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Soil restraint on buckling oil and gas pipelines buried in lumpy clay fill

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Abstract

Offshore pipelines used for oil and gas transportation are often buried to avoid damage from fishing activities and to provide thermal insulation. The soil cover also provides resistance to upward movement of the pipe caused by thermally-induced axial loading, a phenomenon known as upheaval buckling. Previous research has been conducted to investigate the available uplift resistance of a buried object provided by soil. However, most of these studies concerned the uplift resistance in homogenous soils. Pipeline installation by jetting or mechanical trenching and backfilling would result in a highly disturbed soil cover leading to a reduction in soil restraint, as well as stiffness of the response. The uplift resistance of heterogeneous soil cover has received limited research attention.

A series of centrifuge tests was conducted to assess the vertical pressure exerted on a pipeline buried in lumpy clay fill when the pipe was moving upward at a constant speed. A model pipe was buried in clay lumps, which were made from natural clay collected from the Gulf of Mexico. The lumpy soil cover was allowed to consolidate for a fixed time period, before vertical extraction was triggered. The resulting uplift resistance was measured for different uplift velocities. Two different consolidation time periods were considered to investigate the potential benefit of having a longer waiting period prior to putting the pipeline into operation. Results showed that early commissioning of buried pipelines in under-consolidated lumpy fill could lead to a reduction of soil restraint up to 56%, together with a decrease in the stiffness of the response. The suction force generated underneath the pipe, which increased with the uplift velocity, was found to be a significant contributor of the overall uplift resistance. Nevertheless, quantitative analysis suggested that the beneficial effect from a higher degree of consolidation was much more significant than that achieved from a high suction force originating from a fast uplift.

Keywords: Pipelines; Uplift; Lumpy fill; Speed effect; Suction

1. Introduction

1.1. Upheaval buckling

Flexible offshore pipelines used for oil and gas transportation are usually buried to avoid damage from fishing activities and to provide thermal insulation. In order to increase productivity and avoid the solidification of wax fractions, it has become necessary to transport hydrocarbons at high temperature and pressure. These extreme operating conditions tend to cause thermal expansion in the pipeline, which is very often restricted by side friction along the soil–pipeline interface. These combined effects result in an axial compressive force in the pipeline. The slender structural element therefore has a high vulnerability to buckling.

A pipeline buried in a trench is sufficiently confined in the lateral direction by the passive resistance of the trench walls. Restraint in the vertical direction is provided by the backfilled soil, whose minimum required depth is a key design parameter for pipeline engineers. Under-designed cover depth may promote upward movement in the pipeline. In extreme cases, the pipeline may protrude through the soil cover, a phenomenon known as "upheaval buckling".

1.2. Design challenges

The cost of burying a pipeline with a typical length of over tens or hundreds of kilometres can be significant. It is

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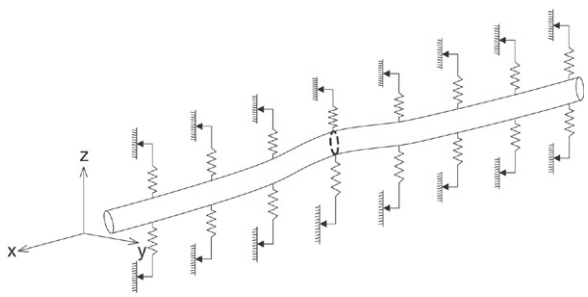


Fig. 1. Conceptual model for soil–pipeline interaction with springs and sliders.

therefore important to be able to optimise the required soil cover depth. The formation and development of buckles along a pipeline with spatial variations in soil restraints represents a complex soil–structure interaction problem. Palmer et al. [1] discussed a simplified design approach in which the pipeline is idealised as an elastic beam carrying an axial force with a given flexural rigidity. Following elementary beam theory, the downward force required to maintain vertical equilibrium in the pipe can be deduced. This value can be checked against the available vertical restraint of the soil cover.

With advances in computer modelling, more sophisticated analysis can be carried out. Fig. 1 depicts a conceptual model in which the problem is idealised as a beam supported by elastic springs and sliders. The combination of an elastic spring and a slider mimics the soil response as elastic–perfectly plastic behaviour, which can be characterised by the elastic stiffness of the spring (k) and the ultimate resistance of the slider (P_v). A comprehensive assessment of the pipeline response can be carried out if the two parameters are known. Nevertheless, these parameters can be very difficult to define due to the following reasons.

Firstly, the soil cover overlying a buried pipeline might have been subjected to severe disturbance during pipeline installation. Jetting is a common method for the pipe laying operation. The elevated hydraulic pressure creates a trench on the ocean floor and the pipe is allowed to sink into the seabed. In this method, the initially homogenous seabed surface would be broken down into very soft soil lumps separated by macro voids. This lumpy fill from which the uplift resistance is derived can be significantly softer and weaker than the intact material. Although the soil lumps will consolidate back into a more homogenous material due to their self-weight, the entire process may take a very long time, longer than the operator of the pipeline can afford. The available uplift resistance is therefore a time-dependent parameter. An alternative method of pipeline installation involves mechanical trenching and backfilling. Cathie et al. [2] suggested that the properties of the backfill material largely depend on the in situ strength of the seabed. Although mechanical backfilling would destructure the soil to a lesser extent as compared to jet trenching, the backfills, especially in soft seabeds, are believed to be highly heterogeneous and the properties are similar to hydraulic fills.

Secondly, the speed at which uplift resistance is mobilised is a parameter with a high degree of uncertainty. For low permeability soils, the uplift rate directly affects the drainage conditions and hence the resistance of the overlying soil. The problem is further complicated by the coexistence of low permeability soil lumps and open flow paths along the macro voids. Without due assessment of the above issues, an appraisal of the soil–pipeline interaction may simply not be representative of the real situation.

1.3. Objectives

This paper describes an experimental study that aimed at investigating the uplift resistance of a pipeline buried beneath a lumpy soil cover consolidated to different degrees and extracted at different uplift velocities. The problem addressed is similar to that described by Bransby et al. [3], in which uplift resistance of pipelines buried in liquefied clay was assessed. In the present study, a series of centrifuge tests was conducted using a small drum centrifuge to correctly mimic the stress level in a scaled physical model. A 1:30 scale model of a 0.4 m (∼16 in.) diameter prototype pipeline was tested using a specially designed strong box and a servo-controlled radial actuator. Offshore clay samples collected from the Gulf of Mexico were shaped into clay balls to form the lumpy soil cover above the buried pipe. Pore pressure transducers were placed around the model pipe to investigate the associated pore pressure changes during consolidation of the lumpy fill and the uplift process. Having consolidated to a prescribed time period, the model pipe buried in the lumpy fill was extracted at a constant speed with the uplift force and displacement measured. Two different consolidation periods were selected to represent fully consolidated and under-consolidated states. The influence of uplift speed on the soil restraint of a lumpy fill at the two soil states was assessed.

2. Basic physics of the uplift problem

The most critical location along a buckling pipeline is at the crest of an overbend. A cross section can be considered if only the available vertical soil restraint is of interest as illustrated in Fig. 2. The available uplift resistance of the soil cover as the buried pipe begins to move upwards depends on the drainage conditions in the deforming soil as well as the adherent conditions on the underside of the pipe (i.e. the soil–pipe interface). Three different scenarios are possible.

Scenario 1 — fully undrained and fully bonded

When soil underneath a buried pipe is moving upward with the pipe due to adhesion, the entire failure mechanism resembles that of a reverse bearing capacity problem. The adhesion is normally provided by the negative excess pore water pressure generated on the underside of the pipe. Fig. 3 shows a simple mechanism which is similar to a basal stability problem as described by Bjerrum and Eide [4]. The constant volume condition ensures that the soil heave above the pipe is compensated by the downward soil movement in soil blocks L and N. This implies that there is no change in potential energy in the entire mechanism. Therefore, the solution of the problem is independent of the soil weight. From the displacement diagram, the uplift force per unit length P_v, which excludes the weight of the pipe, is given by:

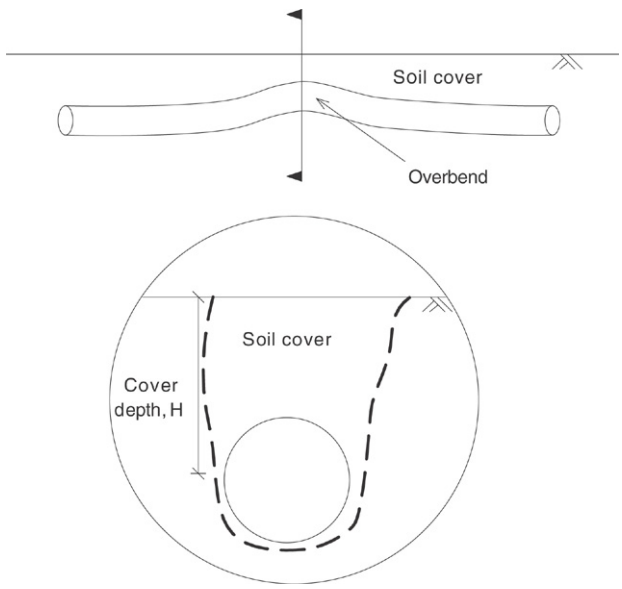


Fig. 2. Geometry of the uplift problem.

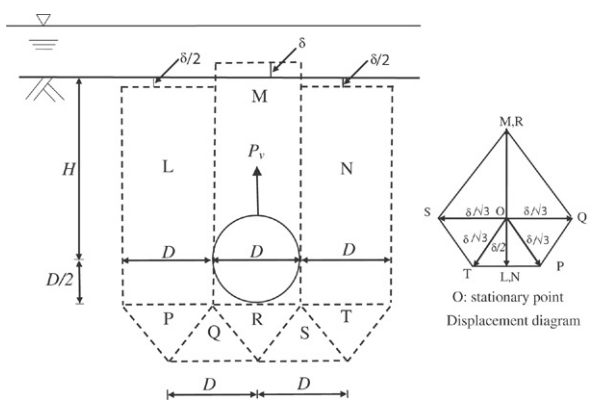


Fig. 3. Uplift mechanisms of a fully bonded buried pipe in clay under undrained conditions.

$$P_v = \left[4\left(H + \frac{D}{2}\right) + \frac{11D}{\sqrt{3}}\right] s_{u,\text{ave}} = \left(\frac{4H}{D} + 8.35\right) s_{u,\text{ave}} D \quad (1)$$

where H is the embedment depth of the pipe measured from the pipe centre; D is the diameter of the pipe, and $s_{u,\text{ave}}$ is an average undrained shear strength for the entire mechanism.

This simple solution is only used to illustrate an assumed mechanism, and by no means represents the lowest upper bound estimate for the uplift resistance. This may also explain the lack of support by observations of this type of failure mechanism. Nevertheless, the solution illustrates that the uplift pressure increases with the embedment depth ratio H/D. As the embedment ratio increases, the failure mechanism becomes localised and independent of the embedment depth. The change in mechanism with embedment depth was observed experimentally in uplift tests on buried plate anchors whose behaviour was considered similar to that of buried pipes [5]. At deep embedment depths, the mechanism is similar to a laterally moving pile with soil flowing around the circular object. Randolph and Houlsby [6] presented plasticity solutions for the limiting pressure on a pile loaded in the lateral direction. They obtained exact solutions for piles with different roughness, but it was later discovered that a region of negative plastic work was omitted in the upper bound solutions. The revised limit solutions suggest that a rough pile (or pipe) has a limiting pressure of about $11.9s_u$, where s_u is the undrained shear strength of the soil. This implies that the simple mechanism shown in Fig. 3 is only more favourable than the flow around mechanism at small embedments, and the uplift resistance is bounded by $11.9s_u$. The flow around mechanism was also supported by observations in numerical studies simulating contractive soil which is equivalent to soil at great depths subjected to high confining stresses [7].

The fully bonded condition assumed in this scenario can also occur in the absence of adhesion as long as the embedment depth is deep enough. The high confining pressure drives soil to move around the pipe and push upward from the bottom of the pipe.

Scenario 2 — fully undrained and unbonded

If a gap forms underneath a buried pipe during uplift, the water pressure condition below the pipe will directly affect the uplift pressure. This scenario is illustrated in Fig. 4(a) in which the forces around an uplifting pipe are drawn. Two extreme cases are considered. In case 1 (Fig. 4(b)), it is assumed that the water pressure condition underneath the pipe is hydrostatic. From the free body diagram shown in Fig. 4(b), the total uplift force per unit length $P_{v,\text{total}}$ can be worked out as:

$$P_{v,\text{total}} = W_s + W_p + 2Hs_{u,\text{ave}} - \gamma_w\left(A_{\text{soil}} + \frac{\pi D^2}{4}\right) = W_s' + W_p' + 2Hs_{u,\text{ave}} \quad (2)$$

where W_s is the total weight of the soil block above the pipe per unit length; W_p is the total weight of the pipe per unit length; W_s' is the effective (buoyant) weight of the soil block above the pipe per unit length; W_p' is the effective (buoyant) weight of the pipe per unit length; γ_w is the unit weight of water; A_{soil} is the area of soil block above the pipe, and H_w is the depth of water above soil surface.

Eq. (2) suggests that effective weight should be used, for both the soil and the pipe, in the calculation of the uplift resistance if there is a gap underneath the pipe. The uplift pressure is again a function of the embedment depth H. In reality, the water pressure inside the gap below the pipe may be lower than hydrostatic. An additional suction force F_s (per unit length) should therefore be included in the uplift resistance. The net uplift resistance per unit length P_v can be written as:

$$P_v = P_{v,\text{total}} - W_p' = W_s' + 2Hs_{u,\text{ave}} + F_s. \quad (3)$$

In case 2, cavitation is assumed to occur underneath the pipe, leading to absolute zero pressure condition below the pipe as illustrated in Fig. 4(c). The absolute pressure is taken as 100 kPa. The total uplift force per unit length $P_{v,\text{total}}$ is given by:

$$P_{v,\text{total}} = 100D + \gamma_w H_w D + W_s + W_p + 2Hs_{u,\text{ave}}. \quad (4)$$

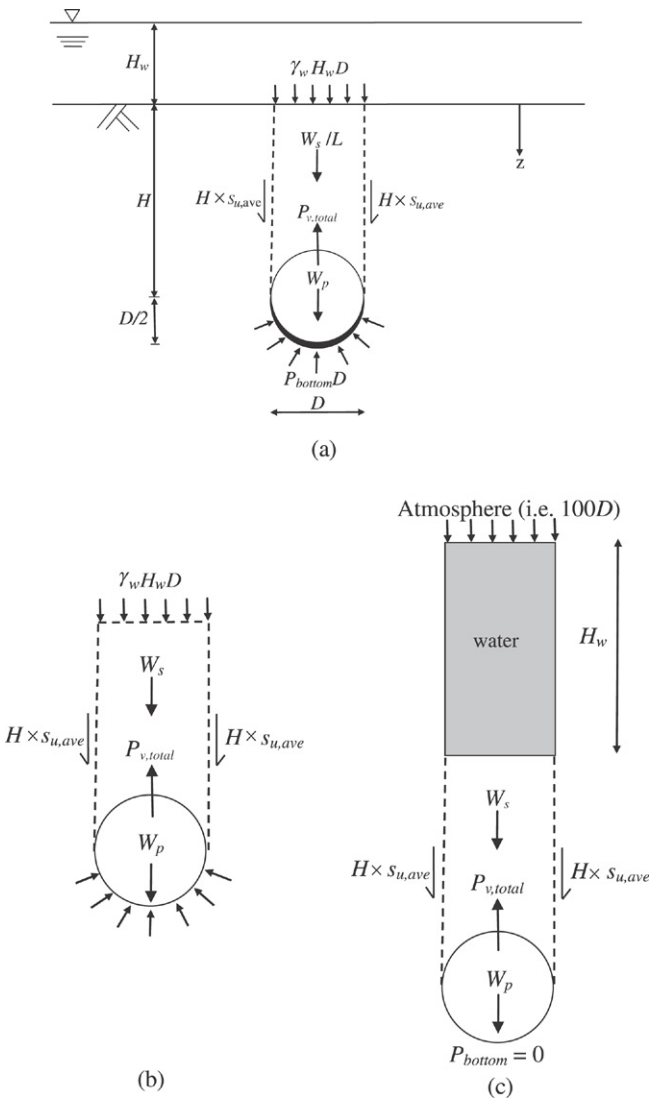


Fig. 4. Uplift mechanisms of an unbonded buried pipe in clay under undrained conditions: (a) Free body diagram; (b) Case 1 — hydrostatic conditions, and (c) Case 2 — cavitation occurs.

In this very extreme case, the contribution from soil shear strength is insignificant compared to other terms. This scenario is physically logical, but is unlikely to occur in reality.

Scenario 3 — fully drained

Under fully drained conditions, the total uplift resistance per unit length $P_{v,\text{total}}$ consists of the effective weight of the soil and the pipe, as well as the shear resistance along the failure surfaces as shown in Fig. 5. The net uplift force per unit length P_v is given by:

$$P_v = P_{v,\text{total}} - W'_p = W'_s + 2H\tau_{\text{ave}} \tag{5}$$

where τ_{ave} is the average drained shear resistance along the slip surface.

The likelihood of the occurrence of this scenario in clay is low due to the low soil permeability. In addition, the mechanism shown in Fig. 5 is only kinematically admissible for non-dilatant soil (i.e. dilation angle, $\psi = 0$). This is normally not the case for soil sheared under drained conditions at low

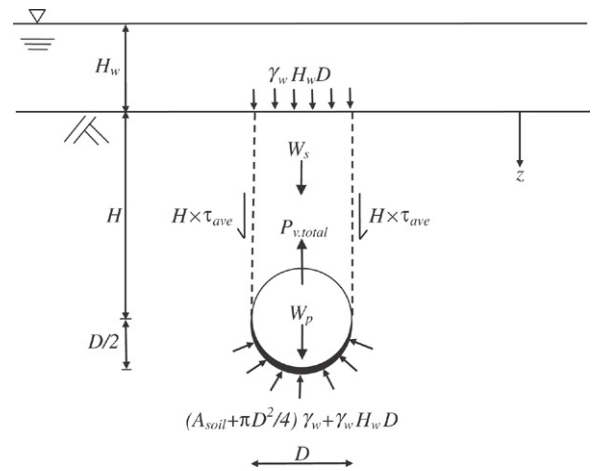


Fig. 5. Uplift mechanism of a buried pipe in clay under fully drained conditions.

confining stresses. When embedment depth is deep enough, the soil may flow around the pipe resembling the situation described in scenario 1 even in the absence of any adhesion force underneath the pipe under a fully drained condition.

In summary, the vertical soil restraint exerted on a pipeline buried in clay is dependent on: (1) the burial depth to diameter ratio (H/D); (2) the shear strength of the soil (s_u), and (3) the adhesion force on the underside of the pipe (F_s). If a cavity is created underneath the pipe, the effective weight of the soil (γ'_s) will also govern the uplift resistance available to the pipe.

Schaminée et al. [8] proposed that the uplift resistance of a soil cover could be non-dimensionalised to give an uplift factor, F_{up}, which can be used to compare soil uplift resistance for different soil states and conditions. This can also be used as a design parameter.

$$\frac{P_v}{\gamma' HD} = 1 + F_{up}\frac{H}{D} \tag{6}$$

$$F_{up} = \frac{D}{H}\left(\frac{P_v}{\gamma' HD} - 1\right). \tag{7}$$

3. Centrifuge tests

3.1. MkII mini-drum centrifuge

A centrifuge is a common tool to replicate the stress state in soil for a small scale physical model by providing elevated gravitational acceleration. The Mk-II mini-drum centrifuge at the Schofield Centre, Cambridge University Engineering Department, is equipped with a 180 mm wide ring channel of height 120 mm. It has a radius of 370 mm measuring from the base of the channel. The maximum spindle speed is 1067 rpm which corresponds to 471 times Earth's gravity (i.e. 471g) at the base of the channel. The centrifuge has a central pivot that allows a 90° rotation of the channel axis from the horizontal to vertical. This allows a model to be positioned in a convenient horizontal position inside the channel before spinning. More details can be found in Barker [9].

3.2. Apparatus

A schematic diagram, a side elevation and a photo of the centrifuge package are shown in Fig. 6(a)–(c) respectively. The specially designed model box comprises a thick glass wall which allows the side elevation to be viewed through a mirror angled at 45°. An on-board video camera is fastened above the model box to capture the view during testing. A 13.3 mm diameter brass model pipe with a length of 100 mm was used to mimic a 400 mm diameter pipe at prototype scale as the uplift tests were carried out at 30g. The pipe is suspended by two wires hanging from the actuator. The speed of the actuator can be adjusted from outside the centrifuge by a servo controller connected to the motor through the slip rings. Under the influence of the normal 1g component, the actuator and the model have to be fastened at a slope of 1:30 to ensure that the pipe is extracted parallel to the direction of the net acceleration.

The force required to lift the model pipe is measured by two miniature load cells fixed at the end of the wires. The corresponding displacement is recorded by a linearly variable differential transformer (LVDT) attached to the actuator. A set of 9 pore pressure transducers (PPTs) are positioned inside the strong box and held in place by aluminium towers.

3.3. Test material

Very soft clay collected from the Gulf of Mexico (GoM) was used in this study. Three cores have been extracted offshore from depths up to 1.8 m below mud-line. These cores were opened at the laboratory and the average in situ moisture content was measured to be about 105%. The average in situ undrained shear strength measured by a hand-held shear vane apparatus was about 2 kPa. Previous tests undertaken by Bolton and Take [10] found that the specific gravity (G_S) of the GoM clay was 2.49. This implies that the saturated unit weight of the soil is about 13 kN/m^3. The GoM clay is highly plastic with plastic and liquid limits of about 35% and 90% respectively. The coefficient of consolidation (c_v) measured from oedometer tests was 0.4 m^2/year, suggesting a very low permeability clay.

3.4. Preparation procedure

Due to the limited supply of the test material, the soft clay was re-used throughout the test programme. In order to ensure consistency between tests, the clay was gently mixed and remoulded with additional water before each test. The average moisture content of the clay at this preparation stage was measured to be about 150%. This is higher than the in situ value obtained from the core samples, thus mimicking the possible softening effect of the soil during pipeline installation through jetting.

The construction of the model took place in two stages. In the first stage, 1.6 kg of soft Gulf of Mexico clay was transferred to the model box in lumps. The lumps, with an average diameter of about 10 mm, were formed by a metal spatula. Due to the low strength of the lumps, the shapes were considered random,

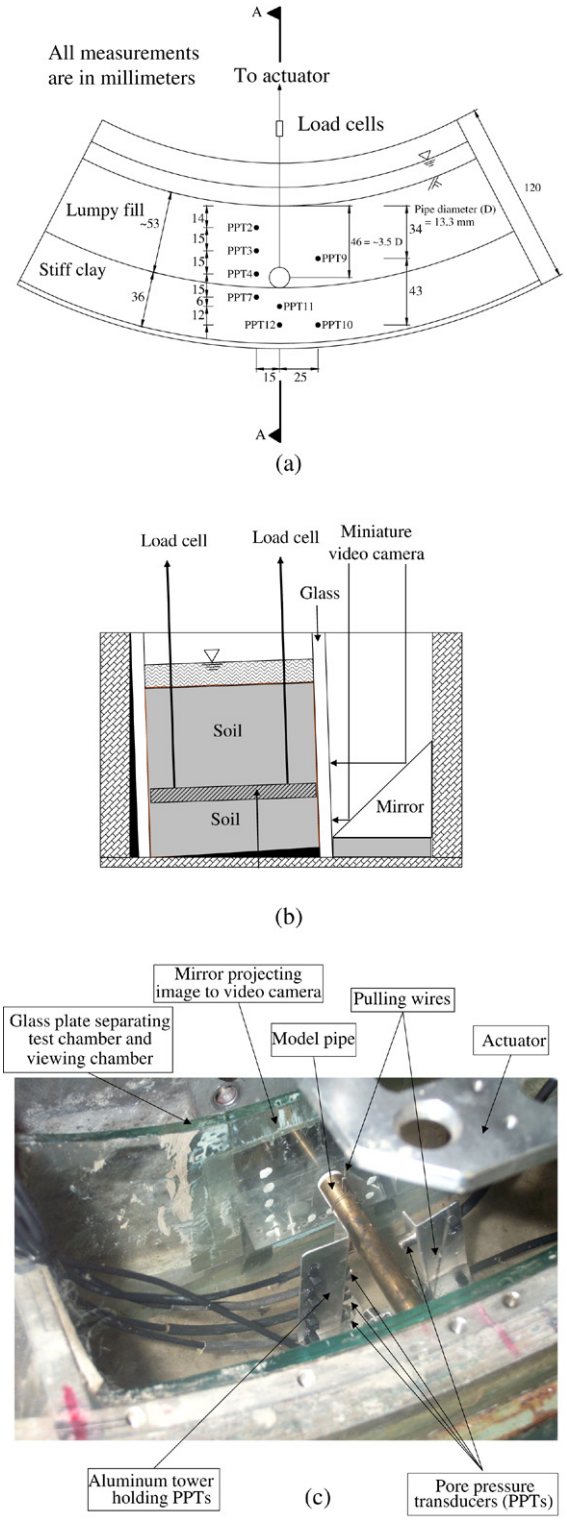
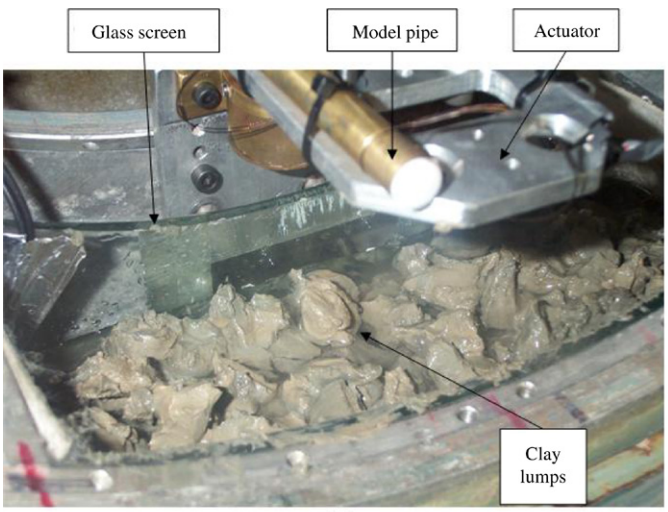


Fig. 6. The centrifuge package: (a) Schematic diagram, (b) side elevation (section A–A), and (c) photo taken before a test.

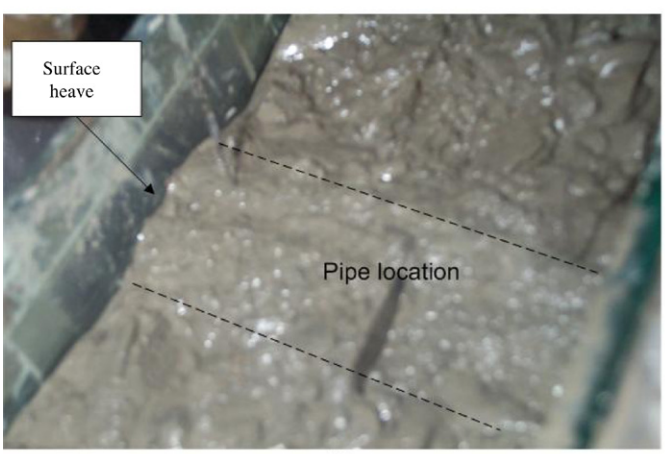
although effort was made to shape the lumps into spheres. The lumps were randomly placed into the model box under water (Fig. 7(a)). The lumpy fill was then consolidated under a fully submerged condition at 100g for 1 h, which corresponded to 14 months at prototype scale. The main aim was to build an over-consolidated soil layer which simulated the natural seabed

Table 1
Summary of the details of the centrifuge uplift tests

Test number	Consolidation time (Model scale) (h)	Consolidation time (Prototype scale) (months)	Uplift rate (Model scale) (mm/s)	Uplift rate (Prototype scale) (mm/h)	Total test duration for $\Delta_v = 1.5D$ (Prototype scale) (months)
GoM0	–	–	0.03	3.6	–
GoM1	10	∼12.5	0.0006	0.072	∼24
GoM2	10	∼12.5	0.03	3.6	∼12.75
GoM3	2.5	∼3	0.0006	0.072	∼14.5
GoM4	2.5	∼3	0.03	3.6	∼3.25
GoM5	2.5	∼3	0.0025	0.3	∼5.75
GoM6	2.5	∼3	0.0082	0.98	∼3.8



(a)



(b)

Fig. 7. Lumpy fill mimicking the disturbance to soil structure during pipeline installation process: (a) Before consolidation, and (b) after consolidation for 3 months (prototype) and pulling.

on which the pipe sat. The consolidated clay layer was then scraped to a depth of 36 mm.

In the second stage, the model pipe was laid down on the over-consolidated soil layer with some slack reserved in the wires connected to the load cells. The slack avoided additional load being imposed onto the load cells when the pipe was dragging downwards during consolidation. Due to this slack in the wires, zero pipe displacement during pull-out was defined at the moment when the load cells began to register a download force. An additional 1.9 kg of soft lumps was deposited into the model box. Upon completion, the centrifuge was spun up to 285 rpm, which was approximately equivalent to $30g$ at the centre of the pipe. The consolidation time of this phase varied from one test to another to study its influence on the uplift resistance. The target final thickness of the clay layer was 89 mm. This provided a cover height H_m (at model scale) of approximately 46 mm for the pipe (i.e. $H_m/D_m = 3.5$, where D_m is the pipe diameter at model scale). Once the consolidation period was over, the pipe was lifted up by the actuator at a prescribed speed. The resulting uplift force and the corresponding pipe displacements were measured. The entire test, from consolidation to pulling, was conducted under a fully submerged condition. Fig. 7(b) shows the surface of the lumpy fill after a test.

3.5. Test programme

Table 1 summarises the test programme. Test GoM0 is a calibration test to measure the buoyant weight of the model pipe. The results of this test are used to evaluate the net pull-out resistance provided by the soil cover. The test was carried out with the test chamber filled with water but no soil. The model pipe was then pulled in the same way as it would be in the real tests. The variations of pipe weight during extraction, due to a change in the g-level, are also quantified. In test GoM1, the lumpy fill covering the pipe was allowed to consolidate for a relatively long period of time (∼12 months prototype). The uplift speed was 0.0006 mm/s (equivalent to 0.072 mm/h at prototype) which was the lowest possible speed of the actuator. The results of this test provide a benchmark for soil uplift resistance in fully consolidated lumpy fill. Test GoM2 aims at assessing the influence of uplift speed in fully consolidated lumpy fill. The pipe was lifted at 0.03 mm/s (equivalent to 3.6 mm/h at prototype) which was about 50 times faster than in test GoM1. Tests GoM3 to GoM6 were conducted in under-consolidated lumpy fill. The consolidation phase lasted for only 2.5 h which was equivalent to 3 months at prototype scale. The actuation velocities varied from 0.03 mm/s down to 0.0006 mm/s at model scale.

The likely drainage regime during pipe uplifting can be assessed by the dimensionless group vB/c_v, where v is the pipe

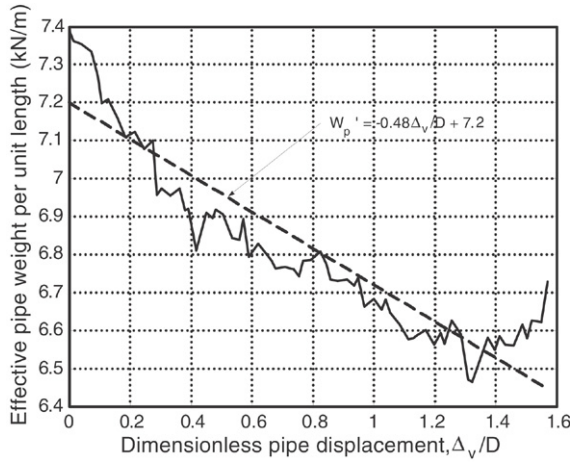


Fig. 8. Variation of effective pipe weight with upward pipe displacement.

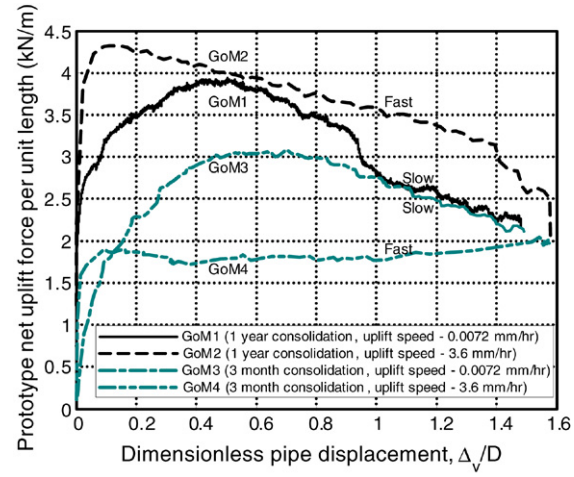


Fig. 9. Influence of uplift speed and the degree of consolidation on uplift resistance in lumpy fill.

velocity, c_v is the coefficient of consolidation and B is some characteristic drainage distance which can be taken as $\pi D/2$ for the case of pipe uplift [3]. Finnie [11] suggested that fully undrained conditions could be achieved if $vB/c_v > 10$, while $vB/c_v < 0.01$ implies drained behaviour. The dimensionless uplift speeds vB/c_v for the fastest (0.03 mm/s) and slowest (0.0006 mm/s) extractions are 98.8 and 1.98 respectively. This implies that a fully undrained event is ensured in all the fast pull-outs, but only partially drained behaviour is achieved even at the lowest uplift speed because of the low permeability of the soil.

4. Tests results

The results of the uplift tests are presented in this section at prototype scale unless stated otherwise. The net uplift force per unit length (P_v) at prototype scale is plotted against the dimensionless pipe displacement Δ_v/D, where Δ_v is the upward displacement of the pipe from its original position. In a 1:30 scaled model, the scale factors for force and displacement measurements are 900 ($=30^2$) and 30 respectively.

It may be useful to classify the failure mechanism involved in the centrifuge tests according to the different scenarios discussed in the previous sections. Photos taken by the on-board camera during the tests revealed that a cavity was formed underneath the pipe at very small pipe displacement. The pore pressure measurements obtained during the tests also confirmed that fully drained conditions were not achieved even in the test with the lowest uplift speed. Therefore, it can be concluded that scenario 2 described above is most relevant to the centrifuge tests.

4.1. Calibration test

The results of the calibration test GoM0, which aims at measuring the effective weight of the model pipe, are presented in Fig. 8. Since the pipe is shorter than the width of the chamber, the friction between the ends of the pipe and the test chamber is negligible. The measured uplift force in the calibration test is equivalent to the buoyant weight of the pipe. The results show that the uplift force decreases as the pipe moves upward. The reduction in the effective weight of the pipe is caused by the variation of g-level which is dependent on the location of the pipe. The centrifugal acceleration exerted on the pipe is proportional to its distance from the centre of the centrifuge; therefore the effective radius of the rotation reduces when the pipe is displaced upward.

In order to take this variation into account in the calculation of the net uplift force in the tests, a linear best-fit equation is employed to estimate the effective weight for a given pipe displacement:

$$W'_p = -0.48\frac{\Delta_v}{D} + 7.2 \tag{8}$$

where W'_p is the effective (buoyant) pipe weight per unit length, and Δ_v is the upward pipe displacement from the initial position of the pipe which is fixed in all the tests.

4.2. Uplift resistance in fully consolidated lumpy fill

The measured prototype uplift forces in tests GoM1 to GoM4 are plotted against dimensionless pipe displacement in Fig. 9. The reported values are obtained by subtracting the buoyant weight of the pipe, which is calculated from Eq. (8), from the total uplift force measured by the two load cells before dividing it by the length of the model pipe. The behaviour of the model pipe in fully consolidated lumpy fill during a slow uplift is demonstrated in test GoM1. The net uplift force increases linearly at very small displacements up to a pipe displacement of $0.015D$. Beyond this linear regime, the uplift force keeps increasing but with a decreasing stiffness in the load–displacement response. The uplift resistance eventually reaches a peak value of 3.9 kN/m at a pipe displacement of $0.42D$. The post–peak behaviour is a gradual reduction of uplift resistance. This reduction was partly due to the lessening of soil cover as the pipe was lifted up. Elimination of the suction force underneath the pipe, which will be discussed later, also contributed to the reduction of the uplift resistance, as may the reduction in shear resistance in the overlying soil.

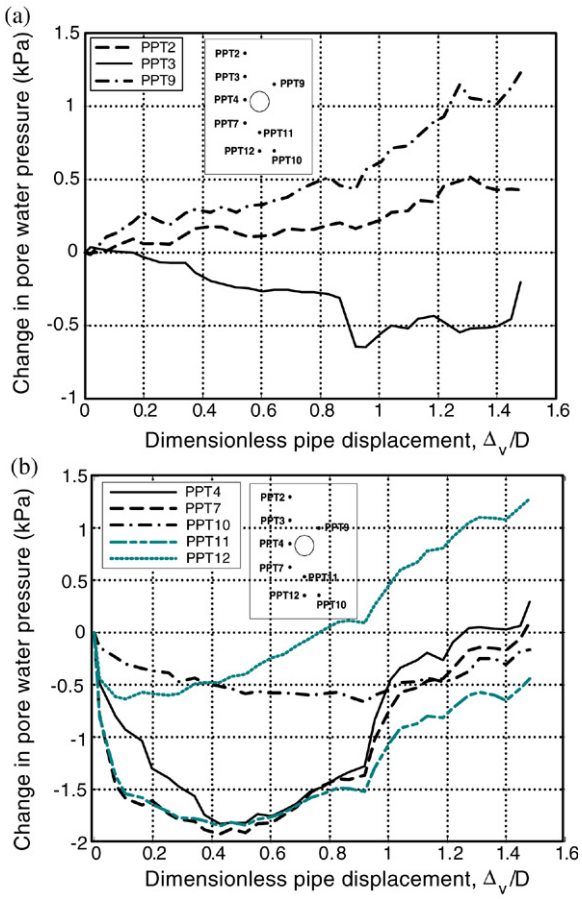


Fig. 10. Change in pore pressure during uplift in fully-consolidated lumpy fill (test GoM1): (a) Above the pipe, and (b) below the pipe.

The change in pore water pressure in the soil above and below the pipe in test GoM1 is shown in Fig. 10(a) and (b) respectively. As shown in Fig. 10(a), positive excess pore pressure was generated above the pipe at PPT2 and PPT9, whilst a small reduction in pore pressure was recorded at PPT3. Although the amount was small, the change in pore pressure indicated that fully drained conditions were not achieved even when the uplift was carried out at the lowest speed of 0.072 mm/h.

The change in pore pressures at PPTs below the pipe is more significant as shown in Fig. 10(b). All the PPTs measured a drop in pore pressure when extraction commenced. The maximum negative excess pore pressure was recorded at a displacement of $0.42D$. This is consistent with the maximum net uplift force reported in Fig. 9. This observation infers that the negative excess pore pressure generated underneath the pipe directly contributes to the uplift resistance by producing a downward force on the pipe. The maximum negative pore pressure recorded at PPT11 is about 1.85 kPa. Using the projected area of the pipe, a prototype suction force can be estimated as 0.74 kN/m, which is approximately 19% of the peak uplift resistance. After subtracting the suction force estimated from the reading of PPT11, the peak uplift resistance derived from soil shearing resistance is about 3.16 kN/m (3.9–0.74 kN/m).

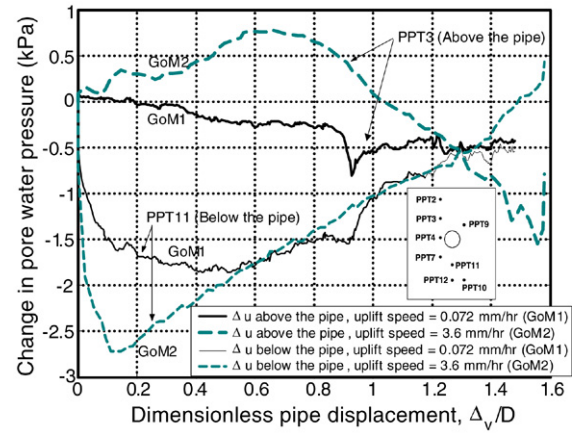


Fig. 11. Effect of uplift speed on pore pressure response in fully consolidated lumpy fill.

4.3. Uplift rate effect in fully consolidated lumpy fill

Tests GoM1 and GoM2, which were carried out at two different uplift speeds (0.072 and 3.6 mm/h), demonstrate the rate effect on uplift resistance of a pipeline in fully consolidated lumpy fill. Fig. 9 shows that the peak uplift resistances in these two tests only differ from each other by 10%. The associated pore pressure response is shown in Fig. 11. As discussed in the previous section, the negative excess pore pressure generated underneath the pipe contributes a significant portion of the total uplift resistance. Fig. 11 shows that the maximum negative excess pore pressure at PPT11 in test GoM2 is about 2.72 kPa. This corresponds to a suction force of 1.1 kN/m, which is about 25% of the peak uplift resistance.

Although the difference between the peak uplift resistance is only about 10%, the initial stiffness of the load displacement curves are remarkably different. The reduction in the mobilisation distance of peak uplift resistance is due to the rapid pore pressure response in the faster test. As shown in Fig. 11, the maximum negative pore pressure was recorded at a pipe displacement of $0.1D$ when the pipe was pulled at a higher speed (test GoM2). This caused the sharper response in the load–displacement curve. The difference between the maximum negative excess pore pressures in the two tests is about 0.87 kPa, which corresponds to a suction force of about 0.34 kN/m. This is comparable to the difference between the peak uplift resistances of the two tests (Fig. 9).

4.4. Uplift resistance in under-consolidated lumpy fill

In tests GoM3 and GoM4, the pipe was extracted after 2.5 h of consolidation (3 months at prototype). The influence of this relatively short period of consolidation on the uplift resistance can be seen in Fig. 9. When the pipe is extracted at a higher speed (3.6 mm/h in test GoM4), the peak resistance reduces substantially by 56% compared to that obtained in test GoM2, which has the same uplift speed but a much longer consolidation time. The peak uplift resistance is only about 1.9 kN/m as opposed to 4.32 kN/m in test GoM2. The reduction of the uplift resistance is associated with a reduction

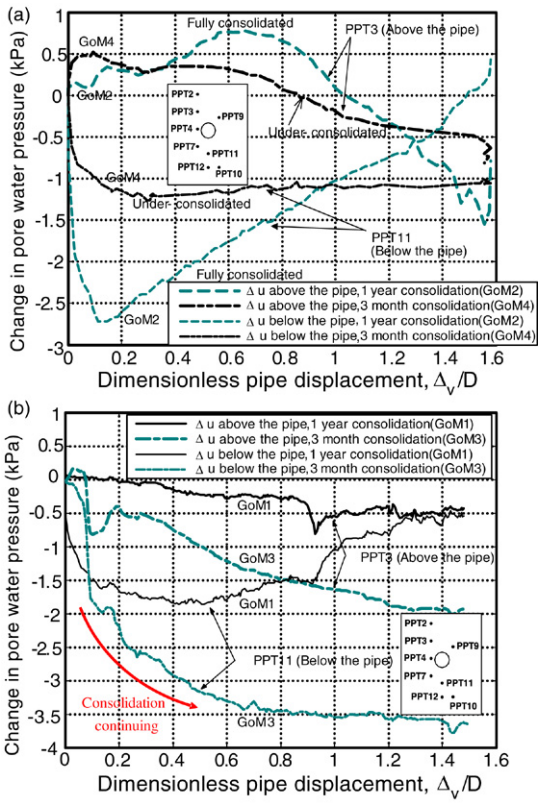


Fig. 12. Effect of degree of consolidation on pore pressure response: (a) During fast uplift, and (b) during slow uplift.

in negative excess pore pressure generated underneath the pipe as shown in Fig. 12(a), where the pore pressure change above and below the pipe is plotted against dimensionless pipe displacement. The difference between the maximum negative pore pressures is 1.47 kPa, corresponding to a difference in suction force of 0.59 kN/m. This magnitude is significantly smaller than the difference between the peak uplift resistances, suggesting that the incomplete consolidation also reduces the uplift resistance through a different mechanism, which is a reduction in shear strength of the soil above the pipe.

When the pipe is pulled slowly in under-consolidated lumpy fill (GoM3), the mobilised peak resistance is 21% lower than that of a fully consolidated soil (GoM1) as shown in Fig. 9. However, it is 63% higher than the measured uplift resistance during a fast uplift (GoM4). The effect of the under-consolidation is also to increase the pipe displacement required to mobilise the peak uplift force as the lumpy fill is still consolidating when the uplift resistance is mobilised. The associated pore pressure response is shown in Fig. 12(b). It can be seen that the pore pressures, both above and below the pipe, decrease substantially in the under-consolidated lumpy fill during uplift. The reduction of pore pressure is mainly caused by dissipation of excess pore pressures due to self-weight consolidation. In other words, the slow uplift speed allows the soil to carry on with the consolidation process before uplift resistance is mobilised. As shown in Table 1, the total duration of test GoM3 was about 14.5 months at prototype. The long duration led to a relatively high uplift resistance in the "initially" under-consolidated lumpy fill.

4.5. Uplift rate effect in under-consolidated lumpy fill and soil stiffness

In the previous section, it has been demonstrated that the influence of under-consolidation on the uplift resistance is dependent on the uplift speed which governs the degree of consolidation. The uplift resistances measured in all the tests expressed as an uplift factor, F_{up}, (calculated from Eq. (7)) are plotted against uplift speed in Fig. 13(a). The under-consolidated tests (GoM3-6) were carried out after an initial consolidation period of 3 months in prototype time.

There are two effects governing the resulting uplift resistance. When the uplift speed is low, the soil above the pipe has a chance to consolidate and gain a higher shear strength, hence increasing the uplift resistance. On the other hand, a higher uplift speed would produce a greater suction force underneath the pipe as demonstrated in the comparison between tests GoM1 and GoM2. Fig. 13(a) shows that the additional resistance obtained from the consolidation effect is more pronounced than that from suction effect for the selected uplift speeds, which is evident from the decreasing trend in uplift resistance as the extraction velocity is increased. In addition, the two tests on fully-consolidated lumpy fill (GoM1 and 2) have significantly higher uplift resistances irrespective of the uplift speed.

Among the under-consolidated tests, the lowest resistance was measured in test GoM4, which was pulled at the highest speed. When the speed was reduced, suction still contributed to the uplift resistance, albeit to a smaller extent. At the same time, the longer duration of the uplift process allowed a higher degree of consolidation, thus a higher uplift resistance. As far as under-consolidated lumpy fill was concerned, the highest uplift resistance was measured in GoM3, which had the lowest uplift speed.

The stiffness of the soil response is also governed by the uplift speed. This attribute is compared in Fig. 13(b) by plotting the stiffness parameter (k_{100}), which is defined as the peak uplift resistance divided by the corresponding mobilisation displacement, against the uplift speed. It can be seen that the stiffness of the response increases roughly with uplift velocity if the results are expressed on a logarithmic scale. The stiffness of fully consolidated lumpy fill is also found to be higher than those measured in under-consolidated fills at all uplift speeds covered in this study. In order to compare the initial stiffness, an alternative parameter (k_{50}) defined as 50% of the peak uplift resistance divided by the corresponding mobilization displacement is plotted against uplift speed in Fig. 13(c). Very similar trends with much higher magnitudes can be seen. This substantiates the conclusion that stiffness of the soil restraint increases with the uplift velocity of the pipe.

5. Discussion and conclusions

The results of the present study reveal that the process of pipeline installation by jetting may significantly reduce the

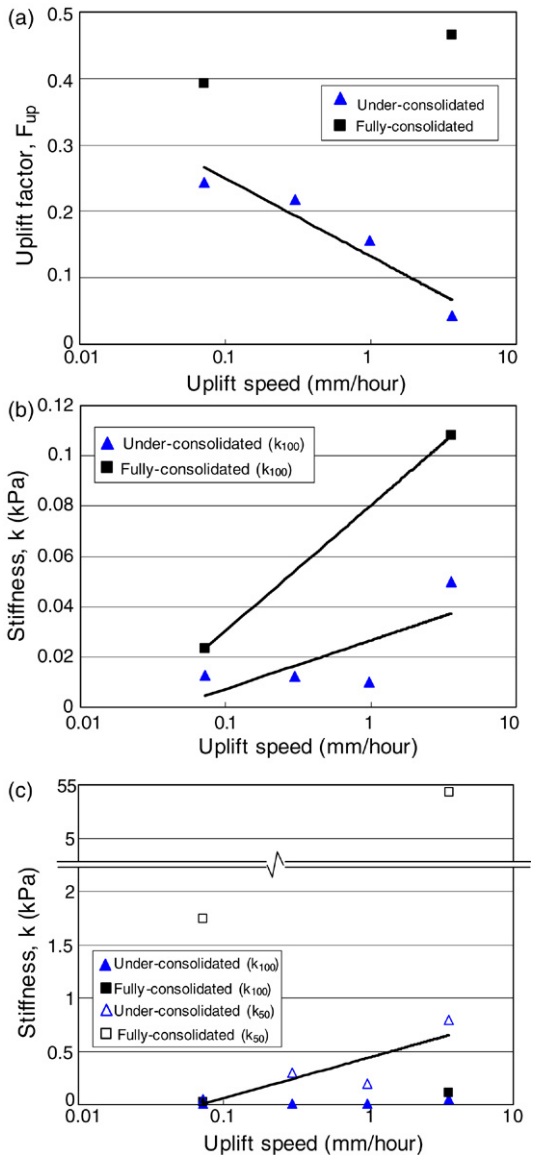


Fig. 13. Effect of degree of consolidation on: (a) Uplift resistance, (b) stiffness of the response corresponding to peak uplift resistance, and (c) initial stiffness of the response.

soil restraint exerted on a buried pipe if the disturbed soil lumps are used as the soil cover. The reduction mainly comes from insufficient consolidation time which in turn decreases the suction force generated underneath the pipe. As the suction force is highly dependent on the speed of the uplift process which cannot be determined in the field, the current design practice may not include the benefit of the suction force. Nevertheless, a simple quantitative analysis has illustrated that a loss of uplift resistance may also be contributed by the reduction in shear strength of the under-consolidated soil above the pipe.

The adverse effect of soil disintegration may vanish when the lumpy fill consolidates back to a homogenous fill. This took approximately 11 months (at prototype scale) for the selected soil and testing conditions. However, lumps of a single size were considered in this study. It is believed that as the sizes of the lumps change, the time required for complete consolidation will change accordingly. In practice, a newly installed oil pipeline has to be in service as soon as possible in order to shorten the idle period. It may not be cost effective to wait for the lumpy fill to consolidate. Designs should therefore consider the short-term uplift resistance provided by the under-consolidated lumpy fill if the pipeline is to be used shortly after installation.

The influence of uplift speed in fully consolidated lumpy fill was found to be small. The peak uplift resistance only reduced by about 10% when the uplift speed was lowered by 50 times from 3.6 to 0.072 mm/h at prototype scale. The major contributor of the difference was the high suction force generated underneath the pipe when it was pulled at a higher speed. Due to the different mobilisation mechanisms, the stiffness of the load–displacement curve was found to be a function of the uplift speed. When the uplift speed was reduced by 50 times, the stiffness of the response reduced by more than 50%, notwithstanding that the peak resistances only differed from each other by 10%.

For under-consolidated lumpy fill, the uplift resistance increased with decreasing uplift speed as the lumpy fill had a longer time for consolidation. Although the suction force generated underneath the pipe was proportional to the uplift speed, a slightly higher suction force was found to be less beneficial as compared to a higher degree of consolidation in the covering soil.

References

[1] Palmer AC, Ellinas CP, Richards DM, Guijt J. Design of submarine pipelines against upheaval buckling. In: Proceedings of 22nd annual offshore technology conference. 1990. p. 551–60.
[2] Cathie DN, Jaeck C, Ballard J-C, Wintgens J-F. Pipeline geotechnics — state of the art. In: Proc. international symposium on frontiers in offshore geotechnics. 2005. p. 95–114.
[3] Bransby MF, Newson TA, Brunning P. The upheaval capacity of pipelines in jetted clay backfill. International Journal of Offshore and Polar Engineering 2002;12(4):280–7.
[4] Bjerrum L, Eide O. Stability of strutted excavations in clay. Géotechnique 1956;6(1):115–28.
[5] Davie JR, Sutherland HB. Uplift resistance of cohesive soils. Journal of the Geotechnical Engineering Division, ASCE 1977;103(9):935–52.
[6] Randolph MF, Houlsby GT. The limiting pressure on a circular pile loaded laterally in cohesive soil. Géotechnique 1984;34(4):613–23.
[7] Vanden Berghe JF, Cathie D, Ballard JC. Pipeline uplift mechanisms using finite element analysis. In: Proceedings of 16th international conference of soil mechanics and foundation engineering. 2005. p. 1801–4.
[8] Schaminée PEL, Zorn NF, Schotman GJM. Soil response for pipeline upheaval buckling analyses: full-scale laboratory tests and modelling. In: Proceedings of 22nd annual offshore technology conference. 1990. p. 563–72.
[9] Barker HR. Physical modelling of construction process in the mini-drum centrifuge. Ph.D. dissertation. University of Cambridge; 1998.
[10] Bolton MD, Take WA. Pipeline uplift modelling. Cambridge University Technical Services Limited; 2000 (unpublished - confidential).
[11] Finnie IMS. Performance of shallow foundations in calcareous soil. Ph.D. thesis. University of Western Australia; 1993.