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Upheaval buckling resistance of pipelines buried in clayey backfill

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ABSTRACT

This paper presents data from a series of Minidrum Centrifuge tests in which the effects of backfill cover (1 m & 1.3 m) and rock-dump (0.5 m) thickness on the uplift resistance were investigated. All the centrifuge tests were carried out at 30g using natural marine clay. The natural clay samples from offshore were characterised and reconstituted before testing. Field backfill conditions were simulated close to reality in the testing. In each of the tests, the resistance of soil cover, the vertical pipe displacement, and excess pore pressure changes at the pipe invert were measured. The results from this study are compared against the current framework of upheaval buckling behaviour in the literature, and are used to provide a better guideline for the design of offshore pipelines buried in clayey backfills.

KEY WORDS: Upheaval buckling; pipelines; backfill; clay; uplift resistance.

INTRODUCTION

Predicting upheaval buckling resistance of buried pipelines has been a challenge as there is a huge uncertainty and randomness in the nature of soil cover created by various pipe burying techniques. Present understanding on uplift resistance of buried pipe lines is based on analysis (Randolph and Houlsby, 1984; Maltby and Calladine, 1995) and experimental work by researchers (Cheuk et al, 2005; White et al, 2001; Bransby et al, 2002; Baumgard, 2000; Dickin, 1994; Finch, 1999; Moradi & Craig, 1998). However, almost all the experimental work on uplift resistance was carried out on granular soils, and there is a lack of experimental work on clay backfill (Cheuk et al. 2007).

This paper presents data from a series of Minidrum Centrifuge tests in which various factors affecting the upheaval buckling resistance were investigated. The factors investigated were depth of burial, time interval between the pipeline burial and commissioning, rate of pipe pull-out, and depth of rock dump. All the centrifuge tests were carried out at 30g on a natural marine clay. The natural clay samples were characterised and reconstituted before testing. Field backfill conditions were simulated close to reality in the testing. In each of the test, the resistance of soil cover, the vertical pipe displacement, and excess pore pressure changes at the pipe invert were measured. The results from this study are compared against the current framework of upheaval buckling resistance behaviour in the literature, and are used to provide a better guideline for the design of pipeline buried in clay backfills.

A total of 4 tests were conducted on a 1 in 30 scale model. The prototype pipe was 261 mm in diameter (8.7 mm at model scale), and was buried under clay backfill. Tests 1 and 2 were conducted to measure the uplift resistance of clay covers of depth 1.30 m and 1.05 m respectively, after 2 months of backfilling. Tests 3 and 4 were undertaken to measure the uplift resistance of a clay cover of depth 1.05 m overlain by a layer of rock-dump of depth 0.5 m and 1.0 m, respectively. In these tests (tests 3 & 4), the clay cover was allowed to consolidate for one month before rock-dumping was carried out. The clay was then permitted to consolidate for another month under the weight of rock-dump before the pipe was pulled up.

REVIEW OF LITERATURE

The uplift resistance per unit length of pipe, F, comprises (i) the weight of the soil above the pipe and (ii) the mobilised shearing resistance of soil. The peak value of F can be interpreted within an effective stress or an undrained strength framework. The conventional interpretation of pipe uplift resistance involves vertical sliding planes above the pipe, with the geometry and nomenclature as shown in Figure 1.

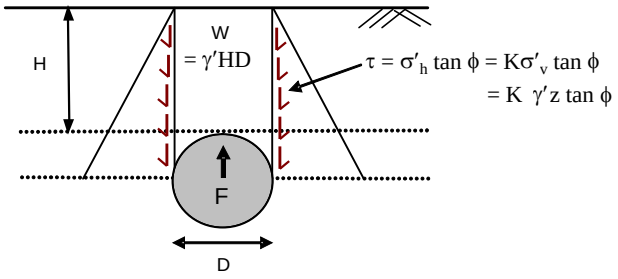

Figure 1. Vertical shear model for pipe uplift resistance.

The resulting resistance comprises the overburden weight ($W = \gamma' HD$) and the shear stress ($\tau = \sigma'_h \tan\phi = K\sigma'_v \tan\phi = K\gamma' z \tan\phi$) on the

vertical slip planes. The effective stress frame work of the Pedersen model (Cathie et al. 2005) uses the whole volume of soil above the pipe and is given in equation (1).

$$\frac{F}{¥'DH} = 1 + 0.1\frac{D}{H} + f_p\left[\frac{H}{D}\right]\left[1 + \frac{D}{2H}\right]^2 \quad (1)$$

For undrained behaviour, the equivalent vertical slip model leads to equation (2) (Cathie et al. 2005).

$$F = ¥'HD + 2s_u H \quad (2)$$

In this paper, the uplift factor, f_p, has been calculated using a constant value of cover depth, H, rather than modifying this value during pullout to reflect the changing height of soil cover.

For a deeply embedded pipe, the uplift failure mechanism involves flow of soil around the pipe periphery. Beyond a critical embedment, $(H/D)_{deep}$, this mechanism offers lower resistance than the heave mechanism shown in Figure 1, due to the increasing length of the idealised shear planes.

Previously reported data from drained uplift of pipes – albeit in sand rather than clay backfill – indicates that the depth at which peak uplift becomes governed by a flow-round mechanism (rather than heaving) is typically H/D ≥ 4 for loose backfill (vanden Berghe et al. 2005, White et al. 2001, Schupp et al. 2006).

Palmer and Richards (1990) proposed the following to predict the uplift resistance for deep flow failure.

$$F = s_u D \min\left[3, \frac{H}{D}\right] \quad (3)$$

EXPERIMENTAL METHODOLOGY

Centrifuge model

The centrifuge model consists of a model container, an actuator, and a model pipe. The general setup of the model package is shown in Fig. 2 and Fig. 3. The bottom of the model container was provided with a layer of geotextile and filter paper to allow drainage during consolidation and testing. All the centrifuge tests were carried out at 30g.

The model pipe can be moved vertically upwards by a displacement controlled actuator. The actuator was mounted on the central turntable of the Minidrum Centrifuge. The actuator 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 pipe uplift resistance was measured by two load cells mounted at the end of the actuator's moving arm (Fig. 3). The model pipe is connected to the load cell through nylon coated stainless steel wire of 0.6 mm diameter and has a safe working load of 50 kg. These thin wire minimise the disturbance caused to the clay backfill or rock dump to a large extent. A displacement transducer was mounted on the actuator to measures the vertical displacement.

The model pipe was made of aluminium. Its diameter was 8.7 mm (the prototype diameter was 261 mm; a 1 in 30 scale model) and length was 120 mm. The pipe was supported on two aluminium saddles during consolidation of the backfill so as to prevent any undesirable pipe movement and drag force coming on to the pipe. Both the actuator and the model pipe were oriented on a 1 in 30 slope so that the resultant of the centrifugal acceleration and the earth's gravity will be normal to the model orientation.

Pore pressure transducers (PPTs) were placed below the pipe invert and on the slope of the trench for monitoring the change in excess pore pressure during consolidation and pipe pullout.

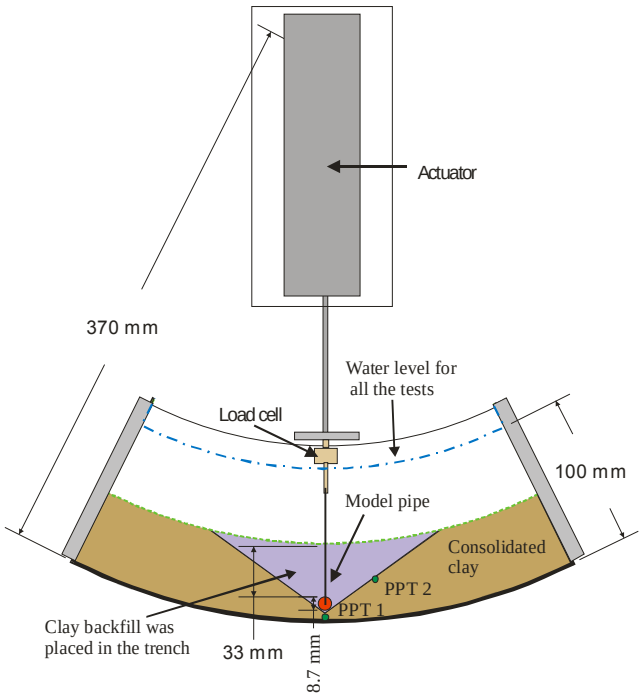

Figure 2. Side view of the centrifuge model

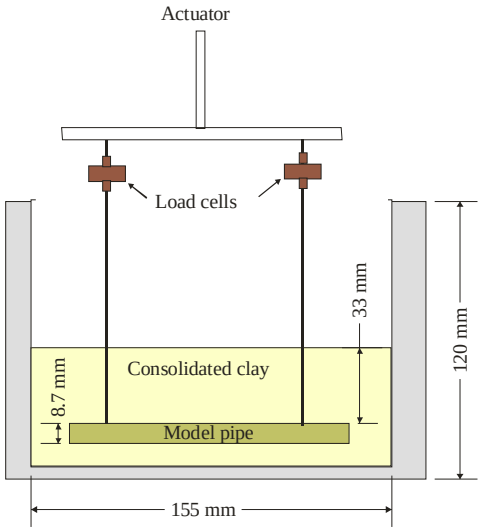

Figure 3. Cross section of the model

Test material

Clay sample

The undisturbed clay samples were obtained from offshore in cores. They were mixed together, reconstituted with saline water, and homogenised. This homogenised sample was then used for the pipe pullout testing. An oedometer test was performed on the homogenised sample and the coefficient of consolidation was found to be 0.05 mm^2/s (1.8 m^2/year). The homogenised samples were also tested for liquid limit and plastic limit, and were found to be 49 % and 15%, respectively.

Gravel (used for rock-dump simulation)

Tests 3 and 4 involved simulation of rock dumping over the clay backfill. Angular and rounded aggregates sieved through 4 mm sieve were used for this purpose. The size of the prototype rock-dump material was about 100 mm.

Figure 4. Gravel used as rock-dumb

Test program and procedure

An initial test was conducted with an empty test container with pipe submerged in water so as to assess the submerged weight of the pipe and pulling wires in-flight at 30g. This force was then subtracted from the measured pull out resistance in subsequent tests in order to obtain the uplift resistance offered only by the clay cover. A total of four tests were performed at 30g, wherein two tests were with only clay backfill (no rock dump) with cover depths 1.3 m and 1.05 m, and the other two tests were conducted on a clay backfill with cover depth 1.05 m overlain by a layer of rock-dump of depths of 0.5 m and 1.0 m. Test programme of the 4 tests is summarized in Table 1.

Table 1. Summary of Centrifuge tests (details are given at prototype scale).

Test	Prototype cover depth, H (m)	Rockdump thickness (m)	Test description
1	1.30	No rock dump	2 months after backfilling
2	1.05	No rock dump	2 months after backfilling
3	1.05	0.5	2 months after backfilling (rock- dump was placed one month after backfilling)
4	1.05	1.0	2 months after backfilling (rock- dump was placed one month after backfilling)

The testing phase involved three distinct stages:
a) model seabed preparation,
b) trench cutting and pipe burial, and
c) backfill consolidation and pipe pullout.

(a) Model seabed preparation

The model seabed was prepared by consolidating the homogenised clay in the Minidrum Centrifuge. The homogenised clay sample was filled in the model container in layers of 5 to 10 mm with a spatula, such that air entrainment was minimal. The initial depth of clay sample was chosen so that a final clay depth of about 65 mm will be available, after consolidation. Suitable drainage layers made of filter paper and geotextile were provided at the top and bottom of the clay specimen. In order to match the field undrained shear strength of 4 to 5 kPa, it was intended to use overburden/surcharge on the clay while consolidation. The overburden pressure required is estimated using the relation (Eq. 4) proposed by Wood (1990), where Λ = 0.7 to 0.9, and the over consolidation ratio (OCR) is ratio of vertical effective stresses between the overconsolidated and the normally consolidated ones. The value of $(S_u/\pm'_v)_{nc}$ is assumed to be 0.30 for soft marine clays. A overconsolidation pressure (surcharge) of 30 kPa was used to achieve an undrained shear strength of about 4 to 5 kPa at the mudline.

$$\left(\frac{S_u}{\pm'_v}\right)_{overconsolidated} = \left(\frac{S_u}{\pm'_v}\right)_{normally\ cconsolidated} \cdot OCR^{\Lambda} \qquad (4)$$

The clay sample was consolidated at 100 times acceleration due to earth's gravity, that is, 100g with a surcharge of about 30 kPa at the top. The clay sample was consolidated for about 7 hours in order to achieve 95% of primary consolidation considering double drainage and coefficient of consolidation equals 1.8 m^2/year. The consolidation process was monitored using a PPT embedded at the mid depth of the clay sample. The clay sample along with the surcharge was completely submerged under water during consolidation.

(b) Trench cutting and pipe burial

When the primary consolidation was fairly complete, a top layer of hard clay crust was scrapped and removed so that the final target depth of 65 mm (at model scale) with a slope of 1 in 30 on the mudline will be achieved. Then, a 'V' shaped trench was cut in the seabed such that the slope of the trench was 35° with the horizontal. The model pipe was then placed into the trench and was resting comfortably on the saddles. The trenched clay lumps of size about 25 mm were allowed to swell underwater for about 2 hours (at model scale) before backfilling. The swelled clay lumps were backfilled into the trench.

(c) Backfill consolidation and pipe pullout

The clay backfill was consolidated at 30 times earth's acceleration due to gravity, that is, 30g for 96 minutes (2 months at prototype scale) in the case of backfill without rock dump (Tests 1 & 2). The consolidation time for Tests 3 and 4, where the clay backfill was overlain by rock-dump, was split into two episodes of 48 minutes each. In the first 48 minutes, the back fill was consolidated at 30g without rock dump, followed by another 48 minutes of consolidation of backfill at the same g- level with rock dump on it. In order to prevent collapsing of the loose rock-dump material into the centrifuge during starting-up, the rock-dump was frozen under water and placed as a block on the clay backfill. The frozen block of rock dump melted down during the initial 10 minutes of the test, leaving a uniform layer of rock-dump on the clay backfill.

The pipe pullout testing was then started with a slow test at a speed of 0.002 mm/s for about 2 mm of vertical pipe displacement or until a steady-state resistance was reached. It was then followed by a fast test at a speed of 0.2 mm/s until the pipe came out of the back fill and rock-dump. The uplift resistance and the corresponding pipe displacement were recorded throughout the test. The excess pore pressure generated beneath the pipe and on the slope of the trench away from the pipe periphery was also recorded.

RESULTS

Uplift resistance

The results of four tests are presented in Fig. 5a, Fig 6a, Fig 7a and Fig 8a. Those figures show the uplift resistance and the excess pore pressure recorded at the pipe saddle level against the vertical pipe displacement. The uplift resistance versus pipe displacement plots are blown up and shown separately in Figs. 5b, 6b, 7b and 8b, to make the response during slow and fast rate of pullout clear.

Pore pressure response

PPT.1 located the pipe saddle level (below the pipe) measured around 1to 2 kPa suction during the slow pull out stage and around 3 to 4 kPa during fast pull out stage. This will results in an uplift resistance of 0.26 to 0.52 kN/m and 0.78-1.04 kN/m during slow and fast pull out stages. If cavitation occurs below the pipe then the uplift resistance will be smaller by the above mentioned values.

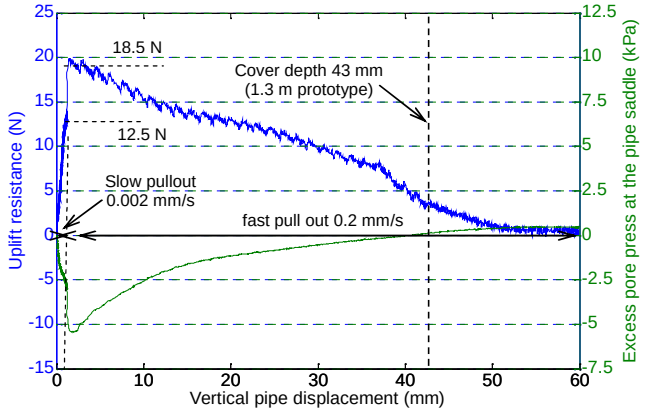

Figure 5a. Test 1, 43 mm clay backfill cover (model scale)

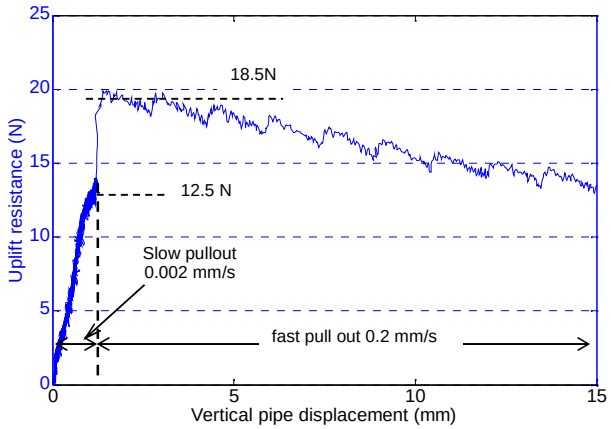

Figure 5b. Test 1, 43 mm clay backfill cover (model scale)

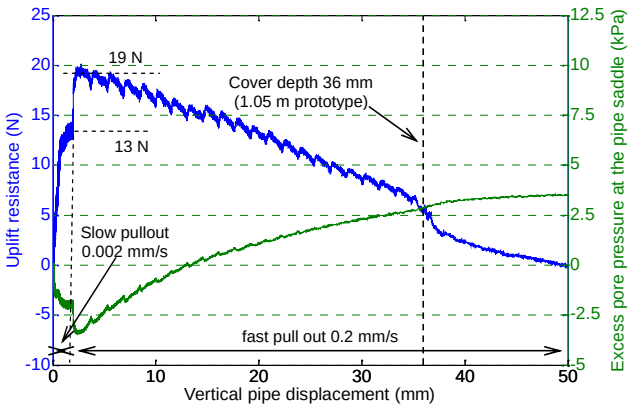

Figure 6a. Test 2, 36 mm clay backfill cover (Model scale)

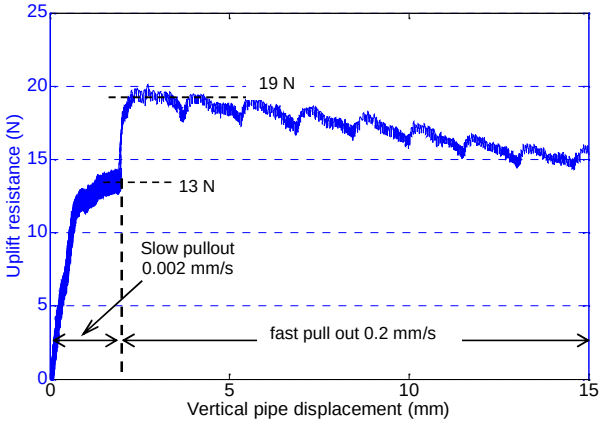

Figure 6b. Test 2, 36 mm clay backfill cover (Model scale)

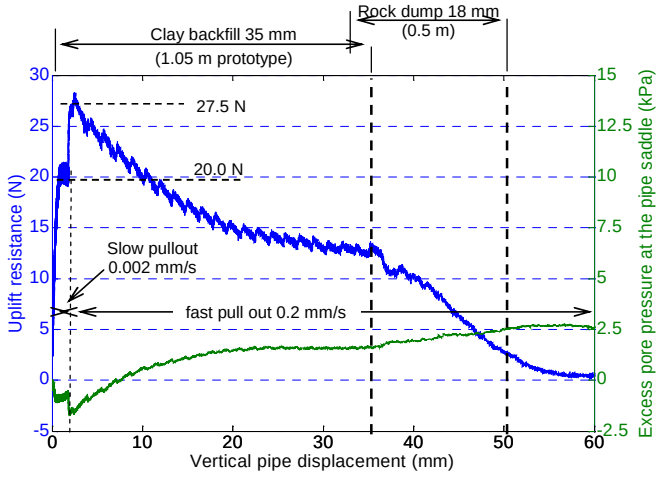

Figure 7a. Test 3, 36 mm clay backfill + 18 mm rock dump (model scale)

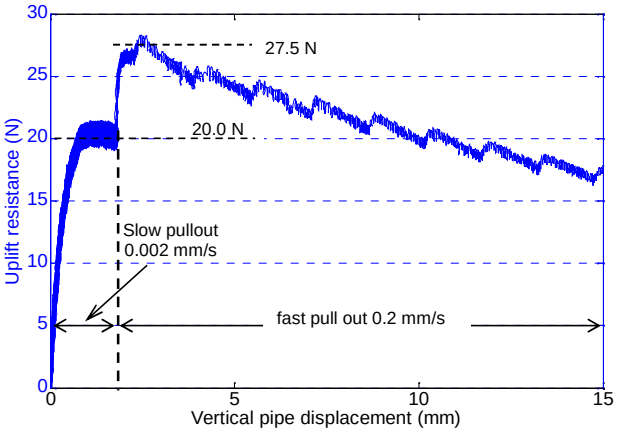

Figure7b. Test 3, 36 mm clay backfill + 18 mm rock dump (model scale)

DISUSSION

Slow pullout stage – Effective stress framework (drained behaviour assumed)

The uplift resistance obtained during the slow pullout stage can be interpreted in an effective stress frame work.
The equation (1) can be rewritten as bellow,

$$F = ¥'DH + 0.1¥'D^2 + ¥' f_p \left[H + \frac{D}{2} \right]^2 \quad (5)$$

The effect of rock dump can be incorporated as shown in equation (6) ,backfill and rock-dump as two layers as shown in Fig. 9.

$$F = ¥'DH_b + 0.1¥'D^2 + ¥' f_p \left[H_b + \frac{D}{2} \right]^2 + 2¥_r'H_r\, f_p H_b + ¥_r'DH_r + ¥_r'H_r^{\,2}\, f_{rp} \quad (6)$$

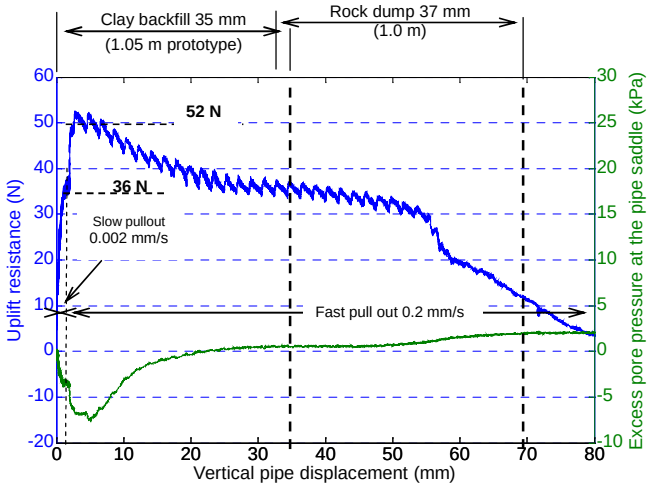

Figure 8a. Test 4, 36 mm clay backfill + 37 mm rock dump (model scale)

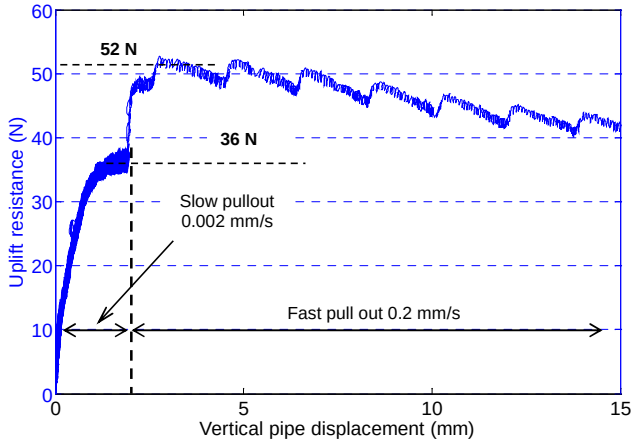

Figure 8b. Test 4, 36 mm clay backfill + 37 mm rock dump (model scale)

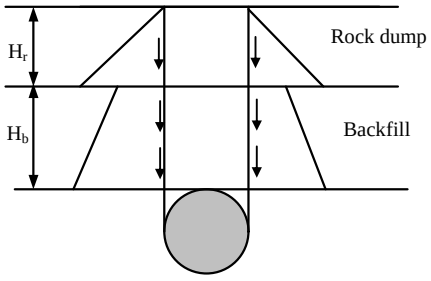

Figure 9. Rock dump and back fill as double layers

The varying contribution of the total shear resistance in the backfill and the weight of the backfill, weight of the rock dump and the shear resistance of the rock-dump on the uplift resistance is shown in Figure 10 (The plot was obtained using equation (6) and the parameters given in the caption in Fig. 10).

Figure 10 also shows the experiment data from Test 2, 3 & 4 with the prediction of uplift resistance using equation (6) and uplift factor of

0.25 for backfill clay. If the Pedersen model is to be used for predicting the uplift resistance, then only the components in the equation (6) that give shear resistance in the backfill and the weight of the backfill should be used as this is close to the experimental results. It is to be noted that, in order to predict the peak uplift resistance measured in Test 1, 3.13 kN/m, f of 0.1 needs to be used. This very low f factor suggests that the failure in Test 1 must have been a deep-seated failure.

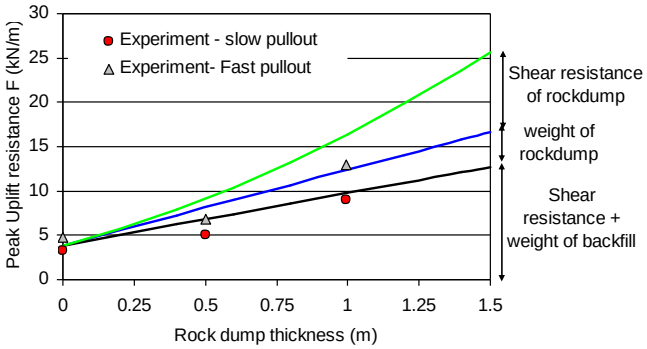

Figure 10. Variation of uplift resistance with rock-dump thickness ($\gamma' = 6.17$ kN/m^3, $\gamma'_r = 10$ kN/m^3, $H_b = 1.05$ m, $f_p = 0.25$, $f_{rp} = 0.4$, D = 0.261)

Fast pullout stage - Deep seated failure (undrained Flow around mechanism assumed)

The water content of the backfill was measured to be in the range (49%-58%) in all 4 tests. This water content range is close to the liquid limit of the clay (49%). Therefore, the shear strength of the back fill clay can be expected to be 1.7-2 kPa (Sharma and Bora, 2003).

The undrained uplift resistance for flow around failure can be predicted using the Eq. 7 (Randolph and Houlsby (1984)).

$$\frac{F}{s_u D} = 10.5 \qquad (7)$$

Table 2 shows the measured peak uplift resistance during the fast pullout stage and the back calculated s_u of the backfill using Eq. 7. It is clear that s_u of 1.7 kPa can predict the peak uplift resistance measured in Test 1 and Test 2 well.

Table 2. Summary or uplift resistance during fast pullout

	Test 1	Test 2	Test 3	Test 4
Peak uplift during fast pullout (kN/m)	4.63	4.75	6.88	13
Back calculated s_u from measured uplift (kPa)	1.69	1.73	2.51	4.74

CONCLUSIONS

A series of Minidrum Centrifuge tests were conducted at 30g, using a 8.7 mm diameter model pipe (261 mm at prototype scale) buried under soft clay backfill. The tests were designed to measure the uplift resistance experienced by a pipeline buried under a clay backfill after approximately 2 months.The model pipe was pulled out initially at a slower (0.002 mm/s) rate until a steady state maximum resistance is observed, and then at a faster rate (0.2 mm/s), whilst the uplift resistance and nearby excess pore water pressures were measured. Vibrocore samples of clay (s_u = 2 – 8 kPa) were obtained from the site, and reworked to simulate the debris created by the ploughing process.

The test results showed that the peak uplift resistance measured during slow pull out (0.002 mm/s) was 3.13 kN/m, 3.25 kN/m, 5 kN/m and 9 kN/m for Test 1, 2, 3 & 4 respectively. There is no established framework for predicting a drained (deep) flow around mechanism yet. Therefore, drained shallow failure mechanism (Eq. 6) was used to predict the slow uplift resistance in the tests. An uplift factor f of 0.25 for backfill can predict the peak uplift resistance measured in the slow pullout stage (0.002 mm/s) in Test 2, 3 & 4 reasonably well if the rock-dump weight and the shear resistance of the rock-dump components are not considered in Eq. 6.

The peak uplift resistance measured during fast pull out (0.2 mm/s) was 4.63 kN/m, 4.75 kN/m, 6.88 kN/m and 13 kN/m for Test 1, 2, 3 & 4 respectively. The peak uplift resistance in the fast pull out stage (0.2 mm/s), with flow around failure, could be predicted using $10.5 s_u D$ if the shear strength s_u of the backfill is known.

Further research is required fully to understand the effect of deep-seated failure and the rate effects.

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