Rate effects during pipeline upheaval buckling in sand

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Knowledge of the uplift capacity of buried pipelines is critical in selecting appropriate burial depths to ensure their stability against upheaval buckling. Much work has focused on their capacity under either fully drained or fully undrained conditions. However, partial drainage conditions may be provoked for low-permeability silty sand seabeds, and this has not been studied in detail. This paper reports a series of physical model tests investigating the effects of pipeline displacement rate on both the uplift capacity of pipelines and the displacements required to mobilise peak load. It is shown that these drainage rate effects can be significant, and the drainage conditions will depend on the dimensionless velocity \( \frac{vD}{c_v} \), as suggested by previous researchers studying penetration and foundation problems. A change in normalised pipeline velocity will trigger different degrees of drainage and provoke different deformation mechanisms in the soil, and these will affect the load–displacement response of the pipelines. Designers should consider carefully how rate affects the upheaval buckling resistance of pipelines, especially in silty sands.

NOTATION
- \( c_u \): undrained shear strength
- \( c_v \): coefficient of consolidation
- \( D \): pipeline diameter
- \( d_{10} \): effective particle size
- \( d_{50} \): mean particle size
- \( E_0 \): one-dimensional Young’s modulus
- \( f_u \): uplift factor
- \( G_s \): specific gravity
- \( g \): acceleration due to gravity
- \( H \): depth of crown of pipeline beneath soil surface
- \( K \): lateral earth pressure coefficient
- \( k \): permeability
- \( N_p \): bearing capacity factor
- \( v \): pipeline velocity
- \( \nu \): vertical displacement
- \( W' \): pipeline buoyant self-weight
- \( W_i \): uplift load
- \( W_s \): soil restraint
- \( W_{umax} \): peak soil restraint
- \( \gamma \): saturated unit weight
- \( \gamma' \): effective unit weight
- \( \delta \): displacement
- \( \rho_{max} \): maximum soil density

1. INTRODUCTION

Offshore pipelines are buried beneath the seabed to protect them from fishing activity and environmental loadings. During commissioning, the high-pressure and high-temperature oil will heat the pipeline, causing thermal expansion. Because of the axial restraint due to burial, the pipeline will experience axial loading, which may provoke lateral buckling. For a pipeline that is buried typically several diameters beneath the seabed (see Figure 1), the direction of least resistance to buckling is upwards, and so pipelines may undergo ‘upheaval buckling’ during initial or subsequent heating-up events. Prevention of upheaval buckling requires sufficient restraint from the soil above the pipeline, and this is a key issue of consideration when selecting pipeline burial depth. Pipeline installation costs increase with burial depth, and so it is important that design methods for predicting the uplift resistance provided by the soil are accurate so that the minimum appropriate burial depth can be selected.

The geometry of a buried pipeline is shown in Figure 1. A pipeline of diameter \( D \) is buried such that the crown of the pipeline is at depth \( H \) beneath the soil surface. Uplift load \( W_i \) is applied to the pipeline, which is resisted by a combination of the self-weight of the pipeline, \( W' \), and the soil restraint, \( W_s \).
The buoyant self-weight of the pipeline \( W \) is easily calculated, but the soil restraint is more complex, and is the focus of this study.

There has been much research conducted investigating soil’s resistance to upheaval buckling.4–9 This research has generally used combinations of physical model testing and simple analytical methods to produce design solutions, although some relevant numerical modelling has been conducted.10,11

Despite the large body of research work, it has been assumed that soil is either fully drained (sand or silty sand) or fully undrained (clay). This assumption is questionable for silty sand, and is the focus of this study.

As well as the above generic studies of the problem, site-specific centrifuge uplift tests have been conducted for many recent pipelines. These tests are carried out in reconstituted seabed samples taken from the intended pipeline route as part of the design process. The pipeline is extracted from the soil (which has to be prepared carefully to recreate soil conditions after pipeline installation) while the load–displacement response of the pipeline is measured. The scaled values of peak uplift force and displacement to peak are utilised in design. However, there are no generally available recommendations for appropriate pipe uplift rates in sandy soils, as it is generally assumed that fully drained conditions are provoked. This last assumption is investigated in this paper.

The aim of this paper is therefore to investigate rate effects occurring during pipe uplift. The paper focuses on how the uplift capacity and soil restraint stiffness are affected by partial drainage, and considers only loose sand (as a likely backfill material). A series of tests are reported that were conducted in the laboratory and the geotechnical centrifuge.

2. EXPERIMENTAL METHOD

Tests were carried out in two separate backfill materials: uniform sand, and a silty sand obtained from the seabed of the North Sea. The tests on loose, saturated uniform sand were conducted in the laboratory, and high rates were selected (from 10 mm/min to 500 mm/min) to provoke a partially drained response. These rates would be considered fast for a pipeline uplift event, but will give similar drainage conditions to real loading events for lower-permeability silty sand. To confirm this finding, a final series of centrifuge model tests were performed with the silty sand sample at uplift rates that might be expected offshore.

2.1. Geometry

Physical model testing was carried out in the laboratory, and in the Dundee University geotechnical centrifuge. Both series of tests consisted of the extraction of a ‘long’ (i.e. approximately plane strain) horizontal circular pipeline section from a box of soil with measurement of uplift resistance and pipe displacement. For both sets of tests the front face of the soil box was constructed from Perspex, and the pipe was positioned perpendicular to the Perspex front face, allowing the soil and pipe to be observed as pull-out progressed.

Pipes 495 mm long with 48 mm outside diameter were tested in the laboratory, and pipes 240 mm long with diameter 25.4 mm were tested at 12.9g in the geotechnical centrifuge. Scaling laws12 ensured that the 25.4 mm pipe at 12.9g would behave as a prototype 12-9 times larger (i.e. \( D = 12.9 \times 25.4 \text{ mm} = 328 \text{ mm} \)). In the laboratory experiments the pipe diameter was 218 times larger than the mean particle size \( (d_{50}) \); this ratio was 212 in the centrifuge (where finer soil was used), and so grain size effects are unlikely.

2.2. Testing methodology

The preparation procedures were the same for both types of test. Soil was placed to a depth of approximately one pipeline diameter into the base of the box, and the pipe was positioned on the surface of the soil. The pipe was located with its ends about 1 mm from the front and back Perspex faces of the box, and grease was pushed into the gap to prevent soil entering the gap between the pipe and the front face. More soil was then added as before, until the final soil height gave the required pipe embedment depth. While the soil was placed above the pipe, it was important that the pipe was free to settle vertically, so that a net vertical load was not applied to the pipe before the pull-out test commenced.

Once the soil had equilibrated either in the laboratory or in the enhanced acceleration field of the geotechnical centrifuge, the pipe was then pulled out of the soil using a rigid hanger arrangement. This hanger ensured that the pipe moved with uniform displacement and vertically (Figures 2a, 2b). In the centrifuge tests, load was measured using a 500 N capacity S-shaped load cell, and displacement was measured with a linearly variable differential transformer (LVDT) while the pipe was displaced with a specially designed linear actuator. In the laboratory, actuation was achieved with an Instron load frame (Figure 2d), which contained a load cell and potentiometer. For each type of test the displacement rate was fixed throughout the uplift event. For the slowest tests a series of digital images were captured of the front face of the box as the pipe moved towards the soil surface, thus allowing later measurement of soil movements and examination of soil deformation mechanisms.

2.3. Soil preparation

One set of tests was conducted on silica sand. The sand was uniformly graded [see Figure 3] with \( d_{10} = 0.18 \text{ mm}, d_{50} = 0.22 \text{ mm} \) and \( G_s = 2.65 \), and the particles were subrounded. The maximum density, \( \rho_{\text{max}} \), was 1958 kg/m³, and the minimum density, \( \rho_{\text{min}} \), was 1517 kg/m³. Samples were prepared with a saturated unit weight \( \gamma \) of 19.42 kN/m³, which corresponds to an effective unit weight \( \gamma' \) of 9.61 kN/m³, voids ratio \( e \) of 0.69, and a relative density of 13%. The critical state angle of friction, \( \phi'_{\text{crit}} \), was \(-32^\circ\).

The second test series was conducted on a silty sand, taken as a vibrocore sample from the top 1 m of the seabed in the North Sea. The particle size distributions of a subsample at the top and base of the soil sample are given in Figure 3. It is expected that the particle size distribution of the mix of the sample can be approximated by the average of these two curves (Figure 3). This distribution has \( d_{10} = 0.06 \text{ mm} \) and \( d_{50} = 0.12 \text{ mm} \). Samples were prepared with an effective unit weight \( \gamma' \) of 10.1 kN/m³.

The aim of the soil sample preparation was to generate repeatable samples with relative densities appropriate for
offshore pipeline cover materials. The two main types of pipeline burial offshore are jet trenching (in which a trencher moves over the seabed surface and ejects water at high pressure from a series of jets to liquefy the soil, into which the pipeline sinks) and ploughing (where a plough is dragged along the seabed producing an open trench, which is then backfilled over the pipeline). It is likely that backfill density will differ between the two installation methods because of the different soil mechanics processes occurring: a recently liquefied jetted trench may be looser than a ploughed trench that has been backfilled by pushing spoil heaps back into a trench. Newson et al.\textsuperscript{13} carried out T-bar testing to investigate jetted and ploughed backfills in very soft clay offshore and found little difference between the two soil types, but this is unlikely to be true in granular soils (or even with overconsolidated clays). The authors are unaware of published information for backfills in granular material. The sample preparation methods used attempted to simulate ploughed pipelines. It is likely that the relative density of the soil samples will critically affect the rate effects observed because of the different amounts of volume change expected during shear.\textsuperscript{14} This will be discussed later.

Soil preparation varied, depending on whether the samples were dry or saturated, and on the material used. Dry, loose

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**Figure 2. Pipeline testing apparatus:**
(a) schematic of centrifuge testing apparatus; (b) laboratory testing apparatus; (c) photograph of centrifuge testing apparatus (for clarity, no soil added); (d) photograph of laboratory testing apparatus

**Figure 3. Particle size distributions for the two soils tested**

sand samples were prepared by pluviation dry sand from a hopper into the strongbox at a controlled rate, and from a fixed height. This allowed soil densities to be repeatable. The soil density was calculated knowing the gross volume and mass of the sand, and also measuring the contents of small sample pots within the strongbox after the experiments. Saturated sand samples were created in a similar way, but the water table was maintained 50 mm above the rising sand surface as the dry sand was pluviated, thus partially mimicking the backfill ploughing process.

The silty sand samples were prepared by sedimentation of vibrocore samples through a water layer. This ensured saturation, and produced a sedimented sample that partly mimicked the undisturbed seabed. The soil was then disturbed to recreate pipeline installation, and the pipeline was placed at the target depth to achieve a cover depth three times the pipeline diameter (i.e. \( H/D = 3 \)). The sample was then placed on the centrifuge and spun at 12.9 \( g \) for 17 min (48 h at prototype time) to equilibrate before the uplift test was performed. Post-test sampling of the soil allowed calculation of the soil density.

2.4. Programme of tests

Model tests were first conducted in loose, uniform dry sand in the laboratory at slow uplift rates to characterise the fully drained uplift capacity, load–displacement response, and to examine the mechanics. Following this, laboratory tests were carried out in the saturated loose, uniform fine sand at high rates up to 500 mm/min (see Table 1 for velocities). The final set of tests comprised centrifuge model tests conducted for pipelines in silty sand samples, at rates that might be experienced by pipelines offshore. The test conditions are summarised in Table 1.

3. EXPERIMENTAL RESULTS

3.1. Loose sand

3.1.1. Uplift capacity. Normalised load–displacement results are shown for the sand test series in Figure 4. The uplift load \( W_u \) (per unit length of pipeline) is normalised by the effective unit weight of soil, \( \gamma' \), the embedment depth \( H \) and the pipeline diameter \( D \). All tests used a pipeline of diameter \( D = 0.048 \) m and the same embedment ratio, \( H/D = 3 \), and were in loose sand.

![Figure 4. Load–displacement data for pipeline tests carried out at different velocities (saturated sand; \( H/D = 3; D = 0.048 \) m)](image)

The test in dry sand must provoke a fully drained response owing to the low viscosity of air (compared with water) and the slow uplift rate. However, the remainder of the tests may have provoked different amounts of partial drainage.

The test carried out at a displacement rate of 10 mm/min gives very similar normalised results to the test in dry sand. This suggests that the displacement rate provokes almost fully drained conditions. However, for the tests with larger velocities, uplift capacity is increased significantly.

The change in uplift load is expected to be due to a change in the drainage regime. Consequently, in Figure 5 the results are replotted as the normalised capacity \( W_u \gamma' HD \) against the normalised velocity \( \nu D/c_v \), as first recommended by Finnie, where \( \nu \) is the pipeline velocity and \( c_v \) is the coefficient of consolidation. Finnie studied the response of shallow foundations of breadth \( D \) under vertical displacements, but the approach has been used while examining drainage rate effects for the cone penetrometer test and the T-bar penetrometer, where \( D \) represents the cone or T-bar diameter.

The coefficient of consolidation \( c_v \) was not measured directly for the sand but estimated using \( c_v = kE_3/\gamma_w \). The permeability \( k \) was estimated using Hazen’s law \( (k = 0.01d_{10}^2) \) with \( d_{10} = 0.18 \) mm. The one-dimensional Young’s modulus, \( E_3 \approx 2775 \) kPa, was measured using an oedometer test. This approach gave \( c_v = 0.09 \) m²/s. This value is in line with the 0.25 m²/s suggested by Brennan for a slightly coarser uniform sand (Leighton Buzzard fraction E), which he calculated both by the method above and by

<table>
<thead>
<tr>
<th>Test identifier</th>
<th>Soil type</th>
<th>(Effective) unit weight: kN/m²</th>
<th>Pipe diameter, D: mm</th>
<th>Initial embedment ratio, H/D</th>
<th>Uplift velocity, ( v ): mm/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry 1</td>
<td>Dry sand</td>
<td>15.30</td>
<td>48</td>
<td>3.0</td>
<td>1</td>
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<tr>
<td>Dry 2</td>
<td>Dry sand</td>
<td>15.20</td>
<td>48</td>
<td>3.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Rate 1</td>
<td>Saturated sand</td>
<td>9.46</td>
<td>48</td>
<td>3.0</td>
<td>10</td>
</tr>
<tr>
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<td>48</td>
<td>3.0</td>
<td>50</td>
</tr>
<tr>
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<td>48</td>
<td>3.0</td>
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<tr>
<td>Rate 4</td>
<td>Saturated sand</td>
<td>9.72</td>
<td>48</td>
<td>3.0</td>
<td>500</td>
</tr>
<tr>
<td>Offshore 1</td>
<td>Silty sand</td>
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<td>328*</td>
<td>2.9</td>
<td>0.71†</td>
</tr>
<tr>
<td>Offshore 2</td>
<td>Silty sand</td>
<td>10.10</td>
<td>328*</td>
<td>3.1</td>
<td>0.11†</td>
</tr>
</tbody>
</table>

* At prototype scale. Equivalent model pipe diameter \( d = 25.4 \) mm.
† Displacement rates at prototype scale.

Table 1. The series of physical model tests
examination of pore pressure dissipation curves in a centrifuge model test, with good agreement. To calculate the $c_v$ value for the dry sand, it was assumed that the ratio of permeability between the saturated sand and the dry sand was equal to the ratio of viscosity between water and air (at 20°C), and that the soil stiffness was unaffected.

The dataset presented in Figure 5 shows a similar form of the curves presented previously for shallow foundations and penetrometers. Figure 5 suggests that a normalised velocity $vD/c_v$ for shallow foundations and fully drained conditions. The values of threshold velocities for each drainage regime differ significantly from previous findings, with both transitions occurring at normalised velocities three orders of magnitude larger (e.g. the transition to almost fully undrained behaviour occurs at approximately $vD/c_v = 10$ according to Randolph and House, but occurs here at approximately $vD/c_v = 0.01$). There are two possible explanations for the disparity, which are discussed in turn below.

First, the drainage path lengths of a pipe uplift problem may be significantly different from that of the surface foundation originally investigated by Finnie, or even the cone or T-bar penetrometers, so the use of $D$ in the dimensionless group may be inappropriate. For example, for cone penetration tests under fully drained conditions, a cavity expansion with a large zone of total stress increase is expected for the penetrometers, with presumably a drainage path length due to drainage either to the far field, or to the soil surface, or around the penetrometer. Therefore the actual drainage path length $D$ may be of the order of the size of the penetrometer, or longer. However, in the pipe uplift problem the failure mechanism may involve shear localisation above the pipeline, which may provoke very local drainage associated with the thickness of the shear band ($\approx 12d_h$). If so, the appropriate length scale $D$ for normalisation in the dimensionless velocity should be smaller than the pipeline diameter, which will increase the calculated normalised velocities in Figure 5.

The second possibility for the mismatch in $vD/c_v$ values is that the operative $c_v$ occurring during deformation was not estimated correctly. This was discussed by Silva et al., who examined drainage regimes numerically for cone penetrometers in clay with different stress histories. The estimate of $c_v$ for these tests is appropriate for normally consolidated sand under vertical drainage conditions. In addition, the soil will be anisotropic (it has been pluviated), and will be subject to different stress paths. This is likely to increase $c_v$ (higher stiffness and larger permeability), and also increase the normalised velocity in the tests.

**Comparison of capacity with previous solutions.**

Recommendations have been given previously about the calculation of pipeline uplift capacity under both fully drained and fully undrained conditions. Calculated capacities from these methods are now compared with the results from the tests.

Fully drained pipeline (and strip anchor) capacity has been examined by many researchers. Generally, for shallow pipelines, assumed mechanisms are in the form of an uplifting block, which may have vertical slip planes emanating from the edges of the pipeline (e.g. Majer; Figure 6a) or at an angle from the vertical depending on either the angle of friction or the angle of dilation of the soil (e.g. White et al.; Figure 6b). Calculation of pipeline capacity requires knowledge of the soil’s unit weight $\gamma$, the angle of dilation $\psi$, and the lateral earth pressure coefficient $K$ acting on the shear plane.

An alternative method is simply to characterise the backfill performance with an empirical uplift factor, as suggested by

![Figure 6. Postulated failure mechanisms for calculation of drained uplift capacity: (a) vertical slip mechanism; (b) mechanism proposed by White et al.](image-url)
Schaminee et al. suggested that $W_u/\gamma' HD = 1 + f_u(H/D)$, where they measured $f_u = 0.4$ for loose sand. A value of $f_u = 0.5$ is used typically by industry, and for example Bransby et al. found $f_u \approx 0.5$ during centrifuge and laboratory tests on loose sand.

The drained capacity using a Schaminee uplift factor $f_u = 0.5$ is shown in Figure 5, together with the calculated capacity using the vertical slip method assuming that $K = 1 - \sin \phi$ and that $\phi = 34^\circ$. The calculated capacities are almost identical. The White et al. method gives a normalised uplift factor $W_u/\gamma' HD = 2$ for $\phi = 32^\circ$ and dilation angle $\psi = 0$. All the methods slightly underpredict the drained capacity, but could be modified to fit the data by increasing the uplift factor to $f_u = 0.55$ in the Schaminee method, or by increasing the angle of friction and/or dilation angle in the other models.

In the fully undrained loading condition it is impossible for soil to detach from the bottom half of the pipeline, and so the undrained vertical slip model cannot be used. Instead, a simple method has been performed, based on the undrained solution by Randolph and Houlsby assuming that the capacity factor $N_p$ varies from 9-14 to 11-94, depending on the disc roughness. The mechanism consists of a local shear mechanism involving displacement of soil from in front of the disc to behind it, and will not extend to the ground surface. This approach was used by Bransby et al. in their analysis of pipeline uplift in consolidating clay backfill.

If it is assumed that the mean effective stress in the soil is unaffected by the pipeline installation, then the value of vertical effective stress at the depth of the pipe centre can be given as

$$\sigma' = \gamma'(H + D/2)$$

If it is further assumed that the effective stress in the soil does not change during shearing, then an approximate operational undrained shear strength $c_u = \sigma' \tan \phi' = \gamma'(H + D/2) \tan \phi'$. Taking an intermediate $N_p = 10-5$ (as customary in pile design), this gives

$$W_{\text{umax}}/D = 10.5\gamma'(H + D/2) \tan \phi'$$

which normalises to

$$W_{\text{umax}}/\gamma' HD = 10.5 \left(1 + \frac{D}{2H}\right) \tan \phi'$$

Equation 3 can be re-expressed in terms of an uplift factor

$$f_u = \frac{10.5(1 + D/2H) \tan \phi' - 1}{H/D}$$

For $\phi' = 34^\circ$ and $H/D = 3$ the above equation gives $W_{\text{umax}}/\gamma' HD = 8.26$ (or $f_u = 2.42$), which is plotted in Figure 5 and shows a good agreement with the results of the fastest test.

Clearly, the assumption of a constant effective stress is simplistic, and relies on no tendency for volumetric strain during shear. Very loose sands will generate excess pore pressure during shearing, as they tend to reduce volume, and so the above analysis is likely to overestimate uplift capacity. Dense sands will generate pore water suction to suppress dilation, and this is expected to lead to increases in effective stress around the pipeline (as evidenced in pipeline ploughs), so the above analysis will underestimate pipeline capacity for dense sands. Therefore the effect of soil density on drainage rate effects requires more study.

In summary, calculation of the fully drained uplift capacity is relatively straightforward. The condition for fully undrained shear is more complex, but a simple method approximated the capacity well. The uplift capacity at rates between the two extremes involves partial drainage, and appears best characterised by a drainage transition curve of the form first recommended by Finnie.

3.1.2. Displacement at peak load. The initial load–displacement responses of the tests carried out at different displacement rates in loose sand are shown in Figure 7. Tests at greater velocities require significantly larger displacements to mobilise peak uplift capacity.

An uplift displacement of approximately 4 mm is required to mobilise peak load for the slow, fully drained tests. This displacement corresponds to 2-8% of the embedment depth (i.e. $\delta/H = 2-8\%$). This compares with mobilisation distances of $\delta/H = 0-5\%$ as measured by Matyas and Davis in their laboratory tests, and with tests by Trautmann et al., who reported $\delta/H = 1\%$.

The results from the test at a velocity of 500 mm/min appear to contain some initial bedding problems or transient drainage effects for small displacements, but even if it is assumed that the initial data followed the dashed line in Figure 7, a displacement $\delta \approx 13$ mm (i.e. $\delta/D = 27\%$) is required to
mobilise peak capacity. This is a much larger displacement than that required for the slower test.

The relationship between the normalised velocity and the displacement at peak load is shown in Figure 8a, and reveals an approximately linear increase in mobilisation distance with displacement rate. This is in contrast to the relationship for uplift capacity (Figure 5). Figure 8b shows the same results with velocity plotted on a logarithmic scale. In both graphs the data point with an open symbol represents the displacement for the 500 mm/s test if the first 5 mm of displacement represents a bedding or transient error. Unlike the capacity–velocity relationship (Figure 5) there does not appear to be a plateau reached as undrained conditions are approached. A similar finding was observed by Vesic et al.\textsuperscript{26} when studying the bearing capacity of footings on sand.

3.1.3. Deformation mechanisms. Soil displacement mechanisms can be found from direct examination of photographs taken during or after testing,\textsuperscript{9} or by performing analysis of sequential digital images taken at different pipeline displacements. The particle image velocimetry (PIV) program GeoPIV written by White et al.\textsuperscript{27} was used to quantify soil displacement fields. As only qualitative displacement fields were of interest, no photogrammetric correction was used to allow for image distortion.\textsuperscript{28} Because a high-speed camera was not available, images were captured only for the slow (drained) tests.

Measured displacements for small displacements at fully drained displacement rates. A series of digital photographs were taken of the front face of the strongbox for a laboratory test with loose dry sand (test: dry 2), while the load–displacement response was also measured using the Instron testing frame (Figure 9). The load–displacement curve is almost identical to that presented in Figure 4 as the embedment ratio and soil density were very similar. Soil displacement fields measured for the pipeline displacements marked A, B and C in Figure 9b are shown in Figure 10. Note that scale factors for plotting the displacement vectors in Figure 10 have been chosen to make the pipeline displacement magnitudes appear identical, to allow comparison of the displacement mechanisms.

Figure 10 shows that first, even at twice the displacement required to mobilise the peak uplift force (Figure 10c), the mechanism is only approximately similar to the vertical slip model. Instead, vertical displacements ($v$) above the pipeline reduce as the soil surface is approached (Figure 11), indicating significant vertical compression of the soil above the pipeline. This is similar to the mechanism suggested by Palmer et al.,\textsuperscript{21} but very different from that observed by Cheuk et al.\textsuperscript{28} for dense sand. Associated with this vertical movement are the two shearing zone/shear localisations radiating vertically upwards from the pipe edges, but with reduced shear strains as the soil surface is approached. There is no evidence of significant soil heave below the pipeline. From comparison of Figures 10a, 10b and 10c and the separate datasets in Figure 11 it can be seen that the mechanism is progressive, with the zone of soil displacement (and the vertical shear zones) progressing upwards away from the pipeline as the pipeline displacement increases.
With increasing vertical pipe displacement (not shown), soil stresses continue to increase on the top of the pipe, and decrease behind, until the normal stress at the back of the pipe reduces to zero. The reduction of stress to zero causes detachment between the pipe and the soil mass (gapping). Further pipe displacement causes the gap size to increase until the side slopes beneath the gap reach the angle of repose, after which a circulation mechanism prevails, ensuring the gap remains the same size. Visual observations suggest that this final mechanism requires a displacement $\frac{\delta}{D} = 0.2–0.3$ or $\frac{\delta}{H} = 0.06–0.1$. Although this final mechanism does not relate to the mechanism at uplift capacity, its presence confirms that a gapping mechanism is forming during the uplift event (i.e. detachment of soil can occur beneath the pipeline during uplift).

Gapping was observed behind the pipeline in all dry sand tests, and in the saturated sand tests at velocities of 50 mm/min or slower.

**Mechanisms observed during high-rate tests.** For the saturated sand tests with displacement rates of 200 mm/min and 500 mm/min there was no observed gap formation behind the pipeline. This suggests that there is a significant mechanism change, with no detachment possible behind the pipeline, and reinforces the use of the Randolph and Houlsby mechanism. Further work is required to identify mechanisms at higher displacement rates, and this will require high-speed photography.

**4. RESULTS FROM SEABED SOIL: SILTY SAND**

The test conditions for the seabed sample tests are shown in Table 1. Two test velocities were selected: (a) 42.6 mm/h (or 0.71 mm/min) in test ‘offshore 1’, and (b) 6.5 mm/h (or 0.11 mm/min) in test ‘offshore 2’. Clearly, specifying any constant rate for an uplift test is an oversimplification of the real loading event. Displacement will not be at a constant rate in reality, as it is a partially load-controlled event: the rate will increase with pipeline displacement until catastrophic ‘snap-through’ buckling occurs; there may be ‘consolidation’ uplift movements, where pore pressures dissipate and cause increasing displacements under long-term uplift loads; and the amount of displacement (and hence displacement rate) will also vary along the length of the buckling section (i.e. it is a three-dimensional problem).

Despite the above complications, a fixed displacement rate is likely to be selected during a typical set of site-specific pipe uplift tests carried out in a geotechnical centrifuge. The
displacement rate of 42.6 mm/h (test ‘offshore 1’) would be the mean displacement if peak uplift resistance (requiring 40–80 mm of upward displacement) was mobilised in 1–2 h during heat-up of the pipeline. The displacement rate of 6.5 mm/h (0.11 mm/min; test ‘offshore 2’) would require 5–10 h of heat-up time for the same displacement (or could occur if peak loads are not mobilised). The pipeline designer should ensure that the pipeline uplift rate selected for testing does not overestimate the long-term capacity of the soil.

Figure 12 shows the results of the two centrifuge tests. Despite the fact that the faster test has a lower embedment ratio than the slower test (H/D = 2.9 compared with 3.1), it has a greater capacity. The difference in capacity can be expressed in terms of Schaminee et al. uplift factors, $b_i = 0.99$ and 0.63 for the faster and slower tests respectively. In addition, about twice the displacement is required to mobilise peak load for the faster test as for the slower one (Figure 13). The change of capacity and mobilisation displacements with rate follow the same pattern as observed for the sand tests. A $c_v$ value was calculated for the silty sand (using $d_{10} = 0.06$ mm), assuming that only the permeability was different from the sand sample. This allowed calculation of normalised velocities $vD/c_v = 3.9 \times 10^{-4}$ and $5.96 \times 10^{-3}$ for the two tests, putting the faster test in the partially drained zone, as plotted in Figure 5, and both data points in good agreement with the sand dataset. This confirms that rate-dependent behaviour may be expected for pipelines in silty sand during offshore operations.

The results suggest that careful consideration of appropriate displacement rates is required when selecting testing conditions for site-specific centrifuge uplift testing for pipelines in silty sand. For example, a centrifuge uplift test carried out at too high a velocity may give unconservative results: that is, long-term load application or lower displacement rates may lead to gradual uplift of the pipeline from the soil, and hence to upheaval buckling. Indeed, the tests conducted here suggest that it is conservative to perform tests and calculate uplift capacities for drained conditions. However, it should be realised that the effect of rate may be very sensitive to the relative density of the backfill soil. It is even possible that extremely loose backfill (e.g. $D_r < 10\%$) may have a tendency to compress during shearing and thus reduce capacity, owing to the generation of positive excess pore pressures with increasing velocity. Further work is required to investigate backfill density for different methods of pipeline burial, and investigate how rate effects are affected by these relative density changes.

5. CONCLUSIONS

A series of physical model tests investigated the effect of displacement rate on the uplift capacity and stiffness of pipelines buried in loose, saturated sand. The conclusions are as follows.

(a) For loose sand in fully drained conditions, there was good agreement with predictions using previously published design methods (e.g. the vertical slip model using $K_{soc}$ and the experimental data.

(b) Faster pipeline uplift displacement rates provoke partially drained soil response. In the tests reported here, this resulted in both the soil uplift resistance and the distance required to mobilise peak load increasing with velocity.

(c) The relationship between normalised pipeline velocity ($vD/c_v$) and uplift capacity ($W_c/y'/HD$) was similar to that found for penetrometers and shallow foundations. However, the normalised velocities for different drainage regimes appeared to differ from the values previously recommended.

(d) The exact form of the uplift capacity–velocity relationship is expected to depend on the relative density of the backfill. Further work is required to ascertain typical backfill densities for pipelines installed by different methods offshore, and how the uplift capacity–velocity relationship varies with relative density.

(e) Rapid uplift velocities were required to provoke partially drained response in fine, loose sand, but these conditions may be experienced by offshore pipelines in soils with significant silt fractions. Tests performed in an offshore seabed material demonstrated rate effects at realistic pull-out rates. Designers should carefully consider critical pull-out rates for pipeline design, and suitable rates for centrifuge tests intended to provide these design parameters.

ACKNOWLEDGEMENTS

The authors would like to thank Dr GiJae Yun (now Technip UK Ltd, Aberdeen), who conducted the two additional (previously unreported) tests while carrying out contract testing. These contract tests were done under the partial supervision of Dr Tim Newson (now University of Western Ontario). In addition, the authors would like to thank the technical staff of the Division of Civil Engineering, University of Dundee.
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