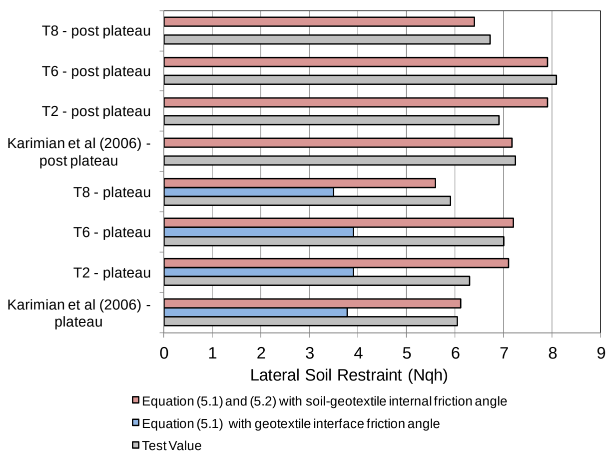

Figure 5.2. Comparison between restraints from tests and restraints from Equation 5.1 and 5.2. Post-plateau calculated at Y' = 0.5D.

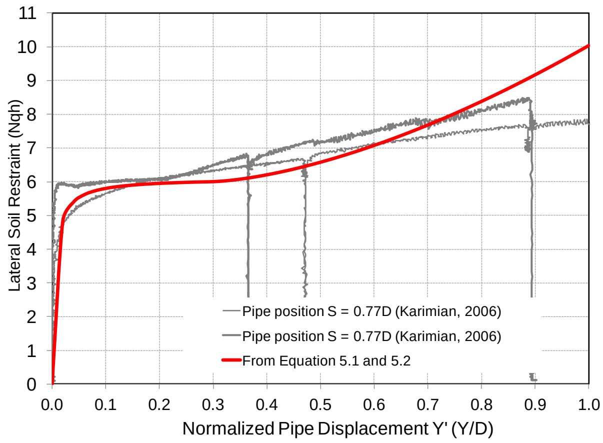

Figure 5.3. Soil restraint relationship from Equation 5.1 and 5.2 vs. data from Karimian et al (2006)

5. CONCLUSIONS

A series of full-scale, 2-D, tests were performed to evaluate the effectiveness of lining pipe trenches with geosynthetic fabrics as means to reduce lateral soil restraint on buried pipelines. Data collected from these tests included the load necessary to laterally displace the pipe specimen, pipe displacement, and displacement of layers of geotextile fabric used to line the trench side-wall. The data from these tests support the following conclusions:

- A sloped trench side-wall lined with two layers of geotextile fabric plays a role in the variation of soil restraint with pipe displacement only when the pipe-trench wall distance is less than half a pipe diameter. If the pipe is more than one diameter from the trench wall, the lateral soil restraint is simply controlled by the shear resistance of the backfill.

- The test results indicated that the lateral soil restraint from geotextile lining of the trench wall is not constant and two distinct modes of instability develop in the soil mass. One mode is temporary and can be quantified by using the sliding friction along the trench wall equal to the geotextile-soil interface friction rather than the low friction of the geotextile-geotextile interface. The second mode of failure occurs after the pipe is in close proximity to the trench wall and represents shearing in a compressed soil mass. The level of soil restraint imposed by this second failure mode could be higher than those from configurations with no trenches.

- A simple mechanical model based on limit-equilibrium and a power law, whose parameters were obtained from the full-scale test results, is proposed to quantify the levels of lateral soil restraint from the two failure modes, respectively. The predictions from the proposed analytical approach are in very good agreement with the results from the tests carried out herein and those reported by Karimian (2006). Further numerical-modeling work is being carried out at UBC to better validate the identified mechanisms.

- Lining sloped pipeline trench side-walls with two layers of geotextile fabric as a means of reducing lateral soil restraint should be discouraged or its use limited to conditions governed by small relative soil-pipe displacements that may arise from thermal expansion, subsidence, or slope creep.

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