UPLIFT RESISTANCE OF BURIED PIPELINES IN SAND

EDWARD A. DICKIN

ABSTRACT
A centrifugal model study into the influence of pipe diameter, depth of burial and backfill density on the resistance to uplift of pipes in sand is reported. No significant differences between the behaviour of buried pipes and that of strip anchors is found, justifying the application of anchor uplift theory to buried pipeline behaviour. Several anchor-based theories overestimate the stability of the model pipes tested, although predictions from the theories of Rowe and Davis (1982), Vermeer and Sutjiadi (1983) and Vesci (1971) yield reasonable agreement for models in dense sand, while the simple Vertical Slip Surface Model is alone in providing sensible predictions for pipes in loose sand. The influence of pipe surface roughness, although significant for pipes at shallow depth in dense sand, is minimal at deeper embedments.

Key words: centrifuge, design methods, pipeline, sand, uplift (IGC: H8)

INTRODUCTION
The behaviour of buried pipelines in response to large differential ground movements arising from offshore slope failures, earthquake induced faulting, urban excavations and landslides has received recent attention by Trautmann et al. (1985). Under such circumstances a detailed knowledge of the soil forces resulting from relative soil/pipeline displacements is a design pre-requisite. The design approach most commonly adopted is to apply theories giving the uplift capacity of strip anchors to the geometrically similar pipeline problem. Trautmann et al. (1985) reported agreement between predictions based on the theories of Vesci (1971) and Rowe and Davis (1982) and unit gravity tests on 100 mm diameter pipes in medium and dense sand, but theories generally overpredicted the forces on pipes in loose sand. Although comparisons with experimental work on strip anchors by previous authors were given to support the underlying assumption that pipes and anchors behave in essentially similar fashion, inevitable differences in sand characteristics, testing procedures and conditions introduce some uncertainty into the approach. The parallel centrifugal testing programs on pipes and anchors in this research provide a direct comparison by which the assumption can be further examined. The majority of tests were performed on 25 mm diameter pipes and 25 mm wide strip anchors accelerated to 40 gravities. The tests therefore simulated the stresses surrounding 1 m field scale structures. A few tests were also carried out to examine the influence of pipe diameter and surface roughness.

THEORETICAL CONSIDERATIONS
A considerable amount of research has been directed towards the pull-out resistance of embedded anchors in cohesionless soils. Most theoretical analyses assume some failure surface extending through the soil above the anchor. The simplest of these, reported by Matyas and Davis (1983) and attributed to Majer (1955), is to assume a vertical slip surface extending above the anchor. The geometry in the case of a buried pipe is shown in Fig. 1(a). The uplift resistance per unit length, \( F_u \), derived from the weight of soil above the pipe, the weight of soil displaced by the upper half of the pipe and the shearing resistance along the vertical slip surfaces, is given by the expression:

\[
F_u = \left(1 - \frac{\pi D}{8H} + K \tan \phi_{ps} \frac{H}{D}\right) \gamma' HD
\]

(1)

in which \( \gamma' \) = effective soil unit weight, \( H \) = depth to centre of pipe, \( D \) = pipe diameter, \( \phi_{ps} \) = angle of soil friction in plane strain and \( K \) = lateral earth pressure coefficient. The value of \( K \) is often taken as \( K_0 \), the at rest coefficient for loose sand, but its value in the case of dense sand is difficult to assess and can often be greater than unity. Trautmann et al. (1985) found this theory gave good agreement with their experimental data with \( K \) values of 0.5, 0.65 and 0.75 for pipes in loose, medium and dense sand respectively. However rupture surfaces above strip anchors are generally curved in broad agreement with the

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19 Lecturer, Department of Civil Engineering, University of Liverpool, England, UK.
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pyramidal shaped geometry analysed by Meyerhof and Adams (1968) shown in Fig. 1(b). For shallow embeddings Meyerhof and Adams (1968) neglected the second term in Eq. (1) and assumed \( K = 0.95 \), while at greater depths the following expression was proposed;

\[
F_v = \left(1 + \frac{2H}{D} - \frac{H_e}{D} \right) \frac{H_e}{H} K \tan \phi_{ps} \gamma' HD
\]

(2)

where \( H_e \) = vertical extent of the rupture surface and depends on \( \phi_{ps} \) and \( D \). More recently Murray and Geddes (1987) derived an upper bound solution from a limit analysis approach which supported the use of Eq. (1) with a \( K \) value of unity. The same equation was derived by Saran et al. (1986), for anchors with \( H/B \leq 3 \). Frydman and Shaham (1989) also found that this simple equation, with \( \phi \) values of 30° and 45° for loose and dense soil respectively, gave excellent agreement with observation in several experimental studies. Murray and Geddes (1987) also proposed an alternative expression from an equilibrium approach;

\[
F_v = (1 + (H/D)(\sin \phi_{ps} + \sin(\phi_{ps}/2))) \gamma' HD
\]

(3)

while Vermeer and Sutjiadi (1985) proposed Eq. (4) from analyses involving straight rupture surfaces inclined at the dilatancy angle \( \psi \) to the vertical;

\[
F_v = (1 + (H/D) \tan \phi_{ps} \cos \phi_{ps}) \gamma' HD
\]

(4)

Vesic (1971) investigated the pressure required to expand a cylindrical cavity to the soil surface and produced uplift coefficients which increased with embedment and soil friction angle, while a design method involving the use of design charts based on finite element analyses of strip anchors was published by Rowe and Davis (1982). The uplift resistance is given by;

\[
F_v = (F_v \cdot R_v \cdot R_e \cdot R_k) \gamma' HD
\]

(5)

in which \( F_v \) = the basic uplift capacity for a smooth anchor in a non-dilatant soil with \( K_0 = 1 \), and \( R_v \), \( R_e \) and \( R_k \) are respective correction factors for soil dilatancy, anchor roughness and initial stress state. Factors \( R_e \) and \( R_k \) were found to have little influence on uplift capacity and may therefore be taken as unity. However the dilatancy correction factor was found to vary linearly with \( H/D \) and increase non-linearly with dilatancy angle \( R_v \) for soil with an associated flow rule. A value of \( R_v \) for non-associated materials may be obtained by linear interpolation. The stress-dilatancy relationship of Rowe (1969b) may be used to determine a value of \( \psi \) from values of \( \phi_{ps} \) and \( \phi_{cr} \);

\[
\tan(45° + \psi/2) = \tan(45° + \phi_{ps}/2) / \tan(45° + \phi_{cr}/2)
\]

(6)

**CENTRIFUGAL FACILITY AND TESTING PROGRAM**

Tests were carried out in the centrifuge in the Civil Engineering Department at the University of Liverpool. The facility has been described in some detail by King et al. (1984) and Dickin (1988). The machine is a Model G-380-3A supplied by Triotech Inc. of California and converted in house to geotechnical use. It incorporates two balanced swinging carriages each capable of carrying a soil package 0.57 m long, 0.46 m wide and 0.23 m deep to accelerations up to 200 gravities. However with a full payload tests are usually restricted to 100 gravities, well within the capacity of the machine. The optimum scaling radius with a full package is 1.15 m and the maximum stress difference between model and prototype is 3.5%. The package arrangement shown in Figs. 2 and 3 was designed to enable strain-controlled uplift tests to be performed. The buried models were attached centrally to the
pulling mechanism, a gearbox shaft driven by a small high torque, low speed A. C. motor (PARVALUX, TYPE 21S1S), by a single 3 mm diameter steel rod. Tests were carried out at a constant rate of displacement of 0.23 mm per minute. Uplift resistances were monitored at 18 second intervals by a NOVATECH tension/compression load cell mounted in-line and vertical displacement by a 25 mm travel SANGAMO linear displacement transducer. The stainless steel model pipes were 25 mm in external diameter, 0.7 mm in wall thickness and 213 mm in length to give a clearance fit between the internal glass-faced walls of the test package. The model pipes were assumed rigid since their pipe diameter/wall thickness ratio of 36 was considerably lower than the value of 120 quoted by Compton et al. (1978) as the lower limit for steel pipes to be considered flexible. Model strip anchors were 25 mm in width, 6 mm thick and 213 mm in length, and were also steel and considered rigid. In the main testing programs the 25 mm anchors were spun to accelerations of 40 gravities to simulate 1m prototypes. In addition a series of tests on 15, 25, 37.5 and 50 mm diameter pipes spun to 66.7, 40, 26.7 and 20 gravities respectively was performed to check the validity of the modelling. A few tests were also included to examine the influence of pipe diameter and surface roughness. All models were embedded in dry Erith sand, a fine, uniform material having 80% of its grains between 0.125 mm and 0.25 mm and a uniformity coefficient of 1.5. The sand has maximum and minimum porosities of 49.5% and 34% and its specific gravity is 2.65. Tests were carried out in both dense and loose sand packings, unit weights being 16 and 14.5 kN/m³ respectively. A dense sand bed was achieved by sand raining at low intensity from a height of 1.85 m through a baffle with 3 mm diameter holes at 50 mm centres. Loose packings were obtained using a short drop from a maximum height of 0.36 m at high intensity through a baffle with 4.5 mm diameter holes at 20 mm centres. Different procedures were employed in uplift tests in dense sand reported by Dickin (1988) which were prepared by vibration. The pipe or anchor was set up at a fixed initial level in all tests, the sand cover was then varied to provide a range of embedment ratios $H/D$ between 1.5 and 7.5. The pipes were tested empty since this condition would clearly be the most vulnerable to disturbance. Separate tests were carried out to determine the resistance between the pulling rod and the sand. This increased linearly with depth but was relatively small, being less than 5% of the total uplift resistance for the lowest case, and reduced with embedment. Uplift values obtained in these tests were deducted, along with the self weight of the model, from the resistances recorded in the main series.

**INFLUENCE OF PIPE DIAMETER AND SURFACE ROUGHNESS ON UPLIFT BEHAVIOUR**

The influence of pipe diameter on uplift resistance was examined from a series of tests on the 25 mm diameter model pipe at accelerations of 10, 20, 40 and 80 gravities. The behaviour of 0.25, 0.50, 1 and 2 m diameter pipes was thus simulated. These tests were performed at an embedment ratio of 3.5 in dense sand. As in previous anchor work comparisons of uplift capacity are facilitated by expressing uplift resistance $F_u$ in dimensionless terms as a breakout factor $N_u$ where $N_u = F_u / y'DLH$ where $L$ is the pipe length. The centrifugal modelling laws give $F_u = N^2 F_{um}$, $D_p = ND_m$, $H_p = NH_m$, $L_p = NL_m$, and displacements $d_p = Nd_m$ and hence $N_u = N_{um}/N$ where subscripts $p$ and $m$ refer to prototype and model respectively. Displacements normalised to diameter $d/D$.

![Graph showing variation of mobilised breakout factor $N_u$ with relative displacement $d/D\%$ for various pipe diameters at $H/D = 3.5$ in dense sand](image)
termed relative displacements, are the same in model and prototype. Relationships between mobilised breakout factor $N_{up}$ and relative displacement $d/D$ are summarised in Fig. 4 which also includes data from a conventional model test in the centrifuge bucket with the machine static. Although ultimate loads from this series increase markedly with diameter, Fig. 5 suggests that the influence of diameter on dimensionless factor $N_e$ is not significant for pipe diameters in excess of 1 m. Similarly failure displacements $d_f$ also increase markedly with diameter, but when expressed in dimensionless terms as in Fig. 6 are almost independent of scale. The influence of size on pipes in loose sand and on pipes at deeper embedments in dense sand requires further investigation.

The influence of surface roughness was investigated by comparing test results from steel pipes with those from similar pipes coated with a thin layer of sand using epoxy resin. This effect was found to be relatively minor for pipes at embedment ratios greater than 3.5, but significant for pipes at shallow depths, in particular in dense sand (see Fig. 9).

### BEHAVIOUR OF PIPES AND STRIP ANCHORS

Typical relationships between pullout force and displacement for steel pipes and anchors at shallow, inter-

![Figure 7](image-url)  
**Fig. 7.** Comparison between load displacement relationships for 25 mm diameter pipe and 25 mm strip anchor in centrifugal model tests at 40 g in dense sand

![Figure 8](image-url)  
**Fig. 8.** Comparison between load-displacement relationships for 25 mm diameter pipe and 25 mm strip anchor in centrifugal model tests at 40 g in loose sand

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Fig. 5. Variation of Breakout factor $N_e$ with pipe diameter $D$

Fig. 6. Variation of relative displacement $d_f/D\%$ with pipe diameter $D$
mediate and deep embedsments in dense and loose sand are shown in Figs. 7 and 8. The anticipated higher resistances at lower displacements are generally observed for models in dense sand compared with loose sand. Maximum resistances in all tests on pipes at embedment ratios shallower than 2.5 are reached at relatively small displacements, followed by a marked loss in resistance. Thus curves exhibit well defined peak values. At intermediate depths the distinct peaks characteristic of tests in dense sand are absent in loose sand, while peaks are not encountered in any tests at deep embedments. All tests exhibit load oscillations at large displacements to some degree probably due to the collapse of sand into the gap beneath the model with upward movement. Difficulties in identifying peak resistances led to the adoption of the maximum value of the smooth curve joining successive crests of these oscillations as the 'peak' uplift resistance value. These smoothed curves are shown in Figs. 7 and 8. Maximum resistance values, expressed dimensionlessly as prototype breakout factors ($N_{op}$), are plotted against embedment ratio in Fig. 9 for tests in both dense and loose sand. Tests on pipes with sand-coated surfaces are also included in Fig. 9. Breakout factors are seen to increase markedly with embedment for models in dense sand, and to a lesser extent for those in loose sand. Although similar resistances are obtained for anchors and steel pipes, sand-coated pipes offer greatest resistance. Steel pipes in loose sand also exhibit slightly greater uplift resistances than strip anchors at shallow depth. In general differences between steel pipes and anchors are relatively small lending support to the assumption of Trautmann et al. (1985). The failure mechanism around a 25 mm diameter model pipe originally at an embedment ratio of 3 in dense sand for a test at 40 gravities shown in Fig. 10 also supports this assumption. Horizontal layers of dyed sand were incorporated in the model during preparation to facilitate subsequent observation of the slip planes. The zone of soil mobilised above the pipe is more extensive than is assumed in the Vertical Slip Surface Model and not dissimilar to that found previously by the author (Dickin 1988) above a strip anchor. However, a significant difference could clearly occur in the extremely shallow case of $H/D = 0.5$ due solely to geometric factors when the $N_{op}$ value of an empty pipe would approach 0.2 while that of a strip anchor would be unity. It is apparent from Fig. 9 that this difference would be considerably reduced, or even reversed, since breakout factors for the shallowest pipes tested, particularly when coated with sand, exceed those for anchors due to the additional contribution from side friction between the pipes' curved surfaces and the soil. Breakout factors from the present study and those obtained by Trautmann et al. (1985) compare well in Fig. 11 considering differences between the two programs including pipe diameters, backfill unit weights and testing procedures. Figure 12 shows the increase in displacement at failure normalised to pipe diameter or anchor width, $d_1/D$, with increased embedment also evident in Figs. 7 and 8, and the larger displacements associated with tests on pipes in loose sand. In general failure displacements for sand-coated pipes are only slightly smaller than those for steel pipes in dense sand. Larger displacements are required to fail anchor plates at all depths in dense sand, than pipes with either steel or sanded surfaces. In contrast smaller failure displacements for plates are observed than for pipes in loose sand. Failure displacements are slightly larger than found by Trautmann et al. (1985) for dense sand according to Fig. 12. Results for loose sand obtained by Trautmann et al. (1985) are very low due to their method of defining failure.
COMPARISON WITH THEORETICAL PREDICTIONS

Comparisons between centrifugal test data for steel pipes and theoretical predictions for 1 m strip anchors are seen in Figs. 13, 14 and 15. These predictions were obtained using plane strain friction angles appropriate to the stress level at the centre of the pipe. Friction angles were obtained from Dickin (1988) and ranged from 51° to 48° for dense sand and 41° to 38° for loose sand. Theoretical values for pipes in dense sand based on the work of Meyerhof and Adams (1968), Murray and Geddes (1987) and Saran et al. (1986) derived from peak plane strain friction angles \( \phi_{ps} \) in Fig. 13 are approximately 60% higher than observation, while values from Vermeer and Sutjiadi (1985) exceed centrifuge data by about 30%. Murray and Geddes (1987) also found that Eq. (3) gave higher results than their tests on 50 mm strip anchors in medium dense sand. The design charts of Rowe and Davis (1982), using dilatancy angles of 13° obtained from Eq. (6), give good agreement at shallow embedments and are within 20% of observation for deeper pipes. Excellent predictions are obtained from the expanded cavity model of Vesic (1971). Theoretical values were also computed using a mobilised friction angle \( \phi_{mp} \) to allow for progressive failure as suggested by Rowe (1969a). Assuming a progressivity index of 0.5, \( \phi_{mp}=\left(\phi_{ps}+\phi_{cr}/2\right) \) where \( \phi_{cr} \) is the critical state friction angle. Applying this ap-
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Fig. 15. Comparison between breakout factors for 1 m diameter pipe in loose sand & theoretical predictions

Fig. 16. Comparison with predictions from vertical slip surface model

approach to the method of Rowe and Davis (1982) produces a reduced $\psi$ value of 6° which results in close agreement with measured capacities at shallow depth and slightly higher values within 15% for deeper cases as seen in Fig. 14. Values from Vesic (1971) and Vermeer and Sutjiadi (1985) are also close to observation. However the other theories still produce unacceptably high values. Figure 15 suggests that all theories overestimate the uplift resistance of steel pipes in loose sand, the difference increasing with depth. Closest agreement is again obtained from the theories of Vesic (1971), Rowe and Davis (1982) and Vermeer and Sutjiadi (1985) although values are at least 25% higher than observation. Computations with a mobilised friction angle would scarcely improve agreement since in the case of loose sand the value of $\phi_{ps}$ is close to $\phi_{m}$. Trautmann et al. (1985) found that their data agreed with values from the simple Vertical Slip Surface Model with $K=0.5$ and 0.75 for loose and dense sand respectively. A similar approach produces good agreement with experimental data for steel pipes in dense sand as seen in Fig. 16, but gives relatively high breakout factors for pipes in loose sand. However excellent agreement is seen for loose sand when the $K$ value is taken as $K_0 = 1 - \sin \phi$.

CONCLUSIONS

The maximum uplift force on pipes in sand is strongly influenced by depth of burial and density of the soil, but the influence of pipe surface roughness becomes insignificant at embedment depths greater than $H/D = 3.5$. The limited study in dense sand shows that although the total uplift force increases with pipe diameter, the dimensionless breakout factor reduces with increase in diameter for steel pipes up to 1 m and is relatively insensitive to further increase in diameter. The assumption of Trautmann et al. (1985) that the uplift resistance of a buried pipe is essentially the same as that of the equivalent strip anchor is broadly supported by the centrifuge data presented in this paper. However significant differences do exist in displacements to failure.

Several of the failure theories considered overpredict observed uplift forces, in some cases quite considerably. However, as found by Trautmann et al. (1985), the finite element-based work of Rowe and Davis (1982) compares well giving values within 20% of observation in the case of pipes in dense sand. The expanded cylindrical cavity approach of Vesic (1971), which is directly applicable to pipeline geometry, and the analyses of Vermeer and Sutjiadi (1985) based on the dilatancy angle, also yield reasonable agreement with observation. Most theories seriously overpredict maximum soil forces on pipes in loose sand, although the simple Vertical Slip Surface Model yields sensible agreement when the value of $K$ is taken as $K_0$, the at rest earth pressure coefficient.

As noted by Trautmann et al. (1985) where an evaluation of pipe stresses is of primary concern, overprediction of soil restraint is conservative, while for pipe stability overestimation would be optimistic.

REFERENCES


NOTATION

- D = diameter of pipe or width of strip anchor
- d = displacement
- \( d_t \) = failure displacement
- K = lateral stress coefficient
- \( K_0 \) = earth pressure coefficient at rest
- \( F_u \) = uplift resistance
- \( F_u^\prime \) = uplift resistance/unit length
- \( F_p \) = basic anchor capacity
- \( H \) = depth of embedment measured to the centre of the object
- \( H_r \) = vertical extent of failure surface (deep anchor)
- \( H/D \) = embedment ratio
- L = length of pipe or strip anchor
- N = acceleration factor
- \( N_a \) = anchor breakout factor
- \( n \) = porosity
- \( R_k \) = correction factor for initial stress state
- \( R_o \) = correction factor for anchor roughness
- \( R_p \) = correction factor for dilatancy
- \( y' \) = effective soil unit weight
- \( \phi_v \) = constant volume friction angle
- \( \phi_{vp} \) = mobilised plane strain friction angle
- \( \phi_p \) = plane strain friction angle
- \( \psi \) = dilatancy angle

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