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# Pipeline Uplift Response in Thawed Sandy Backfills: Preliminary Findings

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## ABSTRACT

Offshore and onshore buried pipelines under high operating temperature and pressures may lead to upheaval buckling (UHB) if sufficient soil cover is not present to prevent the upward movement of the pipeline. In regions where seasonal changes involve ground soil undergoing freezing-thawing cycles, the uplift resistance from soil cover may be minimum when the soil is undergoing thawing. This paper presents the results from 2 directly-comparable minidrum centrifuge tests conducted at the Schofield Centre, University of Cambridge, to investigate the difference in uplift resistance responses between fully-saturated and thawed sandy backfill conditions. Both tests were conducted drained at 30g using an 8.6 mm diameter aluminium model pipe, corresponding to a prototype pipe diameter of 258 mm. The soil cover/pipe diameter ratio, H/D, was kept at 1. Fraction E fine silica sand was used as the backfill. Preliminary experimental results indicated that the ultimate uplift resistance of a thawing sand backfill to be lower than that of a fully saturated sand backfill. This suggests that in regions where backfill soil undergoes freeze-thaw cycles, the thawing backfill may be more critical than fully saturated backfill for uplift resistance. The 2-dimensional displacement field during the experiment was accurately measured and analysed using the Particle Image Velocimetry technique.

KEY WORDS: Thawed backfill, uplift resistance, centrifuge modelling

## INTRODUCTION

Pipeline networks are instrumental for transporting crude oil and gas both onshore and offshore. These pipelines are often buried underground for additional thermal insulation and protection against possible physical damage such as transportation, anchors and fishing activities. As the trenching and installation process has to be conducted at ambient temperatures, large compressive axial forces can develop during operational conditions of high internal temperature (typically 140°C) and pressure. At locations where the seabed profile exhibits an out-of-straightness (OOS) feature in the vertical plane, these compressive forces can cause global upheaval buckling (UHB) of the pipeline if insufficient uplift resistance is provided by the cover soil. In regions where seasonal temperature changes are high and soil undergoes freeze-thaw cycles, additional complications can occur. During winter time, the backfill soil is often frozen, but when summer comes, ice within the backfill thaws. Little is known regarding how differently a thawed or partially thawed backfill would respond to a pipe uplift event compared with an ordinary fully saturated backfill.

Over the past decade, many full-scale experiments and centrifuge tests using core-sampled soils have been carried out at the University of Cambridge to evaluate pipeline uplift resistance in different backfill conditions. The recently upgraded minidrum centrifuge at the Schofield Geotechnical Research Centre now features an enlarged diameter of 1.0 m and a widened channel width of 0.25 m. This paper presents the results from 2 directly comparable minidrum centrifuge tests simulating uplift events in both fully-saturated and thawed backfill conditions.

## BACKGROUND AND LITERATURE REVIEW

The initial breakout resistance of objects embedded underground consists of four components[1] (Versic, 1971), as illustrated in Fig. 1.

1. The submerged effective weight of the object, $W'_o$
2. The submerged effective weight of soil being lifted, $W'_s$
3. The vertical component of the soil shearing resistance, $S_v$
4. The vertical component of the suction force due to excess pore pressure differences above and below the object, $F_s$

Thusyanthan et al. (2008), Wang et al., (2009) and Bransby and Ireland (2009) proved experimentally that the total available uplift resistance would increase with pull-out speed due to increased contribution from $F_s$. Hence for conservative design, sufficiently low pull-out speeds

[1]Versic, (1971) includes a fifth component as the adhesion between the object and the adjacent soil. Schofield and Wroth, (1968) suggests that, in the absence of chemical bonding, adhesion arises from negative pore water pressures during soil dilation, hence is indistinguishable from $F_s$.

must be assumed so that $F_s$ is negligible. In addition, $W'_o$ is usually a constant unless the water table falls below the pipe crown. Hence a convenient definition for the "true uplift resistance", $R$, to be used directly in structural UHB resistance design can be defined by the following equation:

$$R = W'_s + S_v \tag{1}$$

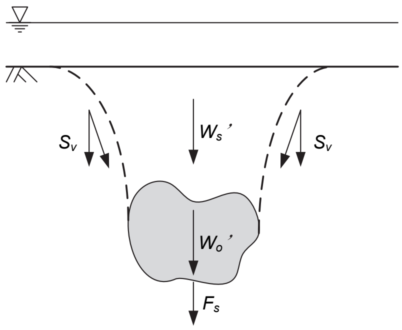


Fig. 1 Generalised break-out resistance for buried objects

A popular and widely-adopted method for evaluating $R$ is the Vertical Slip Surface Model as illustrated in Fig. 2, where $K$ is the lateral soil pressure coefficient during the uplift event.

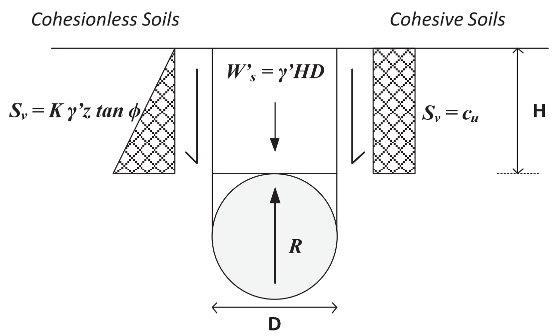


Fig. 2: The Vertical Slip Surface Model (after Schaminée et al., 1990)

Based on this model, the "true uplift resistance" $R$ can be expressed in terms of the following equation (DNV RP F110):

$$\frac{R}{\gamma' HD} = 1 + \left(\frac{1}{2} - \frac{\pi}{8}\right)\left(\frac{H}{D}\right) + f_p\left(\frac{D}{H}\right)\left(1 + \frac{D}{2H}\right)^2 \tag{2}$$

where the uplift resistance factor, $f_p$, is usually taken as a constant and equal to $K\tan\theta$ with $K$ being the lateral effective earth pressure coefficient and $\theta$ the inter-granular soil friction angle. As significant uncertainties exist in determining $K$, a single estimated value for $f_p$ is often sufficient for UHB design in cohesionless soils. Experimental work conducted by Baumgard, (2000), White *et al*., (2001), Cheuk, (2005) and Wang et al., (2010) verified that Eq. 2 could offer a good estimate of $R$ for H/D ratios between 0 and 6. However, the same experimental data also suggested that the actual deformation mechanism usually differ from what the Vertical Slip Surface Model suggests and would depend on the backfill relative density. White et al., (2001) proposed an alternative mechanism for cohesionless backfills featuring inclined slip-planes with the inclination angle equal to the angle of dilatancy during shear. Wang et al., (2011) further suggests that the form of Eq. 2 is independent of the inclination angle of the slip planes, and that the correctness of Eq. 2 relies on an aggregate assessment of $W'_s$ and $S_v$ together but not independently. Nixon (1998) conducted extensive pipeline uplift tests in frozen soils, and concluded that uplift resistance in frozen soils was highly strain-rate-dependent and the deformation mechanism featured extensive cracking. However, little is unknown how a thawed backfill after one freezing-thawing cycle would behave under pipeline uplift conditions, and whether Eq 2 is still applicable.

## EXPERIMENTAL METHODOLOGY

### Centrifuge Modelling

Centrifuge modelling has been used extensively in geotechnical research. It is based on the principal assumption that soil behaviour in a scaled-down model can be made homologous to that of the prototype by ensuring an identical stress state, and this can be achieved by subjecting the smaller model under centrifugal acceleration to create a higher artificial g-field. If the same soil is used in the model as in the prototype, and provided that the model has been subject to the same stress history as the prototype, the vertical stress within the model at depth $Z_m$ (model scale) will be identical to that in the corresponding prototype at depth $Z_p$ (prototype scale) where $Z_p = N \times Z_m$ with N being the ratio between the centrifuge acceleration and the earth's gravitational acceleration. Therefore, the model must be set up at 1/N scale of the prototype in every dimension. Schofield, (1980) offered a rather extensive reference table illustrating scaling laws for common geotechnical parameters.

The Schofield Centre (SC) is the main laboratory of the Geotechnical Research Group at Cambridge University Engineering Department and features a 10-m-diameter Turner beam centrifuge and a recently upgraded 1-m-diameter Schofield minidrum centrifuge. Compared with the beam centrifuge, the minidrum centrifuge is ideal for relatively simple experiments to be repeated for many times within a short period of time. Typical experimental cycle on the minidrum centrifuge is 1 day compared with 1 week on the beam centrifuge.

The minidrum centrifuge has a central pivot which allows a 90º rotation of the channel axis from the vertical to the horizontal position. This permits a model package to be prepared in a convenient position inside the channel before spinning, and then switched to the vertical position for spinning. Water is supplied directly to the base of the ring channel in-flight though inlet pipe. The water level in the ring channel can be varied by an adjustable stand pipe operated though an air motor and the same can be used to drain the water. The base of the ring channel has a radius of 500 mm measured from the central shaft. It can reach a maximum centrifugal acceleration of 636g when spinning at a maximum speed of 1067 rpm, where g is the acceleration due to gravity (i.e. 9.81 $m/s^2$). Practical geotechnical experiments in this centrifuge have traditionally been limited to 100g. The centrifuge features an on-board PC that provides 16 channels of data acquisition at up to 10 kHz. More structural and electronic details of the minidrum centrifuge can be found in Barker (1998).

Over the past few decades, many pipe uplift tests (Baumgard, 2000; White et al., 2001; Cheuk, 2005; Wang et al., 2009; Wang et al., 2010) have been conducted using the centrifuge described above, and the results obtained have shown excellent repeatability.

## Model Set-up

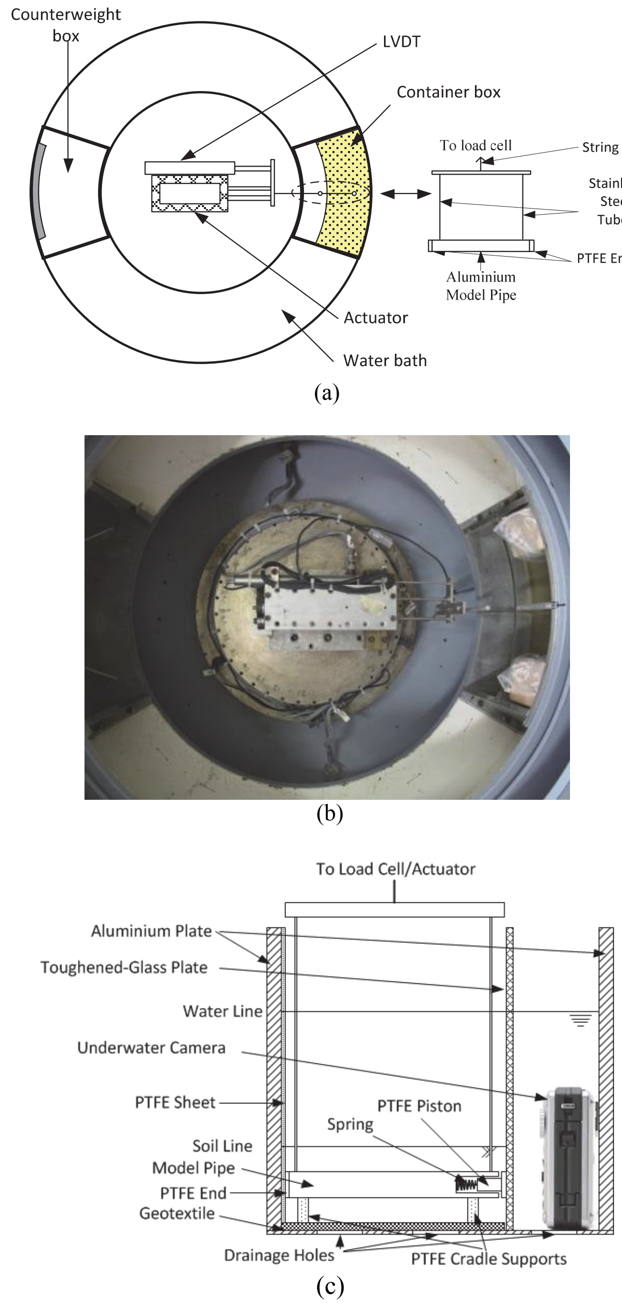


Fig. 3: Model set-up in (a) overall schematic view, (b) photography, and (c) container box cross-section view

The centrifuge model package set-up is illustrated in Fig. 3. The uplift movement is provided by an actuator mounted on the central turntable of the centrifuge that could run at constant speeds ranging from 0.002 mm/s to 0.2 mm/s and has a stroke length of 120 mm. The actuator is connected to the pipe by two stainless steel tubes via a horizontal aluminium balance bar parallel to the model pipe at a distance of 160 mm above it. The balance bar is then connected to the load cell via a central pulling wire made of a 0.6-mm-diameter stainless steel fishing line that has a safe working load of 500 N. The tensile stiffness of this fishing wire is so significant that its extension under the uplift load observed in these experiments is negligible. A Linear Variable Differential Transformer (LVDT) is mounted on the actuator in order to measure the pipe displacement. The instruments and control system were calibrated before and after the first test, showing no change in response or residual hysteresis.

The sides of the model container are made from 10-mm-thick aluminium plates, while the base is made from 1-mm-thick aluminium sheet to allow easier bending and curvature fitting. A 6-mm thick toughened glass sheet further separates inside of the container into two chambers: the soil chamber (125 mm wide) and the camera chamber (99 mm wide). A 1-mm thick geotextile layer is placed at the bottom of the soil chamber for controlled drainage. An additional 1-mm-thick PTFE sheet is glued to the interior of the aluminium side within the soil chamber to minimise end frictional effects. The model pipe assembly has a total length of 124 mm and an external diameter of 8.6 mm. The assembly features a 120-mm-long aluminium rod, a 1-mm-thick PTFE end-plate at one side, and a flexible spring-supported PTFE piston at the other side. The gap between the piston and the aluminium rod is filled up with silicon grease before model preparation begins so that soil particles cannot be trapped inside. This set-up not only minimises end frictional effects, but also ensures full contact between the pipe ends with the interior of the soil chamber so that in-flight photography can be carried out. The pipe assembly is supported at each end on a PTFE cradle during the model preparation phase of the test. When resting on the cradle, the invert of the pipe is located 1 pipe diameter above the base of the geotextile. The pulling wire connecting the load cell to the balance bar is initially loose, which then tighten as pullout begins. A Pentax Optio waterproof camera is fixed to the base of the container within the camera chamber to allow for underwater photography during the tests.

All numerical values quoted in this section are in model scale.

## Test program and preparation procedure

An initial test was conducted with the pipe submerged at intended water depth but without any soil. This test allowed the submerged weight of the pipe assembly and any frictional forces to be assessed so that they could be subtracted from the measured pull out resistance in subsequent tests in order to derive the true uplift resistance $R$ in accordance with Eq. 2.

The preliminary test series is comprised of 2 tests only, mainly as an attempt to examine the suitability and capability of the experimental apparatus. The first test examined the pipe uplift response under fully saturated in-situ backfill conditions. The second test, while keeping all other conditions identical, had passed the same backfill soil through one freezing-thawing cycle before pull-out began. The soil cover / pipe diameter ratio (H/D) ratio was 1.0 in both tests, i.e. 8.6 mm of soil above the pipe crown at model scale. Fraction E fine silica sand of relative density, $I_D$, of 30% was used consistently as the backfill. Saturated unit weight of this sand was 18.35 kN/m$^{-3}$. These backfill conditions were measured at the model preparation stage. Fraction E silica sand exhibits sufficient physical similarity to common shallow-water sandy seabed deposits. The particle size distribution (PSD) curve for this sand obtained from the Single Particle Optical Sizing (SPOS) method (White, 2003) is shown in Fig. 4. $D_{10}$ and $D_{50}$ for this sand could be taken as 0.09 mm and 0.14 mm respectively according to the sieving data. The backfill had excellent uniformity by employing the Centre's automatic sand pourer during the model preparation process (Chian et al., 2010).

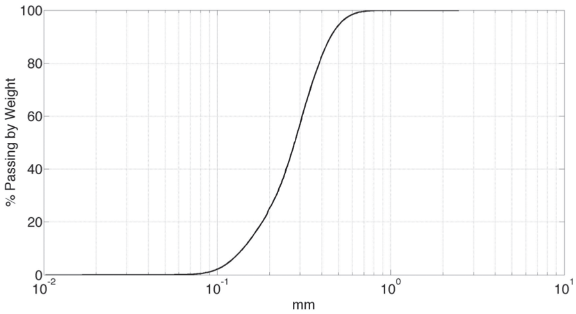


Fig. 4: Particle Size Distribution (PSD) curve for fraction E sand obtained from the SPOS method

Centrifuge acceleration was 30g during both tests. Hence the 8.6-mm-diameter model pipe represented a 258-mm-diameter pipe at prototype scale. Pull-out was displacement controlled at a constant speed of 1 mm/min model scale, which was slow enough to simulate fully drained conditions. Particle Image Velocimetry (PIV) technique was employed in both tests to reveal any differences in deformation mechanism. Photos were shot at 10-second intervals during the pull-up.

The model preparation and test procedure for the second test was:

1. A layer of sand was poured to the base of the container so that the sand surface is exactly at the level of the base of the PTFE cradle supports.
2. The model pipe was greased at both ends, placed on the cradle supports, and held vertically in place using aluminium tape.
3. Sand pouring continued until the soil surface is several millimetres above the desired cover-height.
4. The model was saturated, using de-aired water, from bottom upwards through capillary suction. The saturation process took typically 3 hours to complete.
5. The extra sand was scrapped off the soil surface to achieve the desired H/D ratio for the model.
6. The entire model container was placed within a freezer of internal temperature -7°C for 12 hours.
7. The container was taken out of the freezer and left to warm up until the soil surface showed signs of softening while deeper soil remained frozen. Measured soil surface temperature was -1.5 °C at that instant.
8. The container was placed onto the minidrum centrifuge with camera and lighting securely set up. The centrifuge axis was then rotated from horizontal to vertical. Capillary suction ensures that the model soil remained in place.
9. The centrifuge was accelerated to 10 g.
10. Water was slowly added to the base of the centrifuge ring channel until the model soil surface was fully submerged.
11. The centrifuge acceleration was increased to 30 g.
12. The pipe was pulled out at a rate of 1 mm/min at model scale until the pipe was fully out of the soil surface. Data recording and PIV photography commenced simultaneously.

Similar procedures were adopted for the first test, except that steps 6 and 7 were omitted. During the freezing process, the drainage holes at the bottom of the model container were fully air-borne, and the backfill condition was held constant by capillary suction. Hence no significant excess pore pressure could develop during the freezing process. Steps 7 to 11 took 35 minutes to complete to allow for substantial thawing to take place. Step 12 took 20 minutes to complete. Hence pull-out began when the backfill was fully thawed.

# TEST RESULTS

## Uplift Force-Displacement Response

The recorded true uplift force ($R$) vs. pipe upward displacement ($\delta$) data is shown in Fig. 5. The shapes of the two curves are very similar: $R$ value ascends steeply to peak, falls rapidly for a short region of additional $\delta$, and then the response softens almost linearly to approximately zero. Significant differences are observed, however, in the respective values of key design parameters from these tests: the recorded maximum available true uplift resistance ($R_{peak}$), and the required pipe upward displacement to achieve this value ($\delta_f$). Detailed comparisons are given in Table 1. For completeness, a test result from a previous series (Wang, et al., 2010) was also included for comparison. This previous test was carried out at almost exactly the same conditions as the first test, except that the backfill sand was prepared "dense" at a relative density of 85%. It should be noted that this previous test was not intended to measure mobilisation distance accurately, and hence slacks in the system resulted in exaggerated pre-peak displacement data. Previous research (Wang et al., 2010) also suggests that centrifuge tests seem to offer more conservative $f_p$ values than full-scale tests. The $f_p$ values for both tests in the current series appeared much lower than that of the previous dense-sand test. This signified the influence of the dilation angle $\psi$ on the available uplift resistance: the effective confining stress directly above the pipe crown is approximately 2.5 kPa in both tests; taking an aggregate crushing stress of 10,000 kPa for Fraction E sand under centrifuge conditions, the dilation angle $\psi$ could be estimated (Bolton, 1986) as at 6° for loose sand but as high as 20° for dense sand. Hence the inter-particle friction coefficient, tan $\theta$, was 1.3 for dense sand while 0.8 for loose sand, a difference exceeding 60%. In addition, loose backfills was more likely to trap air bubbles during the saturation process, resulting in even lower $f_p$ values.

Table 1: Comparison of key design parameters, all values at prototype scale

| Test | $\gamma_{sat}^{*}$ (kN/m$^3$) | $R_{peak}$ (N/m) | $f_p$ |
|---|---|---|---|
| Loose, saturated | 18.35 | 847 | 0.18 |
| Loose, saturated, thawed | 18.35 | 744 | 0.10 |
| Dense, saturated | 19.42 | 1505 | 0.55 |

*: Saturated unit weight measured at model preparation, i.e. pre-test

At the same H/D ratio and backfill conditions at preparation, the recorded $R_{peak}$ for the thawed backfill case is 12% lower than the value from the fully-saturated loose-sand test. This suggests that thawing sand backfill to be critical for UHB design of pipelines. Two hypotheses have been identified as possible explanations:

1. The freezing stage caused the backfill soil to heave and loosen up, reducing the effective relative density at actual pull-up. Given the low confining stress levels common in UHB design, any further reduction in relative density could have had significant influence on $\psi$ and hence tan $\theta$.
2. Thawed conditions partially changed the inter-particle frictional media from sand-sand to sand-ice, resulting in a reduction in frictional coefficient (tan $\theta$)

Many additional tests will be needed if any firm conclusions can be drawn on the correctness of either hypothesis.

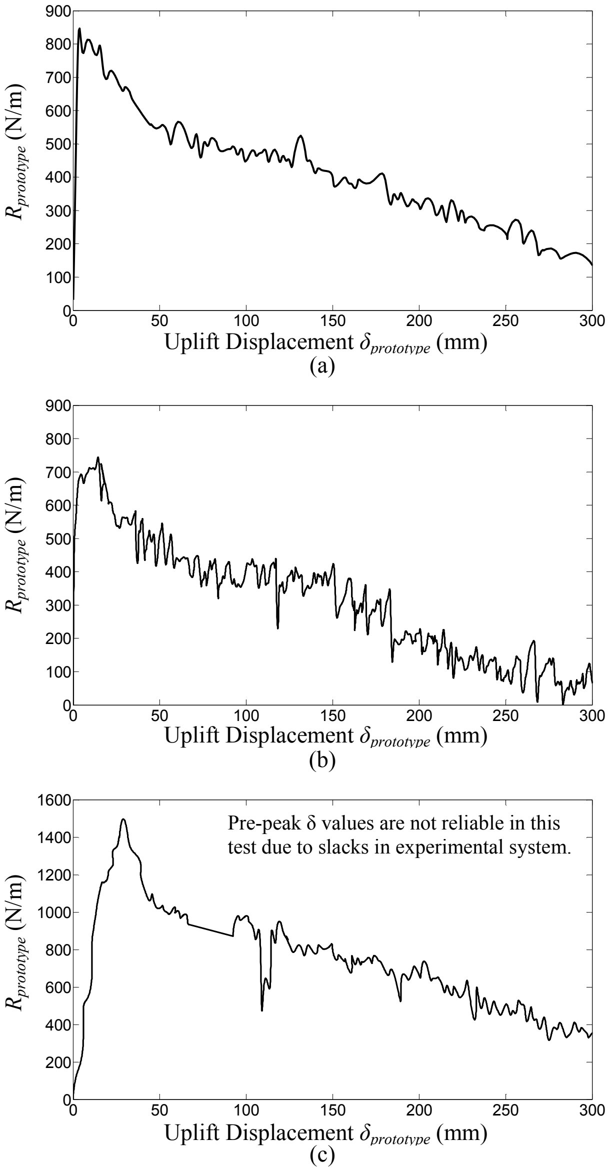


Fig. 5 Uplift force-displacement response from (a) the first test (fully saturated backfill), (b) the second test (thawed backfill), and (c) dense sand test at same H/D from Wang et al., (2010); all numerical values are at prototype scale

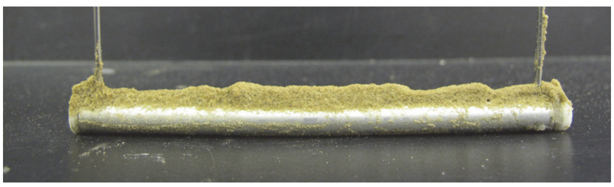

Fig. 6 Residual soil on top of the model pipe after the pull-out in Test 2

It should be noted that the recorded $R$ values should never reach the zero position even after the model pipe has been pulled completely out of the soil surface beyond a prototype-scale upward displacement of typically 300 mm. This is because some sand particles will remain on top of the model pipe as illustrated in

Fig. 6. Frictional forces between the PTFE-PTFE and PTFE-glass interfaces on the two sides of the model pipe could also vary at different pipe upward displacements, although such deviations could be deemed small.

Fig. 6 also illustrates that the backfill soil was fully thawed and not frozen to the steel pulling tubes during the test.

## Deformation Mechanism

In the first test, the 2-dimensional soil displacement field during the uplift event was accurately measured at 10-second intervals, which corresponds to 0.167 mm of upward displacement by the model pipe. This was achieved using the non-contact digital image correlation technique of particle image velocimetry (PIV), described in detail by White et al., (2003). Selected PIV analysis results for this test are shown in Fig. 7. Comparing with the Vertical Slip Surface Model (Fig. 2), the actual mechanism featured a wider region of soil being lifted during the uplift event, as well as flow-around and triangular wedge-formation at the bottom of the model pipe when uplift displacement exceeded 3 mm at model scale.

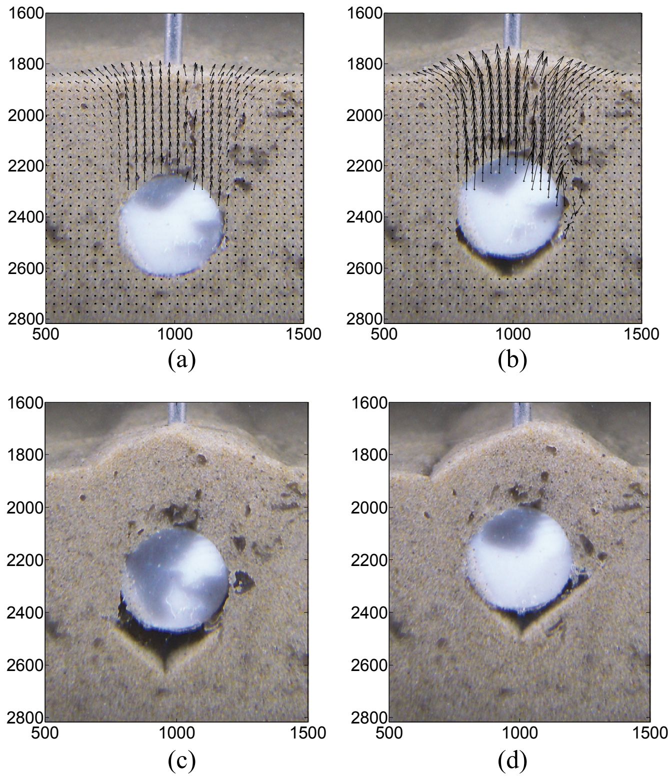


Fig. 7 Deformation mechanism in Test 1 after a model-scale pipe upward displacement of (a) 1.67 mm, (b) 3.33 mm (near peak), (c) 5 mm, and (d) 6.67 mm, with axes in camera pixels

The deformation mechanism in Test 2 was very similar to that of Test 1 with no cracks observed during post-test excavation. Neither was there any indication of frozen soil. Both observations were consistent with the intended backfill condition for Test 1 as being fully thawed.

## CONCLUSIONS

This paper presents the results from 2 directly-comparable minidrum centrifuge tests conducted at the Schofield Centre. The purpose of these tests was to investigate the difference in uplift resistance response between fully-saturated and thawed sand backfill conditions. All other parameters were kept the same. The tests were conducted at 1:30 scale, using an aluminium model pipe of external diameter 8.6 mm (258 mm prototype scale) buried in fraction E silica fine sand at an H/D ratio of 1. The model pipe was pulled out at a very slow rate (1 mm/minute), whilst the uplift resistance was measured along with underwater cross-sectional photography at 10-second intervals.

The recorded uplift force-displacement trends were similar for both tests. However, the peak uplift resistance for the thawed backfill case was approximately 12% lower than that of fully-saturated case. This finding could pose potential UHB design concerns for buried pipelines in regions where the backfill may encounter freeze-thaw cycles, as thawed backfill was seen to provide lower resistance to uplift. A likely explanation is that when frozen soil is undergoing thawing, there will be a stage in which sand particles may not have full contact with the neighbouring sand particles due to the presence of partially melting ice. This will in turn reduce the frictional behaviour of thawing soil compared with fully submerged cases.

It is to be noted that these tests are preliminary and further research in this area is required to verify these initial finding.

The PIV technique allowed detailed 2-dimensional displacement field to be measured very accurately, and the recorded deformation mechanism is compared with the popular Vertical Slip-Surface model widely used in current design practice.

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## REFERENCES

Barker, H.R. (1998). "*Physical modelling of construction process in the mini-drum centrifuge*". PhD dissertation, University of Cambridge

Baumgard, A.J. (2000). "*Monotonic and cyclic soil response to upheaval buckling in offshore buried pipelines*". PhD Thesis. University of Cambridge

Bolton, M.D. (1986). "*The strength and dilatancy of sands.*" Géotechnique **36** (1): 65-78.

Bransby, M.F. and Ireland, J. (2009). "*Rate effects during pipeline upheaval buckling in sand*". Geotechnical Engineering. **162**: 247-256

Cheuk, C.Y. (2005). "*Soil pipeline interaction at the seabed*". PhD thesis. University of Cambridge.

Chian, S.C., Stringer, M. & Madabhushi, S.P.G. (2010), "*Use of Automatic Sand Pourer for Loose Sand Models*", 7th International Conference on Physical Modelling in Geotechnics, Zurich

DNV-RP-F110 (2007). "*Global buckling of submarine pipelines - structural design due to high temperature / high pressure*". Norway, Det Norske Veritas.

Nixon, J.F.D. (1998). "*Pipe uplift resistance testing in frozen soil*". Proc. Permafrost – Seventh International Conference, Yellowknife, Collection Nordicana **55**: 821 – 831.

Schaminée, P.E.L., Zorn, N.F. & Schotman, G.J.M. (1990). "*Soil response for pipeline upheaval buckling analyses: full scale laboratory tests and modelling*". Proc. Offshore Technology Conference, Houston OTC6486 563-572

Schofield, A.N. (1980). "*Cambridge Geotechnical Centrifuge Operations*". Géotechnique **30** (3): 227-268.

Thusyanthan, N.I., Ganesan, S.A., Bolton, M.B., and Allan, P. (2008). "*Upheaval buckling resistance of pipelines buried in clayey backfill*". 18th (2004) Int. Offshore and Polar Eng Conf, Vancouver, Canada, ISOPE, Vol.1, pp. 174–180

Vesic, A.S. (1971). "*Breakout resistance of objects embedded in ocean bottom*". ASCE Journal of the Soil Mechanics and Foundation Division.**97** (9): 1183-1205

Wang, J., Ahmed, R., Haigh, S.K., Thusyanthan, N.I., and Mesmar, S. (2010). "*Uplift resistance of buried pipelines at low cover-diameter ratios*". Proc. Offshore Technology Conference. May, 2010, Houston, USA

Wang, J., Haigh, S.K., and Thusyanthan, N.I. (2009). "*Uplift resistance of buried pipelines in blocky clay backfill*". Proc. International Offshore (Ocean) and Polar Engineering Conference. ISOPE 2009 TPC 564

Wang, J., Haigh, S.K., and Thusyanthan, N.I. (2011). "*Predicting Mobilization Distance for Upheaval Buckling of Buried Pipelines in Sandy and Rocky Backfills*". Paper submitted to the ASCE Pipelines Journal

White D.J. (2003). "*PSD measurement using the single particle optical sizing (SPOS) method*". Géotechnique. **53** (3): 317 - 326.

White, D.J., Barefoot, A.J., Bolton, M.D. (2001). "*Centrifuge modelling of upheaval buckling in sand*". International Journal of Physical Modelling in Geotechnics, Vol. 2 1:19-28

White, D. J., Take, W. A. & Bolton, M. D. (2003). "*Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry*". Géotechnique, **53** (7): 619–631.