Centrifuge modeling of buried continuous pipelines subjected to reverse faulting
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Abstract: Seismic ground faulting is a severe hazard for continuous buried pipelines. Over the years, researchers have attempted to understand pipeline behavior mostly via numerical models such as the finite element method. The lack of well-documented field case histories of pipeline failure due to faulting along with the costly and complex facilities needed for full-scale experimental simulation make a centrifuge-based method to determine the behavior of pipelines subject to faulting the best method to verify numerical approaches. This paper presents results from four centrifuge tests investigating the behavior of continuous buried steel pipelines that were subjected to reverse faulting. The axial and bending strains induced in a pipeline are presented. Also investigated is the influence of factors such as faulting offset, burial depth, and pipe diameter on the axial and bending strain of pipelines and on the ground soil failure and pipeline deformation pattern. Finally, the initial strain at the wrinkling point of the pipe under reverse faulting is studied and compared with theoretical values. It was found that the pipe deformation mechanism and damage type are significantly altered by variations in pipe diameter, burial depth, and pipe section slenderness ratio (diameter to thickness ratio). Increasing the diameter and burial depth of a pipe changes the deformation mechanism from beam buckling to wrinkling. The wrinkling strains from these tests are in good agreement with the findings of Hall and Newmark.

Key words: centrifuge models, buried pipeline, reverse faulting, earthquake, permanent ground deformation.

Résumé : La fissuration du sol suite à un séisme est un danger important pour les pipelines enfouis continus. Depuis plusieurs années, les chercheurs ont tenté de comprendre le comportement des pipelines principalement par des modèles numériques, comme la méthode par éléments finis. Le manque de cas réels bien documentés de bris pipeline en raison de la fissuration du sol, ainsi que les coûts et les installations complexes nécessaires pour la simulation expérimentale à grande échelle, font que la méthode basée sur la centrifuge est la meilleure méthode pour vérifier les approches numériques servant à déterminer le comportement de pipelines soumis à la fissuration du sol. Cet article présente des résultats de quatre essais par centrifuge servant à étudier le comportement de pipelines enfouis continus soumis à la fissuration inverse. Les déformations axiales et en torsion induites dans le pipeline sont présentées. L'étude a aussi évalué l'influence de facteurs tels le déplacement de la faille, la profondeur d'enfoncement et le diamètre du tuyau sur les déformations axiales et en torsion du pipeline, la rupture du sol et le patron de déformation du pipeline. Enfin, la déformation initiale au point de plissage du tuyau sous fissuration inverse est étudiée et comparée aux valeurs théoriques. Il a été démontré que le mécanisme de déformation du tuyau et le type de dommage sont affectés significativement par les variations du diamètre du tuyau, de la profondeur d'enfoncement et du rapport d'élancement (ratio diamètre sur épaisseur) de la section de tuyau. L'augmentation du diamètre et de la profondeur d'enfoncement d'un tuyau change son mécanisme de déformation de flambage à plissage. Les déformations en plissage observées durant ces essais concordent bien avec les conclusions de Hall et Newmark.

Mots-clés : modèles par centrifuge, pipeline enfouis, fissuration inverse, séisme, déformation permanente du sol.

[Traduit par la Rédaction]

Introduction

There are three types of earthquake faults: strike-slip, normal, and reverse (Fig. 1). In some cases, a strike-slip fault may combine with a normal or reverse fault. This is known as an oblique fault. Faulting can endanger pipelines essential to the supply of oil, gas, and water. Damage recorded from previous earthquakes include damage to the water supply network during the 1906 San Francisco earthquake, to buried pipelines in Taiwan caused by normal faulting during the 1999 Chi-Chi earthquake, the 3 m offset of a 2.2 m diameter pipeline caused by strike-slip faulting during the 1999 Izmit earthquake in Turkey, and the buckling of water supply pipelines during the 1990 Manjil earthquake in Iran caused by reverse faulting (Fig. 2).

Much research has been carried out to investigate pipe behavior under seismic faulting because many of these pipes are lifelines that maintain essential services. Analytical research was first carried out by Newmark and Hall (1975), Kennedy et al. (1977), and Takada et al. (2001). Several numerical studies were conducted in recent decades. Ariman and Lee (1991) evaluated pipe strain using a finite element model studying the effect of pipe diameter, burial depth, and soil friction on pipe bending strain. Meyersohn (1991) inves-
tigated pipe strain using Unipip finite element software. Liu and O’Rourke (1997) conducted numerical and analytical research on the behavior of a continuous pipeline.

However, it is necessary to validate the research results by comparing them to the observed behavior of pipelines. The lack of recorded case histories has made physical modeling of a pipeline subjected to faulting the best validation method (Ha et al. 2008a). Takada (1984) implemented the first large-scale physical model to investigate segmental pipeline behavior subjected to normal faulting. O’Rourke et al. (2005) carried out large-scale and centrifuge-based modeling (Ha et al. 2006) of pipelines subjected to faulting. These studies mainly investigated pipe response due to strike-slip faulting and, to some extent, covered normal faulting with right deformation angles (Ha et al. 2008b). Similar studies were conducted by Yoshizaki et al. (2001) in US–Japan cooperative research.

The need to study the behavior of continuous steel pipes subjected to normal and reverse faulting was the major incentive for the present study. A fault simulator was designed and constructed to simulate normal and reverse faulting. Results of a modeled pipeline subjected to reverse faulting are presented. Pipe response to normal faulting and details of a fault simulator have been presented elsewhere by the authors (Rojhani et al. 2011a, 2011b).

### Modeling equipment and preparation

The centrifuge tests were designed to study the behavior of buried pipelines subjected to reverse faulting. All tests were implemented at a gravity level of 40g. Table 1 is a summary of the specifications for the four centrifuge tests. The relations between different parameters in the model and the prototype are shown in Table 2.

#### Fault simulator specifications

The simulator (FSUT-RN 60.8) had outside dimensions of 102 cm × 76 cm × 68 cm (length × width × height) and a weight of 2300 N. The soil split container was 96 cm × 70 cm × 23 cm. The deformation angle of the fault simulator was 60° (β = 60°). The maximum allowable offset of the simulator was ±4 cm, simulating ±2 m offset at 50g. The driving system contained a 5 t hydraulic jack placed horizontally under the soil container. Components of FSUT-RN 60.8 are shown in Fig. 3. The maximum offset rate of the simulator was 30 cm/s on the prototype scale. Details of the FSUT-RN 60.8 fault simulator have been presented elsewhere by the authors (Rojhani et al. 2011a, 2011b).

#### Soil and pipe specifications, model preparation

The test model specifications are shown in Table 1. All tests reported herein were conducted on stainless steel-316 pipes that comply with ASTM (2004) standard A999/A999M. The pipe–fault angle was 90° in all tests and the pipes had a $D/t$ ratio of either 20 or 50. These ratios were based on gas pipelines affected by faulting during the Imperial Valley earthquake in 1979, such as the Holtville–El Centro line with a $D/t$ ratio of 22.8 and line 6001 with a 54 $D/t$ ratio (Roth et al. 1990). In the test model, the pipe was connected to the simulator body using a fixed connection (Fig. 4).

It must be noted that, in practice, the $D/t$ ratio is often more than 20 for water, oil, and gas pipelines. However, modeling a pipe that simulates one with a $D/t$ ratio of more than 50 would require the production of a pipe with a very thin wall thickness (0.2 mm or less), which is impossible in practice.

When it is possible to model a sufficient length of pipe, the pipe supports do not affect pipe response and the connection type is not significant. This is because pipe–soil interaction does not affect the pipe beyond the anchored points. In other words, when the end connections of the pipe model are beyond the anchored points on the two sides of the pipe, the connection type does not affect the response.

Based on the scaling law in the centrifuge, 1 m of model pipe in the centrifuge at a 40g acceleration will be equal to 40 m at the prototype scale. The analytical approach states...
Table 1. Summary of test models used in centrifuge testing (all dimensions in prototype scale).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test label</th>
<th>Peak offset (m)</th>
<th>Acc. (g)</th>
<th>$\alpha$ (°)</th>
<th>$\beta$ (°)</th>
<th>$H/D$ (m)</th>
<th>$D/t$ (mm)</th>
<th>$D$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8S4R-2.8S</td>
<td>2.8</td>
<td>40</td>
<td>90</td>
<td>60</td>
<td>2.8</td>
<td>0.88</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>8S4R-6S</td>
<td>2.8</td>
<td>40</td>
<td>90</td>
<td>60</td>
<td>6</td>
<td>1.9</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>25S5R-2S</td>
<td>2.8</td>
<td>40</td>
<td>90</td>
<td>60</td>
<td>2</td>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>25S5R-0.8S</td>
<td>2.8</td>
<td>40</td>
<td>90</td>
<td>60</td>
<td>0.8</td>
<td>0.8</td>
<td>50</td>
</tr>
</tbody>
</table>

Note: Acc. = acceleration.

Table 2. Scaling laws for centrifuge testing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model/prototype</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>$1/N$</td>
<td>$L$</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
<td>$ML^{-1}T^{-2}$</td>
</tr>
<tr>
<td>Acceleration</td>
<td>$N$</td>
<td>$LT^{-2}$</td>
</tr>
<tr>
<td>Axial rigidity</td>
<td>$1/N^2$</td>
<td>$MLT^{-2}$</td>
</tr>
<tr>
<td>Flexural rigidity</td>
<td>$1/N^4$</td>
<td>$ML^3T^{-2}$</td>
</tr>
</tbody>
</table>

Note: $N$, scaling factor.

that the axial pipe strains are nonzero at 40 m; thus, pipe response in these models may be affected by the connection type. For such models, the pipe-end condition should be semi-rigid, meaning that it will be neither a fully fixed nor fully hinged joint. The pipe is neither fixed, without deformation, nor free because resistance of the remaining part of the pipe prevents free deformation. Because the rate of pipe deformation at the end of the pipe model is unknown, the simulation of a semi-rigid joint is not possible. However, realistic pipe behavior may be realized by considering the limit boundary conditions of fully fixed and fully hinged cases. Fixed connections were modeled in this study for simulation because they are most likely to simulate real conditions. Modeling by hinge connections should be a topic of future investigations on the realistic behavior of pipelines.

In centrifuge modeling, particle-size effect is an important factor in achieving accurate simulation. Here, Firozkoh-191 sand was used. The grain-size curve is shown in Fig. 5 and the specifications of Firozkoh-191 sand are shown in Table 3. The average grain diameter ($D_{50}$) was 0.16 mm. The lowest ratio for the pipe diameter for $D_{50}$ soil ($D/D_{50}$) is 50, which satisfies the criterion of $D/D_{50} \geq 48$ recommended by the International Technical Committee TC2 (2005). The soil moisture content was assumed to be 4.5%–5%, which represents common field conditions. Moist soil was placed inside the soil container in 4 cm layers that were then compacted using a 19.1 N steel hammer with 425 J/m² input energy per unit area. Such compaction led to approximately 85% soil relative density.

Two types of sensors were used to monitor the response of the pipeline subjected to faulting: a strain gauge and a displacement transducer. Twenty-six strain gauges were arranged at seven stations along the pipe in the longitudinal and circumferential directions. The longitudinal gauges were attached at the top, bottom, and spring line. All strain gauges were set in a quarter bridge configuration to allow separation of axial strain from bending strain along the pipeline. Axial strains were calculated as the average of the longitudinal strains at the top and the bottom of the pipe. Bending strains were calculated as one-half the difference between the longitudinal strains at the top and the bottom of the pipe. Displacement transducers measured axial displacement of the pipe at the ends, vertical displacement of the pipe in profile, and fault displacement.

Results and discussion

Post-offset ground surface and pipe deformation

Ground surface, section deformation, and pipe deformation are shown in Figs. 6–9. The deformations occurring along the pipe and at the ground surface during faulting were significant for the four tests. In some cases, the pipe erupted from the ground and, in others, no disturbance occurred at the ground surface (Fig. 6). The similarity of faulting conditions caused similar ground surface deformation within a band at the fault and transverse cracks (parallel to the fault) on the ground surface. Different ground surface deformation and longitudinal cracking patterns were observed within a band above and parallel to the pipeline.

If a pipeline was buried at a shallow depth (low $H/D$), the pipe plowed upward and was exposed at the surface during faulting (Figs. 7a, 7d, 8a, 8d). However, the deformation mechanism was affected by variations in pipe diameter and relative stiffness ($D/t$). The pipe with a lower $D/t$ ratio deformed in a beam buckling mechanism (Figs. 7a and 8a). Such deformation is a preferable outcome as the potential for tearing of the pipe wall is lower. Beam buckling is the basic type of deformation to allow the pipeline to remain in service. The deformation curve is nearly symmetric in a beam buckling mechanism.

When the pipe diameter and stiffness were larger (test 4), the pipe encountered local buckling or wrinkling. Wrinkling occurred on the footwall side in the fault plain direction (Fig. 7d, white dotted line). The ground deformation pattern was the main cause of the wrinkling point occurring on the footwall side. The hanging wall portion of the simulator had no displacement; however, the footwall moved downward. Thus, the pipe moved upward on the footwall side and downward on the hanging wall side. This occurred because greater soil stiffness underneath the pipe limited deformation along the hanging wall. Along the footwall side, the pipe had more deformation because of the lower stiffness of the soil above the pipe. Note that large curvature in the pipe wall at the wrinkling point often led to circumferential cracking of the pipe and leakage (Fig. 9). However, this deformation was observed in an unpressurized pipe and given that pipe deformation is affected by internal pressure, it is likely that the rate of cracking and pipe rupture would increase due to internal pressure. It is suggested that future studies of these models include internal pressure.
Fig. 3. Components of fault simulator (FSUT-RN 60.8). All dimensions in centimetres.

Fig. 4. Pipe-end connection to fault simulator body.

Fig. 5. Grain-size distribution of Firoozkooh-191 sand.

Increasing the length of a model pipe increased the possibility of beam buckling deformation. Keeping the length of pipe constant changed the deformation mechanism of the pipe as the pipe diameter and stiffness increased. Varying burial depth also led to changes in the deformation mechanism. Increasing burial depth resulted in increased soil stiffness around the pipe, preventing it from emerging from the ground. This made the pipe subject to local buckling and deformation below the surface (Figs. 7b, 7c, 8b, 8c). The critical cover depth for a change in the pipe deformation mechanism from beam buckling to local buckling was determined by Meyersohn (1991) (Fig. 10). Using the $h/D$ ratio of a pipe on the horizontal axis and by selecting a suitable soil compaction curve, the critical cover depth can be determined from the vertical axis. The shaded areas in Fig. 10 correspond to different degrees of backfill compaction.

Test 2 repeated the first test at a greater depth, resulting in the pipe deforming into an unfolded S-shape below the surface. Concentrated bending occurred on both sides of the fault. Along the hanging wall side, concentrated bending caused the pipe to move upward to the ground surface, where it sometimes broke the surface. In contrast, concentrated
bending caused the pipe to move downward along the footwall. The amount of upward movement along the hanging wall was greater than the downward movement along the footwall because of the difference in soil stiffness between the upward and the downward movements. In this test, local buckling did not occur and the pipe was not wrinkled because of the high relative stiffness ($D_i$) of the pipe.

In test 3, a pipe of larger diameter was buried deeply so that the soil prevented pipe exposure and the pipe deformed below the surface. The deformation pattern for this test was similar to that of test 2, but local buckling and wrinkling occurred at two points and the pipe deformed into an unfolded Z-shape (similar to an unfolded S-shape, but angular rather than curved) because of the lower relative stiffness of the pipe. At the two points where plastic hinges formed, the pipe encountered circumferential cracking and possible leakage. Figure 9 shows the wrinkled parts of the pipe. Local buckling occurred 4–5 m from the fault along the footwall and within 8–9 m from the fault along the hanging wall at the prototype scale.

In test 3, the part of the pipe that was thrust above the surface showed less upward movement relative to pipes with smaller diameters. This was the result of wrinkling and the greater diameter of the pipe, which increased soil–pipe interaction. Increasing pipe diameter increased passive soil resistance, which prevented pipe deformation. Also, the mobilization of passive soil resistance led to pipe deformation that was concentrated at one or two points and caused greater damage to the pipe. Note that increasing pipe diameter caused upward movement of the pipe to decrease, but longitudinal soil cracks became larger.

**Axial strain**

Axial strain along the pipeline versus fault offset is shown in Fig. 11. The highest offset applied during the tests was 2.3 m at the prototype scale. In these curves, where a local phenomenon such as wrinkling occurred and accurate strain measurement was not possible, the uncertainty is shown as "unknown" by the cross-hatched areas.

In tests 1 and 2, there was no considerable axial strain observed and the predominant pipe reaction was bending during faulting. In the early stages of faulting, axial strain reached a constant value of about 0.15%; however, as faulting offset increased and pipe buckling commenced, axial strain tended to
Fig. 7. Post-test pipe and soil section deformation (left view): (a) test 1; (b) test 2; (c) test 3; (d) test 4.

Pipeline

Pipeline

Pipeline

Pipeline

decrease to nearly zero. In test 2, when the pipe deformed into an unfolded S-shape, slight changes occurred in axial strain at the point of maximum curvature of the pipe. Changes at this point may have been more extensive, but were not accurately recorded because there was no strain gauge at the point of maximum curvature. This prompted the curve to be defined as unknown at this position.

At the early stages of faulting in test 3, the axial strain curve shows a trend similar to that of tests 1 and 2, where axial strain along the pipe was constant. However, the difference was the onset of pipe buckling caused by the 0.6 m faulting offset. Axial compression strains were first concentrated at the curvature points. As deformation and buckling commenced, axial tension strain occurred on the pipe. The maximum recorded value for axial strain was 0.8%.

If pipe wrinkling occurred in tests 3 and 4 at the points where a strain gauge was installed, the strain variation during wrinkling was recorded and shown as a peak in the curve. However, a limited number of strain gauges along the pipe made accurate recording of the strain difficult and strain curves are approximate between the gauge stations. In contrast to what is observed on the curve, the strain gradient is actually larger than the measured strain because of the use of a limited number of strain gauges placed on the pipe surface. In other words, if the measured strain curve is similar to the left curve of Fig. 12, the realistic strain curve may be similar to right curve. This is because wrinkling is a localized phenomenon and affects a very small portion of the pipe. Also, in test 3, large changes occurred with wrinkling on the hanging wall side that were not recorded because there were no strain gauges at the wrinkling point. This deficiency has been accommodated by the use of cross-hatched areas along the curve (Fig. 11).

In test 4, the pipes rapidly experienced bending and local buckling because of the shallower depth and lower HID ratio. Increasing the fault offset increased the axial tensile strain on the pipe. During the test, the peak strain was 0.5% and remained constant. Based on the observations after the test, this seems to have resulted from the separation of the strain gauge from the pipe surface. In this test, axial strain at the ends of the pipe was also affected by offset. It increased because of the shallow burial depth while, in test 3, the ends of the pipe were not affected.

A comparison of the four tests revealed that increasing burial depth delayed the onset of pipe deformation for a given DI/h ratio. Also, the axial strain was the predominant strain of the pipe in the larger offset range.

Figure 13 shows the variation in peak axial strain during testing. As can be seen, increasing pipe diameter resulted in a large increase in axial strain. Increasing burial depth also increased axial strain; however, the effect of increasing burial depth on the peak strain was much less than the effect of pipe diameter. It can also be seen that the strain gradient decreased as burial depth increased, and the peak axial strain was attained at a greater offset. Moreover, Fig. 13 shows that the wrinkling of the pipe began at 0.9 and 0.3 m faulting offset in tests 3 and 4, respectively, because the strain trend reversed after these points.
Bending strain

Bending strain along the pipe is shown in Fig. 14. The bending strain curves are of two types: convex and double curvature. The double curvature is concave on one side of the fault and convex on the other. When the direction of pipe displacement is upward, the bending strain is convex and vice versa. Similar to Fig. 11, the uncertainty in the curve stemming from the lack of strain gauges is denoted as “unknown” by the use of a cross-hatched areas.

The bending strain curve along the pipe experiencing beam buckling deformation (test 1) is shown in Fig. 14a. It can be seen that the bending strain curve is symmetrically similar to the pipe deformation profile. However, the slight asymmetry observed on the footwall side was caused by the deformation angle of the fault plain and the direction of pipe movement.

Along the footwall side, the pipe tended to emerge from the soil surface. After pipe lifting, the overburden weight led to bending in the pipe. This resulted in a slight fall in the
curve along the footwall while, on the hanging wall side, the entire pipe rested on a homogeneous soil bed with high stiffness and no changes occurred.

When beam buckling took place, the maximum bending strain occurred at the middle of the pipe. As shown in Fig. 14, this strain was 0.8% and a strain of 0.6% was recorded at the end of the pipe. Because a semi-fixed pipe connection is the condition that best represents real-life situations, strain at the middle of a pipe with a semi-fixed connector is expected to increase and buckling to occur at a lower offset. Research on analytical methods for beam buckling deformation has been done by Marek and Daniels (1971), Hobbis (1981), Kyriakides et al. (1983), Ariman and Lee (1989), and Meyersohn (1991).

Figure 14b shows the bending strain curve for test 2. It can be seen that the bending strain curve along the pipe is asymmetric with two peaks and different curvature. Along the footwall, the pipe tended to blow upward and become exposed at the ground surface because the footwall portion of the simulator moved downward relative to the hanging wall portion, and the deformation angle of the fault was 60°. The overburden soil caused bending in the pipe in the opposite direction from the bending movement due to faulting. However, the faulting and overburden effects along the hanging wall side were in the same direction. Thus, the bending strain reached 1.2% on the hanging wall side, but was close to zero on the footwall side at the end of the pipe. Also, the peak bending strain was located ~3 m from the fault on the footwall side. The peak bending strain along the hanging wall side decreased because of pipe moment at the end of the pipe. It can be seen that the deformation formed an unfolded S-shape beginning 0.6 m from the faulting offset.

The strain curves at 0.3 m offset for tests 1 and 2 show a 200% increase in peak strain in test 2 that was caused solely by burial depth. In other words, increasing pipe burial depth accelerated the rate of bending strain.

The bending strain curve of test 3 is shown in Fig. 14c. Tests 3 and 4 were conducted using larger diameter pipes with a larger slenderness ratio (D/π) to investigate the effects of pipe diameter and stiffness. In this test, the pipe deformed into an unfolded Z-shape with wrinkled curvature points. Generally, the changes in the curve, particularly up to 0.6 m offset, were similar to test 2, but the bending strain for a pipe with a greater diameter was smaller than for the smaller diameter pipe.

The results for test 3 were similar to those of test 2; the considerable bending movement was not absorbed by the pipe connector on the footwall side. On the hanging wall side, considerable bending strain was observed in the pipe connector. Although strain values decreased as the diameter increased, the location of the peak strain remained constant along the curve. The bending strain along the pipe remained constant until the onset of wrinkling at the site of peak bending strain. This was caused by a plastic hinge formation in the pipe and the concentration of deformations on it. The plastic hinge position, or wrinkling point, was within 4–5 m of the fault on the footwall side. Because wrinkling occurred at a strain station, monitoring local strain during wrinkling became possible on the footwall side, but not on the hanging wall side. The strain recorded at the wrinkling point was not bending strain, thus the curve in Fig. 14c shows total strain in the wrinkling position. As stated, it is more accurate to show the strain as unknown on the curve at the wrinkling point in test 3 and at the peak point in test 2.

Figure 14d shows the results of test 4, which repeated test 3 at a shallower burial depth. Similar to test 1, at the beginning, the pipe was inclined to experience beam-type buckling. As fault offset increased and the pipe had a high slenderness ratio (D/π), beam buckling could not occur and deformation was concentrated at one point. Wrinkling then occurred where the pipe formed a plastic hinge. Under such conditions, increasing the fault offset had no effect on the strain away from the wrinkle, and thus strain remained nearly constant along the pipe. Wrinkling occurred in test 4 at 4 m from the fault along the footwall side. Because wrinkling occurred at a strain station, it could be monitored during faulting.

Figure 15 shows the variation of peak bending strain during faulting. As can be seen, in test 1, the bending strain was small before the onset of buckling. However, at the onset of buckling, the maximum bending strain at the middle of the pipe increased and exceeded 0.8% for an offset of 2.3 m. In test 2, maximum bending strain rose along the footwall side as an approximate linear curve and reached 1.2% for an offset of 2.3 m. On the hanging wall side, after the strain increased to 0.6% for an offset of 0.9 m, the strain remained constant. In both tests 3 and 4, when the pipe encountered local buckling, the peak bending strain curve shows a sudden decline (0.9 and 0.6 m offset, respectively).

Vertical displacement along the pipe

Vertical displacement of the pipelines during faulting is shown in Fig. 16. Displacement was measured by one displacement sensor at the middle of the pipe and one each at 8 m at the prototype scale on the two sides of the fault. In tests 1 and 4, where the pipe was embedded at a shallow depth, displacement was similar. When the overburden soil decreased, the pipe was quickly exposed at the ground surface. In test 1, with beam buckling deformation, vertical displacement reached a plateau when the fault offset exceeded 1.3 m. In test 4, with local buckling deformation, the vertical displacement curves are linear with slight deviations.
Fig. 11. Axial strain at various offsets plotted as function of distance from fault: (a) test 1; (b) test 2; (c) test 3; (d) test 4.

Fig. 12. Pattern of strain curve correction.

In tests 2 and 3, where the pipe formed either an unfolded S- or Z-shape, the curves show similar trends. Displacement was delayed along the hanging wall as pipe diameter increased. While deformation at the middle of the pipe began with the onset of faulting, the rate of deformation in the pipe with a smaller diameter was greater. In these tests, the pipes experienced negligible displacement along the footwall side with negligible deformation. In test 3, the pipe experienced local buckling and deformed as an unfolded Z-shape, and vertical displacement reached a plateau. In test 2, by contrast, the vertical displacement curve increased.

**Local buckling strain**

Initial wrinkling strain caused by compression in a cylinder has been theoretically stated as the following by Southwell (1914):

\[ \varepsilon_{\text{theory}} = 0.6 \frac{L}{R_0} \]

According to Hall and Newmark (1977), compressional wrinkling in a pipe normally begins at a strain of 1/3 to 1/4 the theoretical value. Kennedy et al. (1977) also state that the initial strain at the wrinkling point of larger pipes is equal to 0.4%–0.6%. Based on eq. [1], the theoretical value of wrinkling strain for pipes used in tests 1 and 2 was 6% and for pipes used in tests 3 and 4 was 2.4%. In other words, according to Hall and Newmark (1977), the pipe should begin to wrinkle at 1.5%–2% strain in tests 1 and 2 and at 0.6%–0.8% strain in tests 3 and 4. The test results showed that the total strains in tests 3 and 4 were larger than the Hall and Newmark (1977) predictions and the pipe was also subject to wrinkling. The maximum observed total strain in tests 1 and 2 did not reach 1.5% and no wrinkling occurred in these pipe models. Therefore, the results of the present study are in good agreement with Hall and Newmark (1977) for the onset strain of wrinkling. It must be noted that Hall and Newmark (1977) addressed the wrinkling of a pipe under compression while, in the present study, the pipes were subjected to a combination of bending and compression. It is expected that wrinkling will occur faster for this case than in the case of a pipe that is only subjected to compression.

**Conclusion**

Four centrifuge tests were conducted to study the behavior of continuous buried steel pipelines subjected to reverse faulting. Two pipes with different diameters were tested at two burial depths. Stainless steel pipes were used in all tests and the pipes were connected to a simulator box using a fixed connection. The deformation angle of the fault in all tests was 60°. Strain gauges and displacement transducers were used to record axial strain, bending strain, and vertical displacements caused by faulting.
The pipe deformation mechanism and damage type changed significantly with variations in pipe diameter and burial depth. The type of deformation at the shallower burial depth was beam buckling, which is preferred to wrinkling because the pipe is more likely to remain in operation. Deeply buried pipes were subject to unfolded S- or Z-shape deformations that seriously damaged the pipes.

The pipe section slenderness ratio also played a significant role in pipe deformation. Because pipes with a low $D/t$ ratio were not subject to wrinkling or local buckling, their performance was better, making them safer and experiencing less damage. Slender pipes (high $D/t$ ratio) at deep or shallow depths were subject to wrinkling, which led to circumferential cracking of the pipe wall and leakage. If an actual pipeline carried gas or another flammable material, there would be a significant risk of explosion and serious damage. Therefore, it is recommended to use pipes with smaller diameters and low $D/t$ ratios in fault zones.

The locations of the maximum strain and wrinkling point were the parts of the pipe most susceptible to damage. The use of flexible elements at these locations may help keep the pipe in operation.

The wrinkling strains from these tests are in good agreement with Hall and Newmark (1977).
Fig. 15. Peak bending strain versus fault offset.

Considerable axial strain was not observed in pipes with beam buckling deformation and these pipes were subject to complete bending.

Increasing the diameter and burial depth of a pipe changed the deformation mechanism from beam buckling to local buckling and wrinkling. These results are in good agreement with the Meyersohn (1991) curve for critical burial depth.

Increasing burial depth increased peak axial strain.

Decreasing the overburden, for example, by using lightweight sand or expanded polystyrene (EPS) backfill instead of soil, may change the pipe deformation mechanism from local buckling to beam buckling, which is a more favorable outcome with regard to pipe operation.

The present study did not address the effects of realistic semi-rigid end pipes and internal pipe pressure on pipeline behavior. It is suggested to repeat these model tests using a free-end pipeline and pipes modeled to include internal pressure.

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List of symbols

- \( C_c \) coefficient of curvature
- \( C_u \) coefficient of uniformity
- \( D \) pipe outer diameter
- \( D_{ab} \) average particle size of sand backfill
- \( e_{\text{max}} \) maximum void ratio
- \( e_{\text{min}} \) minimum void ratio
- \( F \) fine content of soil
- \( G_s \) specific gravity of soil particle
- \( H \) depth of soil from the surface to the top of pipe
- \( N \) scaling factor (see Table 2)
- \( R_0 \) pipe radius
- \( t \) pipe wall thickness
- \( a \) pipeline fault orientation angle
- \( \beta \) fault deformation (dip) angle
- \( \beta_F \) normal fault deformation (dip) angle
- \( \epsilon_{\text{theory}} \) theoretical initial wrinkling strain

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