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BURIED PIPELINES SUBJECTED TO TRANSVERSE GROUND MOVEMENT: COMPARISON BETWEEN FULL-SCALE TESTING AND NUMERICAL MODELING

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ABSTRACT

A research program has been undertaken to study the behaviour of buried steel pipelines subject to lateral horizontal ground movements, and to provide appropriate data to calibrate and validate numerical model(s). A large sand chamber (2.5 m W x 3.8 m L x 2.5 m H) available at the University of British Columbia was employed to conduct full-scale lateral pullout tests on steel pipelines, with different diameters and buried in sand simulating different overburden ratios. Numerical analyses were performed using finite-difference-method-based software with the soil response simulated using Mohr-Coulomb and hyperbolic elastic constitutive models. The input parameters for the initial computer modeling were based only on element testing results. The numerical predictions, using the two soil constitutive models, are compared with the results of lateral pullout tests. The numerical model, after validation with full-scale test results can be used to predict soil loads on pipe for different overburden ratios, pipe sizes and soil properties.

INTRODUCTION

Buried pipeline systems form an important part of lifeline infrastructures, and any disruption to the performance of these systems can result in unacceptable impacts on businesses, economies, or the living conditions of citizens. Geotechnical hazards can be a major cause of damage to these utilities, particularly as a result of unacceptable strains induced in pipelines due to permanent ground deformations. Common causes of permanent ground displacement are related to slope instability and ground subsidence, including earthquake-induced ground displacements and fault movement.

The current understanding of the response of pipelines under lateral ground movements have been developed mainly based on the experimental results from tests on anchors, or from relatively small scale models of buried pipelines. Full-scale model tests investigating this response have been limited primarily due to the considerations associated with costs and logistics; however, full-scale testing would be most preferable considering the potential errors due to scale effects arising from extrapolating the results of tests on anchors or small diameter pipe specimens.

Numerical modeling can also play an important role in the prediction of soil loads on pipelines. Again, the acceptability of numerical models ideally requires proper validation using recorded data from field case histories, or from full-scale tests.

With this background, a detailed research program involving full-scale physical modeling has been undertaken at the University of British Columbia (UBC), Vancouver, BC, Canada. Lateral pullout tests were conducted on steel pipelines, with different diameters and buried in sand simulating different overburden ratios. Numerical analyses were performed using finite-difference-method-based FLAC 2D Version 4.0 software (Itasca Consulting Company, Minneapolis, USA) mainly using soil parameters defined from element material tests. The initial analyses did not consider the results from the full-scale tests in order to provide a "baseline" prediction. Subsequent analyses were performed with analytical parameters modified to obtain a better match to the full-scale tests.

TEST APPARATUS, MATERIAL USED, AND TEST PROCEDURE

A large sand chamber (2.5m W x 3.8m L x 2.5m H) as described by Anderson et al. (2002) and shown in Figure 1 was used to perform full-scale lateral pullout tests on steel pipes having 457 mm (18") outside diameter, 13 mm (1/2") thickness and 324 mm (12.75") outside diameter, 10 mm (3/8") thickness. The length of all pipes was 2.4 m, and the surface of each pipe was prepared by sand-blasting prior to the tests. A coupling system consisting of 3-bolt end clamps at the each end of the pipe and double-ended hook cables, as shown in Figure 2, was used for pulling the pipes buried in sand to the desired depth. Each cable was connected to a load cell mounted on the actuator rod, and the pipes were pulled in a displacement-controlled manner using two 500 kN (~100 kips) actuators, commanded by a MTS Inc. (Eden Prairie, MN, USA) manufactured control system to provide synchronized displacement of both ends of the pipe.

Figure 1. Sand Compartment

The soil used for the testing program was locally obtained Fraser River sand, which has been extensively tested and documented during numerous laboratory research programs performed at UBC in the past. The average grain size, D_{50}, of the sand is 0.3 mm, with a coefficient of uniformity, C_u of 1.5. The minimum particle size is 0.074 mm and minimum and maximum void ratios for the material are 0.62 and 0.94, respectively. The sand used in the tests was essentially dry, with average moisture content below 1.0%.

Figure 2. Coupling System

The sand was placed in the chamber in approximately 200 mm lifts and mechanically compacted using a static roller to achieve a target average soil density of 1600 kg/m^3, corresponding to a relative density (D_r) of approximately 70%. A pre-calibrated nuclear densitometer was used regularly to check the as-placed density of soil. The average density for presented test matrix here was 1594 kg/m^3 with standard deviation of 16.6 kg/m^3. Figure 3 graphically shows the distribution of measured densities.

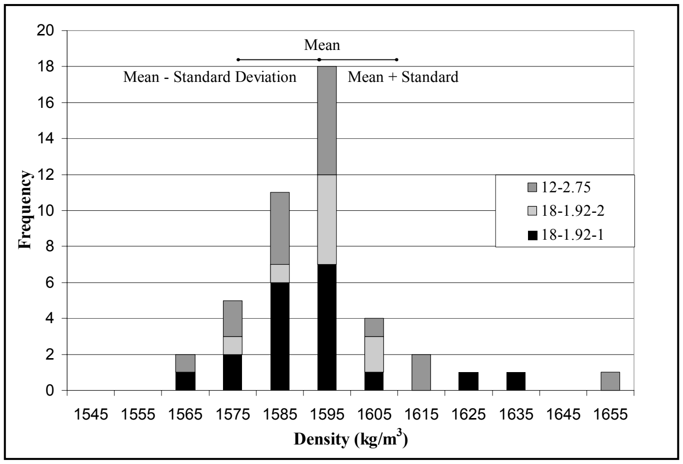

Figure 3. Distribution of Measured Soil Density

After initial placement of sand to a thickness of about 450 mm, the pipeline specimen was placed on the soil bed with its axial direction parallel to the shorter dimension of the chamber. The filling of the box was continued up to the level that corresponds to the desired overburden ratio.

Three lateral pullout tests representing some of the overburden (H/D) ratios and pipe sizes found in typical pipeline installations, as summarized in Table 1, were performed on buried pipes in compacted Fraser River sand [Note: H = Soil cover above springline of pipe; D = pipe diameter]. As may be noted, pipes buried at the overburden ratios of 1.92 and 2.75 were tested with the two different pipe sizes, 324 mm and 457 mm. The test configuration with 1.92 overburden ratio on the 457-mm diameter pipe was performed twice to confirm repeatability of the tests.

During pipe pullout, the rate of lateral pipe displacement was gradually increased from 0 to 5 mm/s over the first 10 mm of pipe movement. This rate was kept constant for the next ~200 mm of the pipe displacement. At this stage, the pipe was unloaded and then reloaded and the rate of pulling increased to 10 mm/s to observe potential effects of pulling rate. The load at each end of the pipe was recorded using the two load cells. In addition to the actuator rod displacement, the displacement at each end of the pipe was measured directly using string potentiometers mounted at the back of the soil chamber. Data were collected at a frequency of 10 Hz through a 16 channel data acquisition system.

Test ID -----	D mm	H/D -----	Density kg/m^3	Std. Dev. kg/m^3
18-1.92-1	457	1.92	1592	15.8
18-1.92-2	457	1.92	1594	22.2
12-2.75	324	2.75	1595	13.3

Table 1. Lateral Pullout Test Program

TEST RESULTS

The measured lateral load (per unit length of pipe) versus displacement response for the four tests are given in Figure 4. The total load on the pipe was obtained by summation of the measured loads on each load cell. At a given displacement, the recorded readings of the two load cells were found to be within 5%, thus, indirectly confirming the symmetry of the prepared test configurations. The displacement plotted in Figure 4 was obtained by averaging the measured displacements at each end of the pipe, which were essentially identical indicating good synchronization of movement between the two actuators.

As can be seen from Figure 4, the peak lateral load was mobilized within the first 100 mm of pipe displacement, and it essentially remained constant during the pullout. The results indicated that unloading and subsequent reloading of the pipe has no significant effect on the pullout load. It is also reasonable to state that the faster rate of displacement used in reloading also did not affect the pullout load.

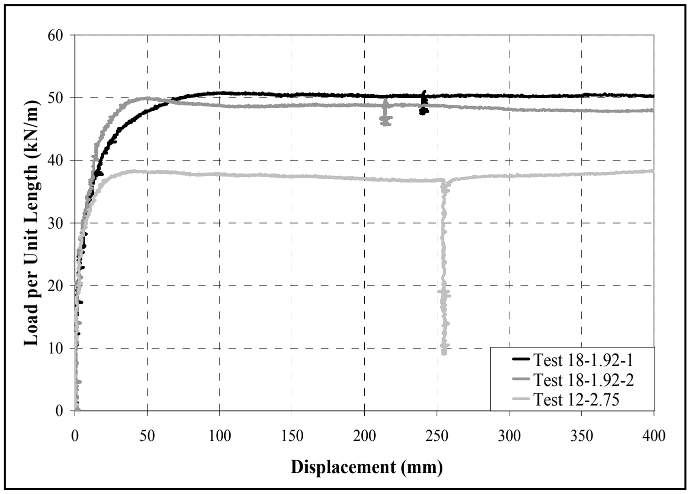

Figure 4. Lateral Load vs. Displacement

The above tests results can also be examined in terms of a dimensionless load (F_L') and dimensionless displacement (Y') as defined in Equations (1) and (2), as per previous research (Hansen 1961, Audibert and Nyman 1977, Rowe and Davis 1982, Trautmann and O'Rourke 1983, and Paulin et al. 1998).

$$F_L' = F / (\gamma.H.D.L) \qquad (1)$$

$$Y' = Y / D \qquad (2)$$

Where:

F = pullout load
γ = soil density
L= length of pipe
H = height of soil over pipe springline
D = pipe diameter
Y = pipe displacement

The results presented in Figure 4 are replotted in Figure 5 in the form of F_L' vs. Y'. As may be noted, in order to focus on the initial part of the load displacement curve, the X-axis has been limited to Y' = 0.3 during plotting. The result from Figure 5 suggests that the peak dimensionless load F_L' increases with an increase in H/D ratio from 1.92 (test 18-1.92) to 2.75 (test 12-2.75).

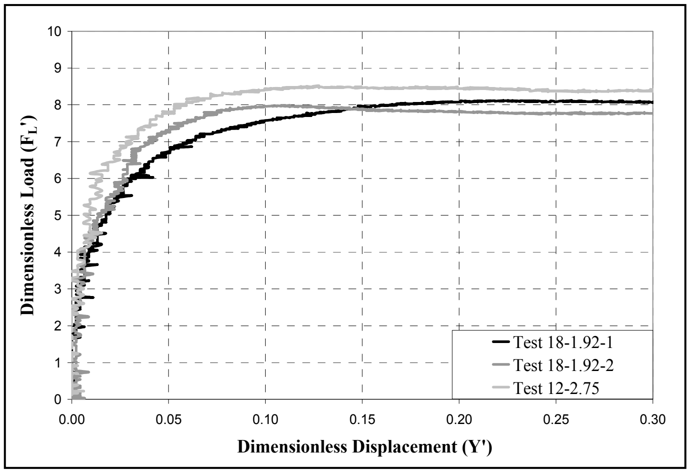

Figure 5. Dimensionless Load (F_L') vs. Dimensionless Displacement (Y')

NUMERICAL MODELING

Material properties of Fraser River Sand for numerical modeling

The ideal approach in the validation of the numerical model would be to obtain soil parameters from independent tests, and then use those parameters to predict the response of the lateral pipe pullout test. With this objective in mind, input parameters derived from element testing were used for the initial numerical predictions (using FLAC 2D Version 4.0). The material properties required for the numerical analyses included: peak and constant volume friction angles, dilation angle, elastic parameters (E ,G , and υ), and interface frictional parameters between sand backfill and pipe.

A value of constant volume internal friction angle for the Fraser River sand was obtained from the data directly available from the previous research work undertaken at the University of British Columbia (Uthayakumar, 1996; Sivathayalan, 2000).

Interface friction angle between steel and sand were obtained by testing a coupon of flat sand-blasted steel against Fraser River sand in a direct shear box.

Considering the relatively low overburden stress levels encountered in the lateral pullout test configurations, and recognizing the lack of available shear test data for such stress levels, a series of triaxial tests were conducted on dry Fraser River sand to assess the shear response of the soil at stress levels ranging from 15 to 50 kPa. The tests were conducted on specimens prepared by placing sand in layers in a cylindrical mold and tamping them to achieve target packing densities comparable with those achieved in the pipe-soil testing chamber (average dry density between 1575 to 1665 kg/m^3). More details on the testing performance and results will be presented in a thesis currently under preparation. The results of triaxial testing also provided an opportunity to directly estimate the deformation modulus values required for numerical modeling (i.e., initial shear modulus G, and bulk modulus B).

The soil parameters derived as per above are summarized in Table 2.

Parameter	Unit	Value	Comment
Internal friction angle (φ_{peak})	deg.	45.5° to 43.0°	for $\sigma'_3 = 15$ to 50 kPa, respectively
Constant volume friction angle (φ_{cv})	deg.	32.0° to 34.0°	
Interface friction angle	deg.	36.0° and 30.5°	peak and constant volume respectively
Shear modulus	MPa	3 to 12	for $\sigma'_3 = 15$ to 50 kPa, respectively

Table 2. Soil Paramters

Numerical model Development

A typical element mesh used for the numerical analysis is shown in Figure 6. As may be noted, the lateral pipe pullout is modeled in a 2-dimensional manner, and the geometry shown in Figure 6 represents the experimental set up used in tests 18-1.92-1 and 18-1.92-2. The mesh representing the three tests had 1174 elements. The mesh configuration was selected after reviewing the results from several alternate mesh configurations.

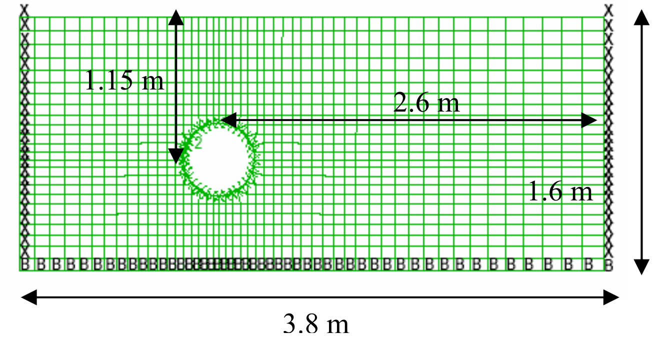

Figure 6. Mesh Generated for H/D = 1.92 and D = 46cm

Structural beam elements were used to represent the pipe. The structural elements were separated from the soil using "unglued" interface elements. Coulomb shear strength criterion limits the shear force at the interface, where a specified average interface friction angle of 33° was used, as per direct shear test data. Interface dilation angle was assumed to be 3° based on the relationship in equation (3) proposed by Bolton (1986). This assumption was determined to be adequate based upon sensitivity analyses that demonstrated negligible differences in computed results for other assumed interface dilation angles.

$$\psi = \frac{\varphi_{peak} - \varphi_{cv}}{0.8} \qquad (3)$$

where:

φ_{peak} = peak friction angle
φ_{cv} = constant volume friction angle

The soil backfill was modeled using bi-linear Mohr-Coulomb and hyperbolic soil constitutive models.

I: Analysis Using Bi-Linear Mohr-Coulomb Model

In the case of the simple Mohr Coulomb model, a peak friction angle of 45° was assumed. This value was chosen as peak friction angle based on data from triaxial tests conducted at stress levels comparable to those computed for the initial stress levels around the pipe. The dilation angle (ψ) of the material was assumed to obey Bolton's relation as presented in equation [1]. A value of ψ equal to about 14° was computed for a peak friction angle (φ_{peak}) of 45° and constant volume friction angle (φ_{cv}) of 34° for Fraser River sand. As for elastic characteristics of the soil, Poisson's ratio of 0.3 is assumed. The elastic modulus (E) for the Mohr Coulomb model was taken as the secant elastic modulus (E_{secant}), which can be considered as a fraction of the initial elastic modulus (E_i) using equation (4). As suggested by Byrne et al. (1987), a value of $\beta \sim 0.3$ was used and the value of E_i was assessed from triaxial test results assuming a stress level corresponding to minor principal stress at the springline of pipe.

$$E_{secant} = \beta . E_i \qquad (4)$$

A comparison between the computed dimensionless load using the assumed Mohr-Coulomb model for the soil response and experimental results for tests 18-1.92-1 and 18-1.92-2 is presented in Figure 7. As can be seen, with constant friction angle and elastic modulus, the Mohr-Coulomb model seems to overpredict the soil loads on pipe. Moreover, the load displacement curve from the analysis reaches the peak value and plateau at a much earlier stage of dimensionless displacement, Y' of about 0.03 compared to a Y' of about 0.10 for the full-scale tests.

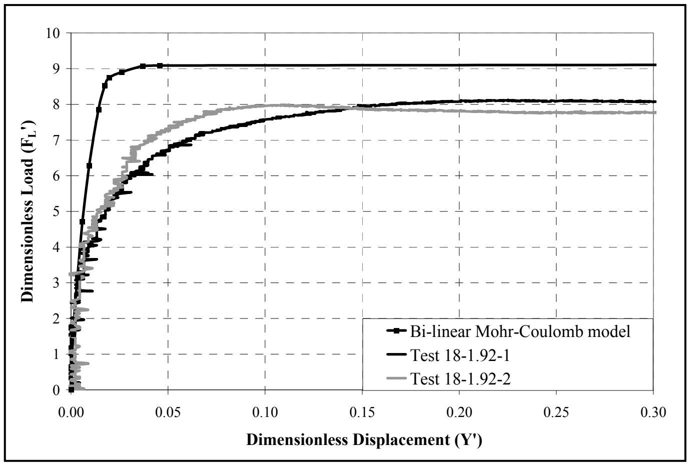

Figure 7. Comparison Between Numerical Model Results and Test Results

II: Analysis Using Hyperbolic Model with considering stress dependancy of material

In the hyperbolic model (Duncan and Chang 1970), the non-linear soil behaviour is accounted using elastic parameters of $G_{tangent}$ (and $E_{tangent}$), as described in equation [5].

$$G_{tangent} = G_i \times (1 - R_f \cdot \frac{\tau}{\tau_{Failure}})^2 \qquad (5)$$

where:

$G_{tangent}$ = tangent shear modulus
G_i = initial shear modulus
R_f = failure ratio
$\tau_{failure}$ = shear stress at failure
τ = shear stress at the current stage

The failure ratio is an experimental value that varies between 0.6 to 1.0 for different materials. In the hyperbolic model, knowing the stress conditions and friction angle of each element, $\tau_{failure}$ is calculated using Mohr-Coulomb failure criterion:

$$\tau_{failure} = \frac{\sigma'_3 \times \sin\varphi}{1 - \sin\varphi} \qquad (6)$$

where:

σ'_3 = minor principal stress
ϕ = friction angle

In the current model, to consider the stress dependency of material properties, initial elastic properties of the material and peak friction angle were updated with the minor principal stress for each element using data obtained from triaxial tests.

The lateral pullout response of a pipe-soil system having D=457 mm and H/D=1.92, was computed using the hyperbolic model, and the results are shown in Figure 8. In order to observe the effect of failure ratio on the model results, the pullout response for three different R_f values; 0.7, 0.8 and 0.9 are presented in Figure 8. The corresponding experimental results (tests 18-1.92-1 and 18-1.92-2) are also plotted in Figure 8 for comparison. In a similar manner, the comparison between numerical and experimental results for the lateral pullout case with D=324 mm and H/D=2.75 (test 12-2.75) is shown in Figure 9. Clearly, the numerical model, with soil represented using the hyperbolic model, seems to capture the non-linear pipe pullout response much better than the bi-linear Mohr-Coulomb relationship. Examining Figure 8 and Figure 9, it can be seen that the R_f value controls both peak load and rate of mobilization of soil load on the pipe. In the current study, after trying several values, it appeared that using a failure ratio of 0.9 provided the best fit between the computer results and the test results. Selection of $R_f = 0.9$ also matches with Duncan's suggestion for silica sand.

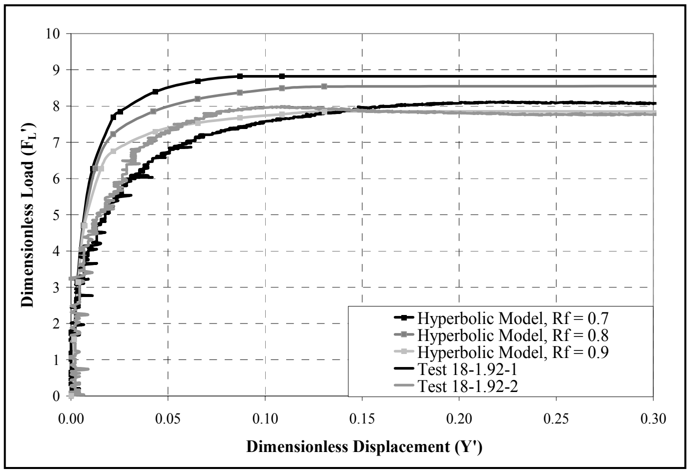

Figure 8. Comparison Between Numerical Model Results and Test Results

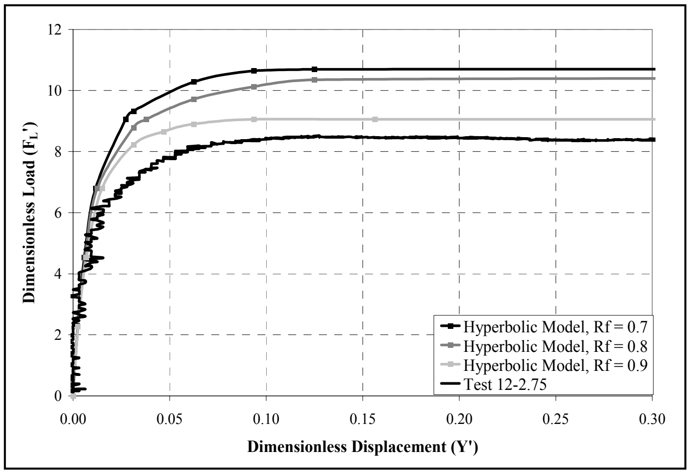

Figure 9. Compariosn between numerical model results a nd test results

CONCLUSION

In this paper, the results of three full-scale tests conducted to study the performance of buried pipelines subjected to transverse ground movement are presented. The tests were performed using a large sand chamber available at the University of British Columbia, Vancouver, BC, Canada. A finite-difference-based numerical model was developed to capture the pipe performance in full-scale tests. It is intended to use the validated numerical model for the prediction of soil loads on pipes during lateral soil displacements for a variety of material properties and geometric configurations.

Comparisons between results from tests and numerical modeling indicate the importance of the selection of a proper constitutive model for representing soil behaviour. The assumption of bi-linear Mohr-Coulomb soil behaviour seems to result in a prediction where the peak soil load is reached at much smaller pipe displacements compared to that observed from full scale tests.

A hyperbolic nonlinear model, in which stress dependency of material stiffness and peak friction angle are included, seems to acceptably capture the load-displacement behaviour observed in full-scale tests. The peak load and the slope of the load-displacement curve were found to be significantly dependent on the selected value of failure ratio (R_f).

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