Uplift Resistance of Buried Pipelines in Blocky Clay Backfill

J. Wang, S.K. Haigh & N.I. Thusyanthan[†]
Department of Engineering, University of Cambridge
Cambridge, Cambridgeshire, United Kingdom

[†]KW Ltd., Fetcham, Surrey, United Kingdom

ABSTRACT

This paper presents the results from 10 minidrum centrifuge tests conducted at the Schofield Centre, compiled with 4 additional test results from Thusyanthan et al., 2008. All these tests were designed to measure the uplift resistance of a pipeline installed into stiff clay by trenching and backfilling, then uplifted approximately 3 months after installation. All tests were conducted at 1:30 scale using soil obtained from offshore clay samples. Experimental results show that clay blocks remained intact after 3 prototype months of consolidation, and were lifted rather than sheared during pipe pullout. The uplift resistance therefore depends on the weight of the soil cover and the shearing resistance mobilised at the softening contact points between the intact blocks and within the interstitial slurry. Slow drained pullout led to lower resistance than fast pullout, indicating that the drained response is critical for design. The varying scatter shows that peak uplift resistance is very sensitive to the arrangement of the backfill blocks when the cover and pipe diameter are comparable to the block size.

KEY WORDS:

Pipeline, Uplift, Clay, Centrifuge, Backfill, Undrained, Drained

INTRODUCTION

Over the past few years, many minidrum centrifuge tests have been carried out in Cambridge to evaluate pipeline uplift resistance in soft clay. The purpose of these tests is to assist the design of pipelines against upheaval buckling. The installation of the pipeline is a trench-and-cover process, using the clay material excavated during the ploughing process as backfill. If insufficient uplift resistance is to be anticipated, the addition of rockdump would be a possibility. This process results in a blocky clay backfill which then consolidates under its own self-weight.

This paper presents the results of 10 minidrum centrifuge tests that were designed to simulate the conditions found offshore along a length of buried pipeline approximately 3 months after installation. This was the anticipated period between pipe-laying and start-up of the pipeline. These results were then compiled with the 4 additional test results, using the same test apparatus, from Thusyanthan et al., 2008, and presented in this paper.

All the 14 tests mentioned were conducted at 1:30 scale, using two model pipes of diameter 8.7 mm (261 mm at field scale) and 13 mm (390 mm at field scale) respectively, both buried under blocky clay backfill in an artificial model trench. This arrangement is used to simulate the ploughing and backfilling process. In each experiment, the selected model pipe was pulled out at both slow and fast rates, whilst the uplift resistance and nearby pore water pressures were measured.

The results of these tests have been interpreted to provide guidance for the selection of a design uplift response. The effects of pull-out rate, trench depth and rockdump surcharge have been investigated.

BACKGROUND TO PIPE UPLIFT RESISTANCE

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 either an effective stress or an undrained strength framework. To determine which framework is more applicable, the rate of loading should be compared with the rate at which water flows through the pore space of the soil. If the pipe is pulled out sufficiently quickly, there is not sufficient time for soil volume change to take place. The soil's tendency to dilate or contract during shear therefore gives rise to negative or positive pore water pressures respectively. The measured uplift resistance then corresponds to the undrained case.

Under drained conditions, the pull-out speed is low and the overlying soil is sheared slowly. There is sufficient time for seepage to take place, so that no excess pore pressure is generated. As a result, the soil changes in volume during shear. Below a critical speed – dependent on the ratio of the pipe velocity to the consolidation coefficient of the soil – fully drained conditions occur. Above a critical speed, fully undrained conditions occur. There is an intermediate range of partially-drained behaviour. Since the permeability of clay is extremely low, drained conditions are only achieved at very slow rates of pipe pullout in intact clay.

As upheaval buckling is an instability-driven process, the rate at which the soil is sheared cannot be derived from the speed at which the pipeline is heated. Whilst buckling is in general considered to be a rapid process, if the resistance that is generated during slow (drained) movement is less than that during rapid undrained movement, then there is the possibility of a slow initiation of buckling until the undrained resistance is insufficient to maintain stability, after which time a rapid undrained buckle will be formed.

Additional Complications due to Blocky Clay Backfill

When considering intact soil, the drained and undrained resistances form bounds on the resistance that can be generated. If a lower bound on resistance provides a conservative value for design, the minimum of these two values can be used in design and the other extreme ignored. However, if the pipe is buried in a trench backfilled by stiff clay, it will lie under blocks of clay with slurry and water filling the large voids between. This complication means that true 'undrained' conditions are difficult to define. The large voids between the clay blocks have very high permeability, whilst the intact clay blocks have very low permeability. The voids between the blocks are too large to generate excess pore pressure during practical velocities of pipe pullout. However, the intact blocks will be undrained at fast pullout rates.

In this case, the resistance to pullout comes both from the resistance at the contact points between the blocks, and from within the slurry material between the blocks. This resistance depends not on the intact soil strength, but on the submerged weight of the blocks and the slurry. This weight creates the contact forces between the blocks and drives consolidation of the slurry. Similarly, if the pipe is buried in sand, the behaviour is drained for most practical pull-out rates, and so an effective stress analysis is appropriate.

It is therefore difficult to identify the fully undrained resistance for a blocky backfill, but the drained resistance can be achieved at sufficiently slow speeds. Drained conditions can be checked by measuring pore pressures, which should remain hydrostatic. If the undrained resistance exceeds the drained resistance (which is usually the case for stiff materials such as over-consolidated clay that dilate on shearing, or generate negative pore pressure in undrained shear) then the drained resistance provides a conservative estimate of the lowest available resistance.

Heave Mechanism

The conventional interpretation of pipe uplift resistance involves vertical sliding planes above the pipe, with the geometry and nomenclature as shown in Fig. 1. The resulting resistance comprises the overburden weight of the rectangular soil block ($W = \gamma' HD$) and from shear stress ($\tau = \sigma'h \tan \varphi = K\sigma'v \tan \varphi = K\gamma' z \tan \varphi$) on the vertical slip planes. It is conventional to define the cover depth, H, from the ground surface to the pipe crown, although some publications define H to the waist of the pipe.

The uplift resistance is hence expressed as the sum of the overburden weight and the shearing resistance. Using an effective stress framework, the Trautmann model gives (White et al., 2001):

$$\frac{F}{\gamma' HD} = 1 + \left(\frac{H}{D}\right) K \tan\phi = 1 + f_t\left(\frac{H}{D}\right) \tag{1}$$

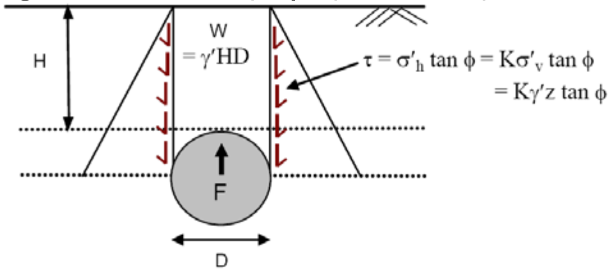


Fig. 1. Vertical shear model for pipe uplift resistance

A variation to this is the Pederson Model (Equation (2), Cathie et al., 2005, DNV RP F110), where the whole volume of the soil above the pipe is included:

$$\frac{F}{\gamma' HD} = 1 + 0.1\left(\frac{H}{D}\right) + f_p\left(\frac{H}{D}\right)\left(1 + \frac{D}{2H}\right)^2 \tag{2}$$

White et al., (2001) proposed a sliding block mechanism with inclined shear planes. Rather than assuming a stress state equivalent to the "at rest" conditions, the authors proposed that the normal effective stress would remain constant during the uplift event. The expression for non-dimensional F, based on this model, is indistinguishable from that of Equation (2), except that f_p is expressed in terms of fundamental geotechnical properties: angle of dilation, ψ, peak shear resistance angle, $\square_{peak}$, and earth pressure coefficient at rest, K_0.

For undrained behaviour, the equivalent vertical slip model leads to equation (3) (Cathie et al., 2005).

$$\frac{F}{\gamma' HD} = 1 + \frac{2s_u}{\gamma' D} \tag{3}$$

This paper concentrates on the Pederson Model, and the associated 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. It could be assumed that the cover depth during pullout is equal to $H - \delta$, where δ is the pipe displacement. However, since heave is observed at the ground surface, this assumption is an approximation that underestimates the actual cover. Peak uplift resistance is usually reached at a pipe displacement of less than 10% of the cover depth, justifying the assumption of constant H.

Deep Mechanism

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 (Fig. 1), due to the increased 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 heave) is typically $H/D \geq 4$ for loose backfill (vanden Berghe et al. 2005, White et al. 2001, Schupp et al. 2006). The undrained uplift resistance for flow around failure can be predicted using Equation (4) (Randolph & Houlsby, 1984).

$$\frac{F}{s_u D} = 10.5 \quad (4)$$

BACKGROUND TO TEST FACILITY

The Schofield Centre

The Schofield Centre (SC) is the main laboratory of the Geotechnical Research Group at Cambridge University Engineering Department. The SC hosts over 40 staff and researchers who are involved in geotechnical physical modelling and has an annual turnover in excess of £1M. The 10 m diameter Turner beam centrifuge and the 0.74 m diameter Schofield Minidrum Centrifuge allow geotechnical processes to be modelled at small scale, whilst maintaining similitude with prototype structures.

Minidrum centrifuge

Centrifuge modelling has been extensively used in the study of soil-structure interaction. It works on the principle that the soil behaviour in a small scale model can be made to be identical to that in a full scale prototype if the stress conditions can be made homologous to those of the prototype. The increase in weight required for the scale model to have the same stress state as that of the prototype is accomplished through increased acceleration of the scale model due to centrifugal acceleration. The strain response of the soil should then also be similar, if rate effects in the small model can be allowed for, and if grain size effects do not intrude.

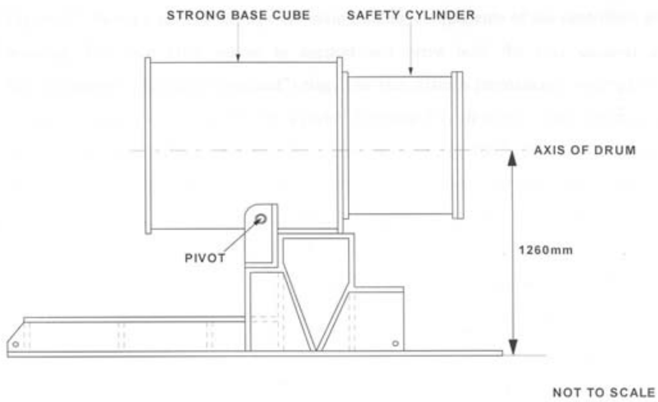
The fundamental scaling law in centrifuge modelling is to ensure the same effective stress state between the model and the prototype. If the same soil is used in the model as in the prototype, and provided that the model has been subject to the same stress history as the prototype, then for the centrifuge model subjected to an inertial acceleration of N times the Earth's gravity, g, the vertical stress at depth Z_m will be identical to that in the corresponding prototype at depth Z_p where $Z_p = N \times Z_m$. Therefore, the model can and has to be set up at 1/N scale of the prototype in every dimension.

Fig. 2 portrays the Minidrum Centrifuge setup at the Schofield Centre. Fig. 3 shows an elevation and cross section through the vertical axis of the Minidrum Centrifuge. The base of the ring channel has a radius of 370 mm measured from the central shaft. It can reach 471g when the centrifuge is spinning at a maximum speed of 1067 rpm, where g is the acceleration due to gravity (i.e. 9.81 m/s^2). An on-board PC provides 16 channel of data acquisition at up to 10 kHz. Miniature video cameras can be used to monitor the progress of an experiment, and actuators are available to provide radial or circumferential movement.

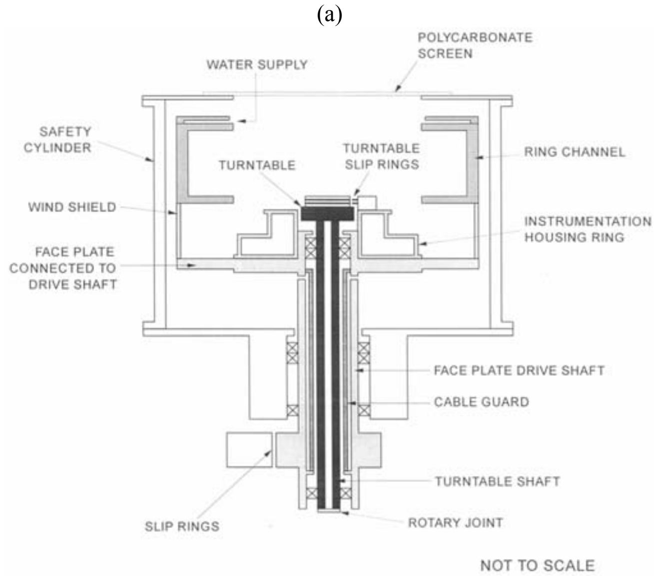
The centrifuge has a central pivot which allows a 90º rotation of the channel axis from the vertical to the horizontal position. This permits a model package to be prepared in a convenient position inside the channel before spinning, and then switched to the vertical position for spinning. It should be noted that the centrifuge has to operate with its axis in the vertical position during a test. This ensures that the 1g component due to the normal gravity is uniformly acting on one side of the model. This 1g component is taken into account by setting up the model on a 1in N slope, where the test will be carried out at N times earth's gravity, so that the resultant acceleration will be perpendicular to the model surface.

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 features of the Minidrum Centrifuge are presented in more detail by Barker (1998).

Fig. 2. The Minidrum Centrifuge setup



(a)



(b)

Fig. 3. The Schofield Minidrum Centrifuge in (a) elevation and (b)

cross-section

EXPERIMENTAL METHODOLOGY

Centrifuge Model

The general arrangement of the centrifuge model package is shown in Fig 4. The uplift movement is provided by an actuator mounted on the central turntable of the centrifuge. The actuator is connected to the pipe by two wires, via load cells at the top of each cable (Fig. 6). A displacement transducer mounted on the actuator is used to measure the pipe movement. Extension of the pulling wires is negligible. The instruments and control system were calibrated before and after the first test, showing no change in response. All instruments returned to their initial readings after each test.

The vertical pipe uplift movement was triggered by a radial 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. 4). The model pipe is connected to the load cell through nylon coated stainless steel fishing lines of 0.6 mm diameter and has a safe working load of 50 kg. These thin lines minimise, to a large extent, the disturbance caused to the clay backfill or rock dump. A displacement transducer was mounted on the actuator to measure the vertical displacement. The base of the model container is made from aluminium sheet formed into a symmetrical trench with 30° sideslopes. Two model pipes are used, of 8.7 mm and 13 mm in diameter respectively, both 120 mm in length. The pipe is supported at each end on a cradle during the model preparation and consolidation phases of the test. When resting on the cradle, the invert of the pipe is located 0.5 pipe diameters above the based of the trench. The trench has a length of 15 mm, so the pipe is not fitted tight against the ends. The length – diameter ratio of the pipe (120 mm / 10 mm) is sufficiently high for end effects to be eliminated.

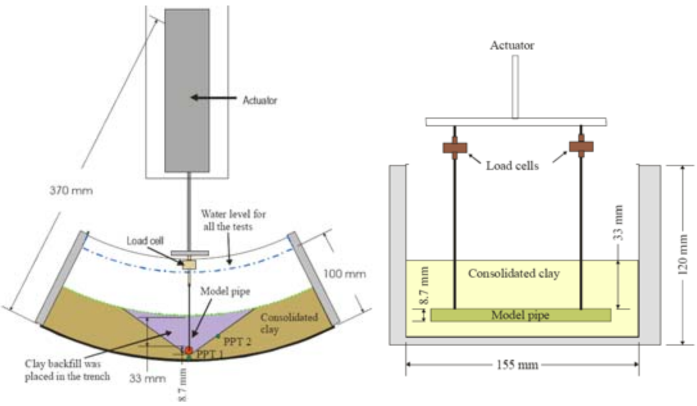


Fig. 4. Schematic setup of the centrifuge model with 8.7 mm diameter pipe in (a) side view and (b) cross-section

Both the actuator and the model pipe are oriented on a 1:30 slope within the test chamber, in order that the resultant self-weight due to the Earth's gravity and the centrifugal acceleration acts normal to the pipe and parallel to the direction of pulling.

Pore pressure transducers (PPTs) are mounted within the test container, so that pore pressures above, below and distant from the model pipe are monitored throughout the experiment.

The support wires connecting the actuator to the pipe are initially loose, and the pipe weight is supported by a cradle. The support wires then tighten as pullout begins and the package is carefully assembled to ensure that both wires become taut concurrently, so that pullout occurs exactly perpendicular to the pipe axis.

Testing Programme

An initial test was conducted on the empty container with the pipe submerged in water. This test allowed the submerged weight of the pipe and pulling wires to be assessed. This force was subtracted from the measured pull out resistance in subsequent tests in order to provide the uplift resistance (i.e. purely the resistance due to the soil).

Fig. 5. Gravel for rock dump simulation (< 4 mm)

Table 1. Test programme, all quoted values are at prototype scale

Test No.	Pipe Diameter D (mm)	Cover H (m)	H/D ratio	Rock dump thickness (m)	Block size (mm)
1	390	0.5	1.282	0	300
2	390	0.5	1.282	0	150
3	390	0.5	1.282	0	300
4	390	0.5	1.282	0	300
5	390	1.0	2.564	0	300
6	390	1.0	2.564	0	300
7	390	1.0	2.564	0	300
8	390	0.5	1.282	0	150
9	390	0.5	1.282	0	300
10	390	0.75	1.923	0.5	100
11	261	1.3	4.981	0	100
12	261	1.05	4.023	0	100
13	261	1.05	4.023	0.5	100
14	261	1.05	4.023	1	100

A total of 14 tests were performed. The first 10 tests used the 13 mm diameter model pipe, whereas the last 4 tests employed the 8.7 mm diameter model pipe. All the clay samples used were of offshore origin, and had been tested for undrained shear strength, s_u, and moisture content. s_u values vary between 40-55 kPa for the clay sample used in Tests 1 to 6, and between 35-45 kPa for the clay sample used in Tests 7 to 10. The s_u values for samples used in Tests 11, 12, 13 and 14 are 3.0 kPa, 3.6 kPa, 3.4 kPa and 4.0 kPa respectively. The clay samples were then reconstituted separately with saline water and homogenised to be used in the test. Tests 10, 13 and 14 involved simulation of rock dump over the clay backfill. Angular and rounded aggregates sieved through 4 mm sieve were used for this purpose (Fig. 5). The size of the prototype rock-dump material was informed to be about 100 mm. The main test programme is summarised in Table 1.

Model preparation and test procedure

The basic preparation and test procedure for centrifuge modelling of pipe uplift is as follows:

1. The pipe is placed on the support cradle and connected to the actuator.
2. Blocks of backfill material are placed at random orientations around the pipe, until the average level of the blocky material lies slightly above the intended level of cover. This additional cover depth is lost during the consolidation phase.
3. The centrifuge axis is rotated from horizontal to vertical. The stickiness of the clay blocks is sufficient to hold the backfill in place.
4. The centrifuge is accelerated to 10 g.
5. Water is added to the base of the centrifuge ring channel, and fills the trench from either side (Fig. 4).
6. The centrifuge acceleration is increased to 30 g.
7. The model is permitted to consolidate for a period of 3 prototype months (150 minutes model scale).
8. The pipe is pulled out at a slow rate (0.002 mm/s model scale) until the peak uplift resistance is identified. The pull out rate is then increased to 0.2 mm/s, to identify whether the undrained resistance is higher. The pipe is pulled out until above the original soil surface.

TEST RESULTS

Characteristic Response

The characteristic response during a typical test is shown in Fig. 6 (a), with the representative data for Test 3 shown in Fig. 6 (b). Fig. 7 shows photographs at 4 stages during Test 3. Each test showed similar behaviour as follows. During the slow stage an almost linear initial response until 'yield' (A) was followed by a gentle plateau as the peak drained resistance was reached (B). After this peak value was identified, the pullout speed was increased and a higher peak value of uplift resistance was identified for this fast stage (C). As the pipe was pulled out further, the uplift resistance reduced as the cover depth decreased. Since the ground surface above the pipe heaved as the pipe was lifted, significant uplift resistance remained after a pipe displacement equal to the initial cover depth (D). Occasionally a sudden reduction in uplift resistance is observed, associated with blocks of clay falling away from the pipe crown (E). Even after pulling the pipe free from the soil, a positive value of uplift resistance usually remained, indicating the weight of soil resting on the pipe crown (F).

The surface of the blocks softened, but the strength remained high when assessed post-test. Observation of the backfill post-testing indicated that the blocks generally moved aside as the pipe was pulled out, rather than being sheared. Therefore, the resistance is governed by (i) the shear resistance in the slurry and at the contact points between the blocks and (ii) the weight of the backfill blocks lifted or pushed aside. Table 2 summarises the key results from all 14 tests.

Submerged Unit Weight of Backfill-Slurry-Water Mixture

In order to back-analyse the measured uplift resistance, it is necessary to calculate the submerged unit weight of the backfill-slurry-water mixture. This unit weight is not the submerged weight of the blocks, but a lower value reflecting the lower density of the large water/slurry-filled voids between the blocks. The submerged unit weight of the backfill mixture was calculated using the following procedure:

1. The mass of clay blocks used in each test was measured during model preparation (this value was constant for each cover depth).
2. The submerged unit weight of the clay blocks was known
3. From 1 and 2 the volume of clay blocks was known.
4. The volume of trench occupied by backfill was calculated from the trench geometry (minus the volume of the pipe).
5. From 3 and 4, the volume of free water added during saturation of the trench was known, and converted into a mass.
6. Summing the mass of water from 5 and the mass of clay from 1, and dividing by the trench volume in 4 gave the total unit weight of the backfill mixture, from which the submerged unit weight was found by subtracting 9.81 kN/m3.

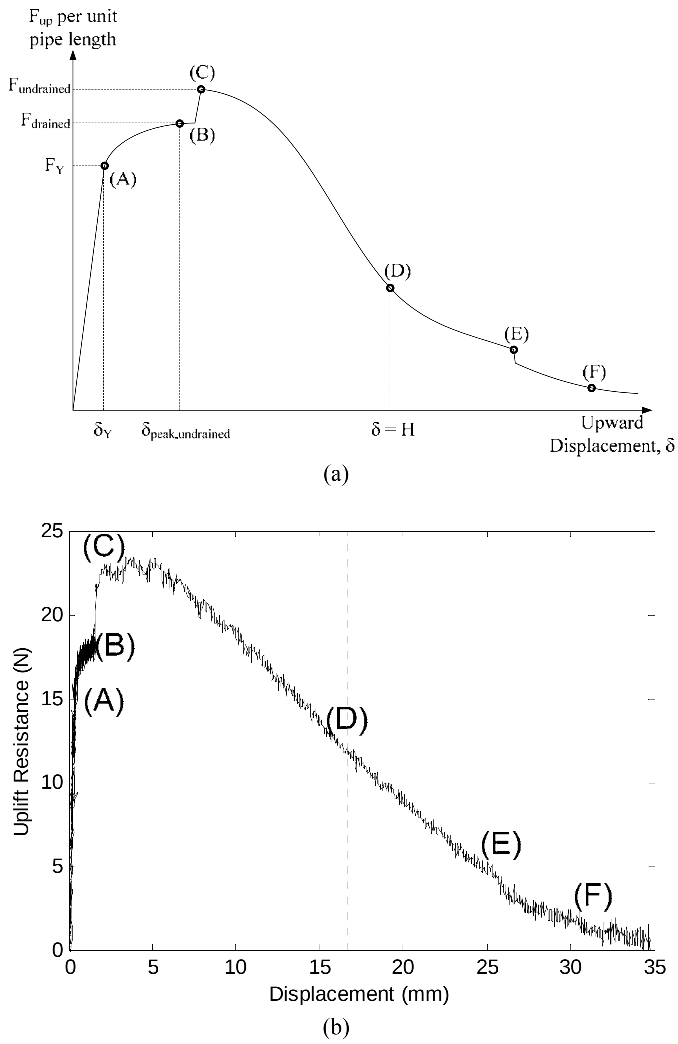


Fig. 6. Uplift response in (a) characteristic; (b) representative data from Test No. 3

The submerged unit weight for the saturated backfill mixtures were in the range 6-8 kN/m^3 (Table 2) compared to a value of 10 kN/m^3 for the intact clay in the backfill blocks. These values are multiplied by the centrifuge acceleration level to give the unit weight applicable in Equation 2 (when using model scale units for dimensions). The above procedure was modified as necessary for tests involving rock dump.

It should be noted that if the in situ submerged unit weight is used to convert uplift resistance to an uplift factor, negative values can result, since the overburden contribution to uplift resistance is over-predicted. Similarly, an uplift factor derived using the submerged weight of the backfill mixture should not be combined with the in situ submerged unit weight: to do so would lead to an over-prediction of the uplift resistance.

Table 2. Summary of test results in prototype scale

Test	γ' (kN/m^3)	Peak Uplift Resistance F (kN/m)		
		Slow, approx. drained	Fast, approx. undrained	Diff. (%)
1	6.68	1.60 (f_p = 0.04)	2.63 (f_p = 0.36)	64.1 (900 for f_p)
2	6.68	4.38 (f_p = 0.90)	5.38 (f_p = 1.21)	22.9 (34.4 for f_p)
3	6.68	4.50 (f_p = 0.94)	5.75 (f_p = 1.32)	27.8 (40.4 for f_p)
4	6.68	4.38 (f_p = 0.90)	6.05 (f_p = 1.41)	38.3 (57.8 for f_p)
5	6.17	6.38 (f_p = 0.38)	9.63 (f_p = 0.75)	51.0 (97.4 for f_p)
6	7.18	7.50	9.00	20.0
7	6.17	5.50 (f_p = 0.28)	8.25 (f_p = 0.59)	50.0 (111 for f_p)
8	6.68	4.00 (f_p = 0.78)	5.25 (f_p = 1.17)	31.3 (50.0 for f_p)
9	6.68	3.00 (f_p = 0.47)	4.00 (f_p = 0.78)	33.3 (66.0 for f_p)
10	7.99	11.3	16.0	42.2
11	6.30	3.13 (f_p = 0.00)	4.63 (f_p = 0.12)	48.0 (N.A. for f_p)
12	6.36	3.25 (f_p = 0.10)	4.75 (f_p = 0.28)	46.2 (280 for f_p)
13	6.99	5.00	6.88	37.5
14	6.99	9.00	13.0	44.4

Note: 1. γ' in the Table 2 is the submerged unit weight of the backfill material, i.e. mixture of clay-slurry-water.
2. Back-calculated values for f_p, the uplift factor from the Pederson Model, are listed for tests not involving rock dump
3. The backfill used in Test 6 was clay-sand mixture with 35% sand by mass
3. The clay backfill in Tests 11 & 12 was subject to 96 minutes of consolidation at model scale, i.e. 2 months at prototype scale.
3. The clay backfill in Tests 13 & 14 underwent two consolidation stages: stage 1, consolidation of clay backfill alone at 30g for 48 minutes (1 month); and stage 2, consolidation of clay backfill overlain by rock-dump at 30g for 48 minutes (1 month).
4. More details on Tests 11 to 14 can be found in Thusyanthan et al., 2008

Undrained versus Drained Peak

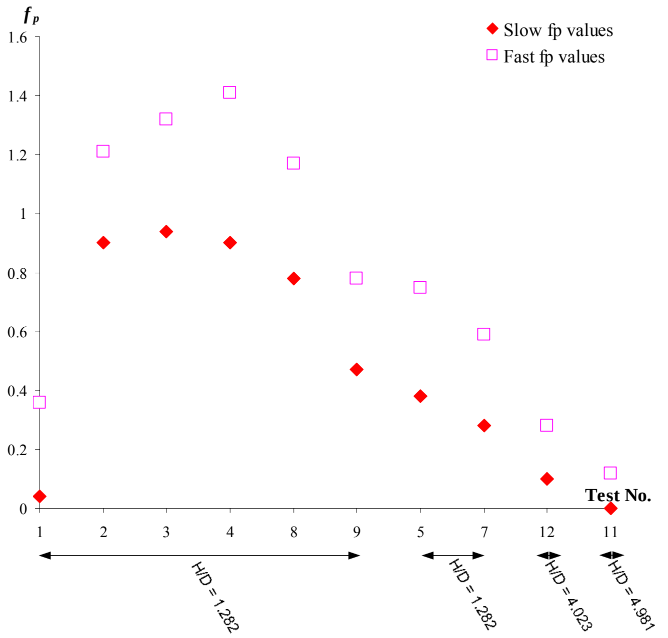
The observed peak uplift resistance during the slow pull-out stage is consistently smaller than that during the fast stage. The discrepancies between the two range from 20% to over 60%. This is consistent with PPT readings during the test. No excess pore pressure was generated during the slow pull-out stages, indicating drained conditions. The initial slow stage therefore indicates the peak drained resistance. During fast pullout, significant negative pore pressures were measured beneath the pipe and in the backfill during periods in which the pipe was moving upwards. These negative pore pressures will add significantly to the resistance measured under undrained conditions. This suction is a transient event which will be dissipated by seepage through the clay and can thus not be relied on to prevent pipe buckling.

Fig. 7. Photographs of Test 3 with 0.5 m prototype cover depth, large clay blocks

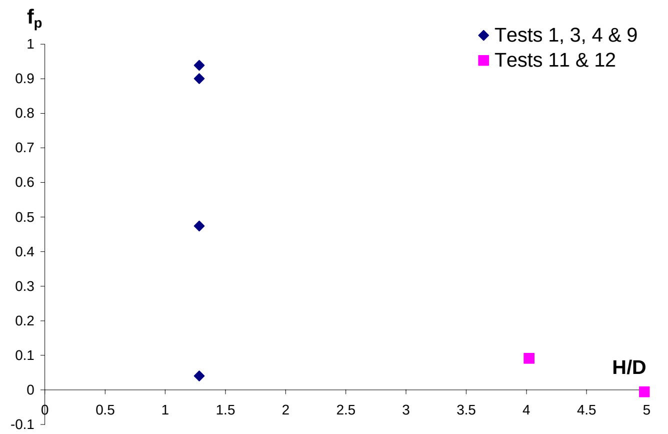
Pederson Model Back Analysis

The peak drained uplift resistance data of Tests 1, 3, 4 & 9, and Tests 11 & 12, can be back-analysed using Equation 2, so that the Pederson uplift resistance factor, f_p, can be calculated, as shown in Fig. 8. The following observations are made based on these results:

1. Large scatter is evident at H/D ratio of 1.28, with f_p values ranging from 0.04 to 0.94. Data shows more consistency at a higher H/D ratio of 2.56. However, there is not a clear relationship linking fp with H/D.
2. The clay blocks were placed randomly around the pipe. It is important to note that the clay block size (10 mm edge, or 17 mm corner-to-corner) is comparable to, or greater than, both the pipe diameter (13 mm and 8.7 mm) and the cover depth.
3. The highest measured uplift resistance was 25.5 N. Ignoring the weight of the overlying soil, this could be equated to a shear stress of 25.5 N / (17 mm × 120 mm × 2) = 6 kPa acting on vertical slip planes either side of the pipe. This strength is far lower than the intact strength of the soil, which confirms that the shear resistance during uplift is governed not by the intact material strength but by the mobilised strength at the contact points between blocks and within the slurry.
4. From these six base case tests, it is concluded that for such large blocks of stiff backfill with shallow cover, the uplift resistance is significantly dependent on how the blocks are oriented on top of the pipe: it may be possible for the pipe to be lifted through the blocks with minimal disturbance, whilst being loaded by only minimal block weight, or the blocks may be interlocked in a manner that requires additional force for the pipe to move.
5. For a continuous pipeline (instead of the 10 D length used in these tests), the influence of the block orientation will be smeared out along the pipe length. However, without further consideration, use of the average uplift factor cannot be justified for design.



(a)



(b)

Fig. 8. Pederson uplift resistance factor back-analysis against: (a) Test No., and (b) selected H/D ratios

Deep Flow-round Mechanism

For Tests 11 & 12, the f_p value is particularly low (0.0 and 0.1 respectively). This is probably a result of the relatively high H/D ratios (4.98 and 4.0 respectively). Therefore, the deep flow-round mechanism offers lower resistance than the heave mechanism assumed in the Pederson model.

The water content of the backfill in both tests was measured to be in the range (49%-58%). 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). Therefore, the applicability of the deep flow-round can be checked by back-calculating the s_u values using Equation (4) and the undrained data for Tests 11 & 12, and comparing them with this 1.7-2 kPa range. The results is summarised in Table 3 below. It is clear that s_u values of 1.7 kPa can predict the peak uplift resistance measured in both tests well. Therefore, it is reasonable to conclude that the deep flow-round mechanism dominates the response. Results from Tests 13 & 14 lead to similar conclusions. More detailed results and analysis on these 4 tests can be found in Thusyanthan et al., 2008.

Table 3. Back-calculated s_u values for fast pull-out in Tests 11 & 12

Tests No.	11	12
Measured $F_{peak, undrained}$ (kN/m)	4.63	4.75
Back-calculated s_u (kPa)	1.69	1.73

Design Implication and Future Work

From a designer's point of view, the slow and fast peak pull-out resistances can be regarded as the lower and upper bounds on the actual available soil resistance during the uplift event. However, the in-situ orientation of the clay blocks significantly affects the available shear resistance, and the range of the f_p value can be as wide as from 0 to 1. It will be over-conservative to forego all the shear component, while the use of the average f_p value cannot be justified without a significant database. In addition, more tests are also necessary to mark the dividing line between the heave mechanism and the deep flow-round mechanism, and to suggest appropriate analytical models for the latter. Based on the results from Test 11, it could be unconservative to use the Pederson model at H/D ratios greater than 5, as the f_p value drops just below zero, suggesting that the deep flow-round mechanism starts to take over.

The authors have planned additional tests, over the next three years, to resolve these issues. Both additional centrifuge tests and full-scale tests will be carried out. The aims will be:

1. To assess the reliability of the shear component during uplift in blocky clay backfill, and to suggest appropriate bounds for use in design.
2. To match the results from centrifuge modelling with those from full scale tests, so as to derive appropriate scaling laws on peak uplift resistance and mobilisation distance.
3. To investigate how backfill arrangement affects f_p values, and ways to control it.

CONCLUSIONS

This paper presents the results from 14 minidrum centrifuge tests conducted at the Schofield Centre. The tests were designed to measure the uplift resistance of a pipeline installed into stiff clay by trenching and backfilling, then uplifted approximately 3 months after installation. By using the small-diameter minidrum centrifuge, 14 model tests were conducted using soil obtained from offshore.

The tests were conducted at 1:30 scale, using two model pipes of diameter 8.7 mm (261 mm model scale) and 13 mm (390 mm model scale) respectively, buried under blocky backfill in a pre-formed trench. The model pipe was pulled out at both slow and fast rates, whilst the uplift resistance and nearby pore water pressures were measured. Offshore samples of stiff clay were obtained from North Sea sites, and cut into cubic blocks to simulate the debris created by the ploughing process.

The blocks remained intact after 3 prototype months of consolidation, and were lifted rather than sheared during pipe pullout. The uplift resistance therefore depends not on the intact clay strength, but on the weight of the soil cover and the shearing resistance mobilised at the softening contact points between the intact blocks and within the interstitial slurry.

Slow drained pullout led to significantly lower resistance than fast undrained pullout, indicating that the drained response is critical for design. Despite the fact that buckling is normally considered to be a rapid event, the lower drained resistance to pipe movement will imply that the initiation of instability may be a slow drained process, leading to rapid buckling when the undrained resistance of the displaced pipe is no longer sufficient to maintain stability.

The measured uplift resistance was interpreted within an effective stress framework to deduce values of uplift factor, f_p, in order to characterise the variation in uplift resistance with cover depth. It was necessary to assess the overall density of the backfill mixture to account for the large voids between the backfill blocks. Use of the in situ submerged unit weight would over-predict the surcharge acting on the pipe and lead to unrealistic negative values of uplift factor. The submerged unit weight of the backfill mixture was in the range 6-8 kN/m^3 compared with 10 kN/m^3 for the intact clay.

For shallow cover with large backfill blocks, f_p was higher on average, but significant scatter was observed between 4 tests. Less scatter was observed in 2 tests using smaller blocks. The varying scatter shows that peak uplift resistance is very sensitive to the arrangement of the backfill blocks when the cover and pipe diameter are comparable to the block size.

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