Centrifuge Modeling of Laterally Loaded Pipelines

F. Poorooshasb, M. J. Paulin, M. Rizkalla, and J. I. Clark

The state of practice (SOP) for pipeline design in areas where pipelines may move relative to the soil involves considering the pipeline to be made up of discrete segments and the segments to be coupled to the soil via a set of spring/sliders. Much of the theory behind this SOP is derived from other geotechnical applications such as pile/soil interaction. There is little or no physical verification of the mechanisms or the magnitude of forces assumed during pipeline displacement. An experimental model examination of displaced pipelines using the centrifuge modeling technique to create similitude between model and prototype or the actual situation is presented. The SOP, the experimental program, and the results of eight pipeline model tests are presented. The results are discussed, with particular reference to the magnitude of loads transmitted to the pipes and the development of the pipeline/soil interaction. The test results are compared with the loads that would be predicted by the SOP design calculations. The main conclusion is that the SOP formulation appears to be unconservative, predicting loads acting on the pipeline about 50 percent lower than those measured experimentally.

When a buried pipeline is subjected to ground movement such as a landslide or downslope creep of soil, the pipe's integrity and operating safety are both of concern. Computer-based analyses are the primary engineering tools available to evaluate the state of the pipeline to determine the need for remedial or mitigative action. In finite element modeling techniques used in pipeline analyses, the interaction between the pipe and soil is commonly described by spring elements. The parameters describing these spring elements have generally been assumed from other soil/structure interaction studies (e.g., anchors and piles), and much of the experimental and analytical work cited in pipeline analysis has been undertaken from a foundation design perspective.

Review of the literature has indicated that there is little realistic pipeline-specific experimental information available. Studies carried out by the NOVA Corporation of Alberta have shown that typical rates of ground movement for creeping type landslides experienced by the industry range from less than 1 to 6 cm/year. Lateral pipeline/soil experiments reported are generally small in scale, ignore construction considerations such as the presence of a distinct backfill material, and generally use idealized soils. Experimental work in this field needs to be extended.

A simple engineering analysis of the pipeline/soil interaction problem can be expressed as (*1*)

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$$P_{ult} = DC_uN \quad (1)$$

where

P_{ult} = ultimate load transferred to the pipe,
D = pipeline diameter,
C_u = undrained shear strength of the soil, and
N = interaction factor.

The objectives of this experimental program were to examine the phenomenon of pipeline/soil interaction, and, specifically, to determine the value of N, the shape of the load-displacement curve, and the effect of ditch width and depth on the interaction.

LATERAL PIPELINE/SOIL INTERACTION

Modeling Considerations

Several considerations arise in undertaking either physical or numerical modeling studies of lateral pipeline/soil interaction. Figure 1 shows a laterally displaced pipeline and indicates the various aspects of the problem that present modeling complexities. Modeling of the soil separation behind the pipe and both the contact surface and the soil strain hardening or softening in front of the pipe pose numerical modeling difficulties. Choosing an appropriate rate of pipeline displacement against the soil is another significant modeling consideration (depending on whether the pipeline is loaded by a creeping soil or a landslide condition).

Development of SOP Formulations

Rowe and Davis (*2*) simulated the behavior of vertically oriented smooth anchors in saturated clay using elastoplastic finite element analyses. For such anchors, the limiting loading cases are those in which the back of the anchor either remains in contact with the surrounding soil (no breakaway condition) or breaks away immediately (immediate breakaway condition).

A monograph by the Committee on Gas and Liquid Fuel Lifelines (CGL) (*3*) concluded that pertinent data on laterally displaced pipelines in clay indicate a trend toward increased levels of ultimate load until H/D (depth to pipe's springline divided by diameter) reaches a value of 6. Furthermore, the CGL suggests that the Hansen bearing capacity model (*4*) can be used to estimate the maximum horizontal pipeline force per unit length in clay.

The SOP formulations routinely used in design are based on the work of Rowe and Davis (*2*) and the CGL (*3*) guidelines. In a later section, the results of the centrifuge program will be compared with these formulations.

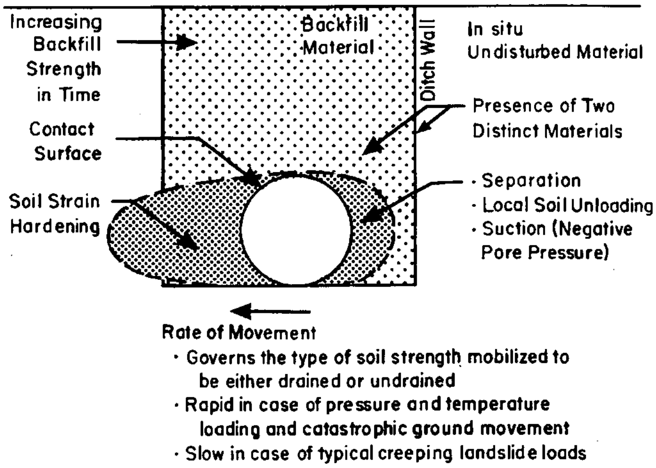

FIGURE 1 Aspects of lateral pipeline/soil interaction that present modeling complexities.

CENTRIFUGE MODELING

Centrifuge modeling has been used for several decades for geotechnical investigations. Its current stage of development is considered by many in the geotechnical engineering community to be comparable with the developmental stage of numerical modeling two decades ago (*5*). As an experimental method, it has been widely accepted in Europe and Japan and is being increasingly used in North America.

The centrifuge modeling technique allows gravitational effects to be replicated by substituting the centripetal acceleration experienced by an object in circular flight at the end of a rotating beam for true gravitational acceleration. The rationale behind this modeling technique is as follows. If an actual earth structure is represented by a model manufactured of the same material to a scale of N (every linear dimension in the prototype being N times greater than in the model), the stress levels due to self-weight will be N times greater at any position in the actual structure than at the corresponding point in the model. However, if the model is subjected to an acceleration N times greater than gravitational acceleration, the stress distribution in the actual structure and in the model will be identical. The strain fields will also be identical, since the constitutive laws governing the soils are the same. Furthermore, if any external loadings are applied, they must be scaled to maintain correspondence of stress fields. If these conditions are met, the reaction of the model to the external loading will be identical to the actual structure's behavior and will provide a valuable understanding of the deformations and failures involved in actual events (*6*).

A theoretical treatment of the basis of centrifuge modeling is beyond the scope of this paper and is presented elsewhere (*5,7,8*). The scaling relationships for a modeling program are presented in Table 1.

EXPERIMENTAL PROGRAM AND RESULTS

Model Development

In small-scale modeling, three situations need to be considered. The first is the actual situation of interest to the modeler. The second is the model itself. Both of these items are tangible situations. The third item is the prototype, which refers to the actual situation that the small-scale model represents. The prototype is a theoretical situation and is determined by applying the correct scale factors to all aspects of the model. The similitude of the prototype to the actual situation determines the relevance of the small-scale modeling to reality. Figure 2 shows the actual situation of interest, the prototype, and the centrifuge model tested at 50 *g* (1:50 scale). The centrifuge model (described later) behaves at prototype scale like an extremely rigid pipeline section in soft kaolin clay. The degree of similarity of this prototype to a particular case of industrial interest determines the relevance of the experiments to the state of practice.

Experimental Description

The experimental program was conducted at the Laboratoire Central des Ponts et Chaussées (LCPC), Centre de Nantes. The facility houses an Acutronic 680-1 centrifuge, which has an effective radius of 5.5 m and a payload capability of more than 2 tonnes at 100 *g*.

The experimental program consisted of two centrifuge tests: Test Set A and Test Set B. A scale factor of 1:50 was chosen for

TABLE 1 Centrifuge Modeling Scaling Relationships

Quantity	Full Scale (Prototype)	Centrifugal model at N g's
Linear Dimension	1	1/N
Acceleration	1	1
Velocity	1	N
Stress (Force/Area)	1	1
Strain (Displacement/Unit Length)	1	1
Density	1	1
Frequency	1	N
Mass	1	$1/N^3$
Force	1	$1/N^2$
Displacement (Distance)	1	1/N
Time (Drainage)	1	$1/N^2$
Time (Dynamic)	1	1/N
Time (Viscous)	1	1

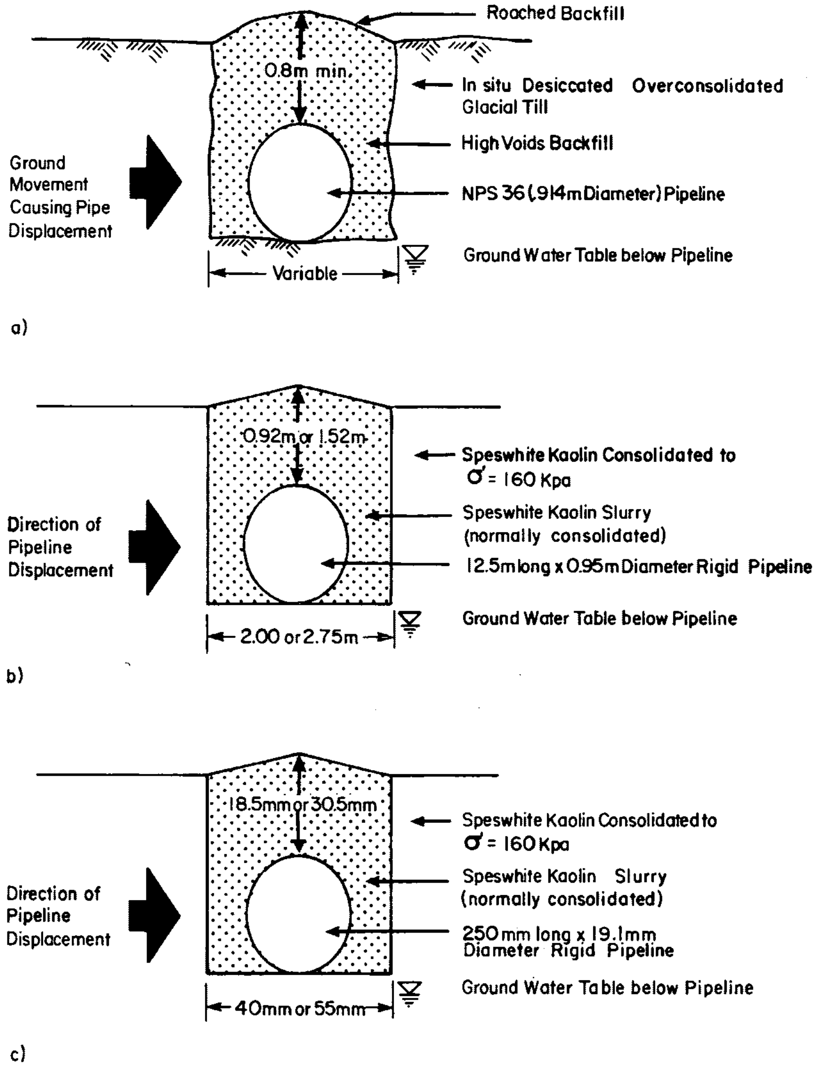

FIGURE 2 (*a*) **Actual event,** (*b*) **prototype event, and** (*c*) **model event.**

the current experimental program. This scale allows four pipelines to be tested during each test set. During the experiments, conducted at an acceleration level of 50 *g*, the rigid model pipelines (1:50 scale) were moved laterally through overconsolidated kaolin clay. The pipelines had been placed in model trenches at 1 *g*; these trenches were then backfilled to simulate construction procedures. As the pipelines were displaced, force displacement curves were obtained.

Experimental Apparatus and Setup

The model used to obtain the present results is shown in Figure 3. The equipment was contained in a standard LCPC strongbox, 800 by 1200 mm in internal plan and 360 mm deep. The central section of the box contained kaolin clay, which was retained by two bulkheads. Trenches were carved in the clay to the required width and depth to contain the model pipelines. Clay slurry was used to backfill the trenches after the pipelines had been positioned. Each of four pipelines was pulled through the clay by a pair of tension cables, which were connected to a prime mover, a DC variable speed gear drive, by means of pulleys mounted on a shaft. The pipelines were displaced at a nominal speed of 1 mm/sec. Data were collected before, during, and after the displacement of each pipeline.

During the test, the water level was kept constant by using a weir. The soil sample was probed with a miniature cone penetrometer to obtain the shear strength value required in Equation 1. After the tests, the sample was extruded from the strongbox and sectioned on the laboratory floor. Water content samples were obtained, and internal inspection was undertaken to determine displacement patterns within the soil.

Prototype

The prototype of the model described earlier is a system of four pipeline segments buried in overconsolidated kaolin clay. Before

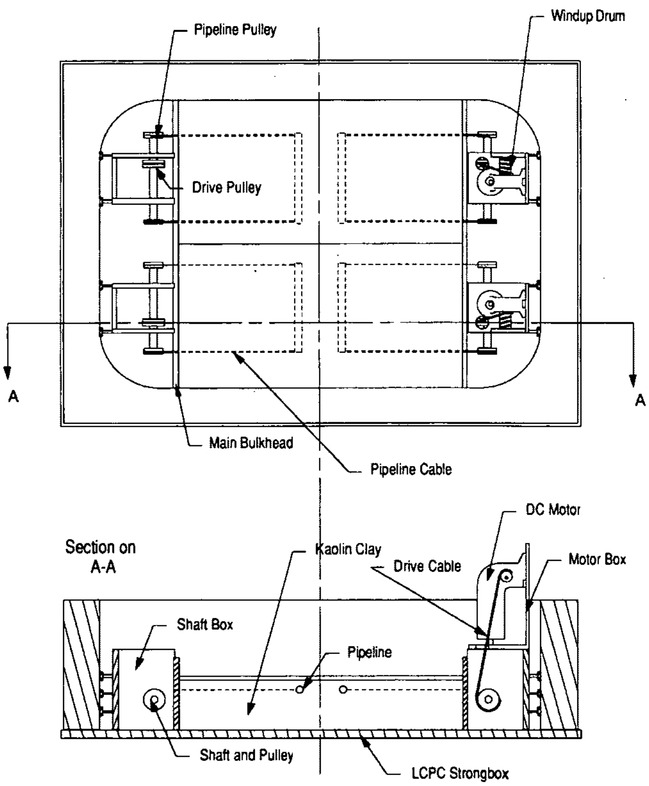

FIGURE 3 Package used in test series. DC motors are omitted from left-hand side of the package.

consolidation in the centrifuge, the clay was one-dimensionally preconsolidated to a vertical effective stress of 160 kPa. The backfill material was also kaolin clay, which was normally consolidated in the centrifuge.

The stainless steel pipelines were 0.95 m in diameter and 12.5 m long. They were pulled by a pair of stainless steel cables 0.158 m in diameter. At one end of each pipeline, an electrical cable approximately 0.25 m in diameter was pulled through a lubricated plastic channel.

The pipelines were pulled at a slow rate, so that the event was practically undrained and inertial events were insignificant. Two extremes of velocity were recorded during the same pipeline movement (Test Set A, Pipeline 4), and no significant effect on the soil response could be seen in the recorded data. The pipelines were pulled horizontally but were free to move vertically. The movements at either end of each pipeline were constrained so that they were equivalent.

The base of the pipeline trenches was approximately 5 m above a hard impermeable surface. There was 0.92 m of clay cover above each shallow pipeline and 1.52 m of cover above each deep pipeline. The distance between the pipelines was 5.48 m, and the distance from the end of the pipelines to the lateral vertical wall was 4.77 m. The water table was 3.1 m above the hard impermeable surface, about 1.9 m below the base of the pipeline trenches. Below the water table, the soil was saturated. Above the water table, various degrees of desiccation had occurred; this resulted in a lack of certainty about the effective stresses within the region above the water table. The trench geometry for each test is presented in Table 2.

TABLE 2 Trench Geometry and Undrained Shear Strength Measurements

Test Set	Test	Trench Width	Trench Cover Depth	C_u In Situ (kPa)	C_u Backfill (kPa)	Evidence of Desiccation
A	Pipeline 1	2.00m	0.92m	22.0	17.34	Strong
A	Pipeline 2	2.00m	1.52m	13.80	12.54	Strong
A	Pipeline 3	2.75m	0.92m	17.61	14.64	Strong
A	Pipeline 4	2.75m	1.52m	10.24	9.73	Weak
B	Pipeline 1	2.00m	0.92m	8.50	7.29	Weak
B	Pipeline 2	2.00m	1.52m	8.50	7.29	None
B	Pipeline 3	2.75m	0.92m	8.29	7.29	None
B	Pipeline 4	2.75m	1.52m	8.29	7.29	None

Review of Experimental Procedures

The measured shear strength of the in situ material was lower than expected from empirical correlations for 100 percent saturated kaolin clay. As a result, a distinct difference in strength between the in situ and backfill materials was not achieved. This obscured the effects of the ditch width. The shear strengths of the soil interpreted at the base of the pipelines are presented in Table 2. The cone penetrometer test locations from Test Set A are presented in Figure 4.

Below the water table, the soil was saturated. Above the water table some desiccation had occurred, so the magnitudes of the effective stresses within this region could not be determined exactly. To reduce desiccation during the second test, a layer of thin plastic film was used to cover the surface of the soil. Weights were placed on the perimeter of the film, and a slit was made in the film at the points where the boreholes were to be placed. The film reduced desiccation but did not eliminate it.

Pipeline Force Displacement Records

During Test Set A, of the four pipelines, Pipelines 1, 2, and 3 were displaced at nominally constant rates. Pipeline 3 was accelerated at the end of its displacement, but this appeared to have very little effect on the results. Pipeline 4 was displaced initially at a very low rate of 3.7×10^{-7} m/sec (prototype scale) and then stopped (some stress relief occurred). After a period of 30 min (52 days, prototype scale), excitation was reapplied and the pipeline began moving at a rate that varied between 1.08×10^{-8} and 7.3×10^{-6} m/sec. Figure 5 indicates that the effect of the loading being stopped and then reapplied was negligible.

The general features of each curve are as follows. The initial response is one of increasing force with displacement. In Pipelines 3 and 4, the peak load is followed by a decrease in load and then a slow increase. The peak is observed between a pipeline movement of 0.25 to 0.5 diameters, and the subsequent decrease in response is at the minimum at approximately the point where the pipeline first touches the native material. The subsequent rise is observed as the pipeline begins to penetrate the native material. This peaked behavior is more marked than it was in the other two tests. Table 3 summarizes the results from these tests.

Examination of Table 3 indicates that there is some scatter in the displacement required to peak resistance, with extremes of 0.26 and 0.63 m. This is a function of different shapes of the peak. Pipelines 3 and 4 display much sharper peak behavior than

Pipelines 1 and 2. This may have been because the effect of the softer backfill was more pronounced in the wider trenches. A more consistent measurement of displacement to peak loads may be obtained by assessing the displacement required to achieve 90 percent of the peak load. This quantity may be expressed as

Displacement to 90 percent peak load

$$= 0.21 \text{ m} \pm 18 \text{ percent} \tag{2}$$

The peak load is achieved when the pipeline is still totally within the trench, so the native material may not be a factor in calculating pipe loading. The 18 percent spread in this displacement value is also associated with the degree of desiccation (i.e., the greatest distance to 90 percent peak load is displayed by the most desiccated samples, Pipelines 1 and 3). The ratio of peak resistance to shear strength at the springline falls within a fairly narrow band. In all cases, the peak resistance is attained before the pipeline begins to enter the native material; therefore, the backfill shear strength is a suitable normalizing factor. The ratio, as measured in Test Set A, is given by

$$\frac{\text{Peak resistance per unit length}}{\text{backfill shear strength}} = 11.70 \pm 11 \text{ percent} \tag{3}$$

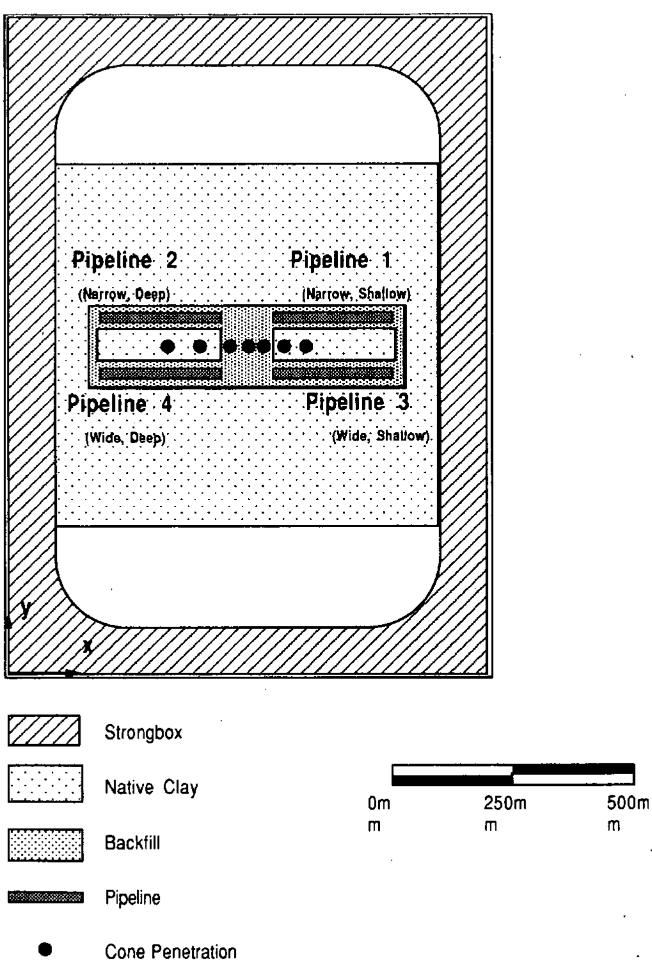

FIGURE 4 Test Set A geometry showing CPT locations.

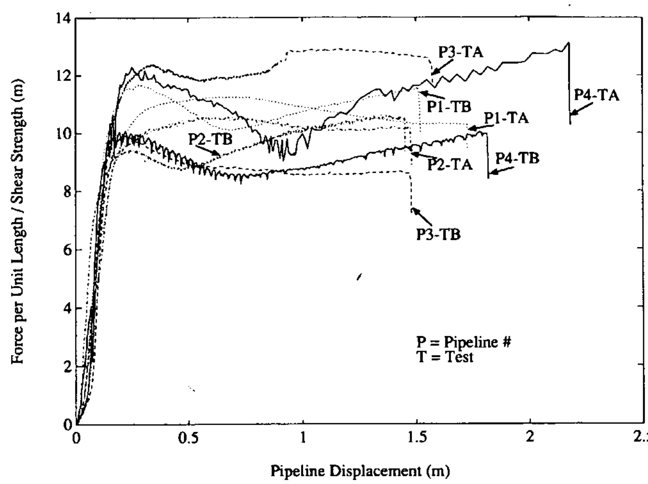

FIGURE 5 Normalized prototype force/displacement data, all tests.

The lower values in this range are those associated with the narrow trenches. This is contrary to intuition. It might be explained by a lower degree of consolidation of the backfill material in the narrow trenches (because of friction against the rigid pipeline). The effect, however, is not marked enough to draw a firm conclusion.

Also contrary to intuition, it is not obvious from this test alone that trench cover depth has any significant effect. Deeper pipelines would be expected to display a greater resistance to displacement, but this is not apparent.

The results obtained in Test Set B were similar to those obtained in Test Set A. The results are presented in Table 4 and Figure 5. The peak in resistive load was noticed in all pipelines; a postpeak minimum and subsequent rise occurred in all pipelines except Pipeline 3. In Test Set B, pipelines were displaced at a much more uniform speed. The maximum speed was 6.5×10^{-6} m/sec (prototype scale), and the minimum was 1.25×10^{-6} m/sec. A variation of less than 5 percent was maintained for all tests.

The peak loads normalized against shear strength (of the backfill) can be represented by

$$\frac{\text{Peak resistance per unit length}}{\text{backfill shear strength}} = 10.01 \pm 11 \text{ percent} \tag{4}$$

and the distance to 90 percent of the peak resistance is given by

Displacement to 90 percent peak load

$$= 0.15 \text{ m} \pm 12 \text{ percent} \tag{5}$$

Overall, for all data sets, the normalized peak resistance is given by

$$\frac{\text{Peak resistance per unit length}}{\text{backfill shear strength}} = 10.86 \pm 20 \text{ percent} \tag{6}$$

and the distance to 90 percent of the peak resistance can be expressed by

Displacement to 90 percent peak load

$$= 0.18 \text{ m} \pm 39 \text{ percent} \tag{7}$$

TABLE 3 Summary of Test Set A Pipeline Results

	Pipeline 1	Pipeline 2	Pipeline 3	Pipeline 4
Peak Resistance per Unit Length (kN/m)	196	131	182	118
Normalised Peak Resistance (m)	11.30 (8.91)	10.45 (9.49)	12.43 (10.34)	12.13 (11.52)
Post Peak Minimum Resistance (kN/m)	179	127	174	88
Normalised Minimum Resistance (m)	10.32 (8.14)	10.13 (9.20)	11.89 (9.88)	9.04 (8.59)
Distance to Peak Resistance (m)	0.61	0.63	0.34	0.26
Distance to 90% of Peak Resistance (m)	0.25	0.20	0.21	0.19
Distance to Minimum Resistance (m)	1.73	1.21	0.57	0.91
Evidence of Desiccation	Strong	Strong	Strong	Weak
D_d, Distance to Peak Resistance per Unit Length = 8.263 C_u*D	0.151D	0.071D	0.119D	0.131D

(The figures in brackets refer to the native material)

DISCUSSION OF RESULTS

Force-Displacement Curves

The equations presented in an earlier section of the paper show the normalized peak loads and the distance to 90 percent peak load. It can be seen that the uncertainty bands are fairly low; however, the mean values for the two different tests (for both displacement and load) vary widely. This indicates that the results of these tests must have been influenced by desiccation, the only factor that varied considerably between tests.

The initial portions of the traces all coincide (see Figure 5). At a normalized resistance of about 9 C_u, the displacement curves begin to diverge, but before this point they are essentially the same curve. This might be the zone of elastic deformation, but the displacements at this stage are rather large (0.15 m, prototype scale) for this to be the case.

A theoretical means of predicting the stresses on a pipeline due to relative motion is thus given by this linear portion. If a benchmark nondimensionalized, normalized resistance [peak resistance per unit length/(C_u * pipeline diameter)] of approximately 8.3 is considered, this normalized resistance will be reached when the displaced distance, D_d, is approximately 0.125 pipeline diameters (0.12 m). This would give an indication of the correct point for remedial action. D_d values for each of the tests are presented in Tables 3 and 4.

Comparison of Centrifuge Modeling and SOP

Table 5 compares the backcalculated interaction factor N of Equation 1 from the centrifuge tests with the recommended factors of Hansen (*4*) and Rowe and Davis (*2*). For the conditions modeled, the Hansen interaction factors tend to underestimate the magnitude

TABLE 4 Summary of Test Set B Pipeline Results

	Pipeline 1	Pipeline 2	Pipeline 3	Pipeline 4
Peak Resistance per Unit Length (kN/m)	85	69	73	74
Normalised Peak Resistance (m)	11.66 (10.00)	9.47 (8.12)	10.01 (8.81)	10.15 (8.93)
Post Peak Minimum Resistance (kN/m)	74	64	62	72
Normalised Minimum Resistance (m)	10.15 (8.71)	8.78 (7.53)	8.50 (7.48)	9.89 (8.68)
Distance to Peak Resistance (m)	0.27	0.23	0.21	0.22
Distance to 90% of Peak Resistance (m)	0.16	0.15	0.15	0.13
Distance to Minimum Resistance	0.69	0.46	1.06	0.74
Evidence of Desiccation	Weak	None	None	None
D_d, Distance to Peak Resistance Per Unit Length = 8.263 C_u*D	0.119D	0.156D	0.140D	0.115D

(The figures in brackets refer to shear strengths in the native material)

TABLE 5 Comparison of Experimentally Derived Interaction Factors and SOP Factors

Test Set	Pipeline	Evidence of Desiccation	Experimental Interaction Factor (N) Normalized to: Backfill Strength	In Situ Strength	Average Strength	CGL Recommended Interaction Factor (N) Based on Hansen (1961)	Rowe and Davis (1982) Interaction Factors (N): Assuming Immediate Breakaway	Assuming No Breakaway
A	1	Strong	11.9	9.4	10.5	5.5	4.3	9.5
A	2	Strong	11.0	10.0	10.5	6.0	4.7	11.1
A	3	Strong	13.1	10.9	11.9	5.5	4.3	9.5
A	4	Weak*	12.8	12.1	12.4	6.0	4.7	11.1*
B	1	Weak*	12.3	10.5	11.3	5.5	4.3	9.5*
B	2	None*	10.0	8.5	9.2	6.0	4.7	11.1*
B	3	None*	10.5	9.3	9.9	5.5	4.3	9.5*
B	4	None*	10.7	9.4	10.0	6.0	4.7	11.1*

of loads transferred to the pipeline by nearly 50 percent. Similarly, the Rowe and Davis factors assuming "immediate breakaway" conditions significantly underestimate the loads transferred to the pipe.

There is agreement between the experimentally derived interaction factors and the Rowe and Davis factors for the "no breakaway" conditions for the five tests marked with an asterisk in Table 5. Reasonable agreement (within ± 20 percent) was achieved in these cases. The development of such a condition was unlikely for the following reasons:

1. Rowe and Davis note in their paper that tension cracking for shallow anchors and pipes (embedment ratio less than 3) as modeled in the present study would be expected to relieve suction and reduce the soil's bearing resistance. This would lead to a condition nearer to the "immediate breakaway" limit of their formulation.
2. The experimental setup precluded the development of suction behind the displaced pipeline by introducing air through the conduit around the pulling cables and in turn through the partially hollow pipe section.
3. The very large pipe displacements probably dissipated the suction that might have developed at the onset of the displacement. This would have been reflected as a postpeak drop in the measured load-displacement response. No significant postpeak drops in resistance were observed.
4. The values of interaction factors in the asterisked tests are very similar to those measured for the first three tests, in which strong physical evidence of breakaway was observed. It should follow that similar interaction factors would arise from similar breakaway conditions.

The practical implications of the SOP formulations underestimating the magnitude of ground movement induced loads being transferred to a buried pipeline are significant. Whereas further testing is certainly required to verify the results of this preliminary study, indications are that the lateral pipe/soil interaction inputs to pipeline stress analysis based on the presently accepted formulations may lead to errantly favorable assessments of the integrity and operating safety of pipelines in unstable ground conditions. Such errant assessments may contribute to decisions delaying necessary remedial strain-relieving operations.

SUMMARY AND CONCLUSIONS

Eight model pipelines were tested in a centrifuge at 50 *g* to investigate the soil response to laterally displaced buried pipelines. The experimental program is relevant to studies being carried out by pipeline engineers to determine the effects of slowly creeping slopes on buried pipelines. The two primary objectives of this experimental investigation were to obtain force-displacement curves for the buried pipelines and to ascertain the effects of trench width and cover depth.

As a result of the experimental program and comparison with SOP, the following conclusions were reached:

1. Centrifuge modeling is a suitable technique for determining the soil response to laterally loaded pipelines.
2. For model pipelines 0.95 m in diameter, average peak loads per unit length of 11.7 and 10.0 times the shear strength of the soil were recorded for desiccated and saturated soils, respectively. The lateral displacements required to develop 90 percent of the peak loads were 0.21 and 0.15 m.
3. The loads predicted by the SOP are approximately half of those measured in this test program. The current SOP may be unconservative.
4. For most samples, the distance required to develop a peak resistance per unit length of approximately $8.3C_u * D$ (a suitable benchmark) was approximately 0.125 pipeline diameters.
5. The effects of trench geometry were negligible compared with effects due to variations in soil properties.
6. A further test program is required to elucidate the effects of trench geometry when there is a significant difference between backfill and native material properties and to investigate the effect that rate (and hence drainage) has on the lateral load transfer to a pipeline.
7. Future refinements to pipe/soil interaction modeling should use centrifuge modeling to complement numerical modeling work.

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