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PIPELINE UNBURIAL BEHAVIOUR IN LOOSE SAND

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ABSTRACT

Small diameter pipelines are routinely used to transport oil and gas between offshore production plants and the mainland, or between remote subsea well-heads and a centralised production facility. The pipelines may be placed on the soil surface but it is more usual that they are placed into trenches, which are subsequently backfilled. For the buried pipelines a well established problem has been that of upheaval buckling. This occurs because the fluid is usually pumped through the pipes at elevated temperatures causing the pipeline to experience thermal expansion which, if restrained, leads to an increase in the axial stress in the pipeline possibly resulting in a buckling failure. A secondary phenomenon that has also been identified, particularly in loose silty sands and silts, involves floatation of pipelines through the backfill material, usually shortly after burial.

At the University of Oxford a project sponsored by EPSRC and Technip Offshore UK Ltd has commenced to investigate in detail the buckling and floatation problems. The main aim of the research programme is to investigate three-dimensional effects on the buckling behaviour. The initial experiments involve the more typical plane strain pipeline unburial tests to explore the relationship between depth of cover, uplift rate, pipeline diameter and pullout resistance under drained and undrained conditions. The second and main phase of experiments involves inducing a buckle in a model pipeline under laboratory conditions and making observations of the pipe/soil response. This paper will describe the initial findings from the research including a) plane strain pipe unburial tests in loose dry sand, and, b) initial small scale three-dimensional buckling tests. The paper will then describe the proposed large scale three-dimensional testing programme that will be taking place during 2006 and 2007.

Keywords: Offshore Pipelines, Upheaval Buckling, Sand, Laboratory Tests, Pipeline Floatation.

NOMENCLATURE

c_v	Coefficient of Consolidation [L^2/T]
D	Diameter [L]
d	Pipe uplift displacement [L]
E	Young's modulus [F/L^2]
f_d	Dimensionless uplift factor [-]
F_u	Uplift force [F]
H	Depth of cover [L]
I	Second Moment of Area [L^4]
L_p	Length of pipe [L]
N_u	Dimensionless uplift force [-]
P	Axial force [F]
v	Velocity of pipe [L/T]
x	Axial displacement [L]
v_n	Normalised pipe velocity [-]
ϕ'	Effective friction angle [degrees]
γ'	Effective (buoyant) unit weight [F/L^3]

INTRODUCTION

Pipelines are used extensively in the offshore oil and gas industry for the transportation of product between production plants and the mainland, or between remote subsea well-heads and a centralised production facility. It is typical for the fluid within the pipe to be at a higher pressure and temperature than the ambient pressure and temperature of the surrounding water. The pipelines may be placed on the soil surface, but it is more usual that they are placed into trenches, which are subsequently backfilled. Trenching typically involves processes that can be categorised as; (a) jetting: the soil (sand or soft clay) is fluidised, allowing the pipe to sink into the resulting 'quicksand' or, (b) ploughing: a trench is mechanically ploughed into the soil (sand or clay), the pipe is laid and the spoil is mechanically filled back over the pipe. In both cases the remoulded strength of the soil above and around the pipe is significantly lower than that of the in-situ material.

Burial of the pipeline has two advantages:

(a) The pipeline becomes protected against damage by marine vessel activity, for example the laying of drag anchors or fishing equipment such as trawl boards.

(b) Heat loss along the length of the pipe is minimised. Temperature has a significant impact on the viscosity of the fluid and hence flow rate (the pipe is also insulated to mitigate this effect).

The pipeline is usually installed in a stress free condition, apart from residual stresses which have accumulated during the fabrication and installation procedure. When the hot fluid is pumped through the pipeline, the pipe section will undergo both thermal expansion and pressure induced stresses. Sections of pipe will seek to expand but will be restrained axially by the frictional resistance of the pipe against the soil. Stresses will therefore build up and a point may be reached where the axial load acting on a section of the pipe will be greater than the buckling load for a similar length of pipe. A buckle might occur if the pipe can overcome the resistance of the soil to lateral movement, with the pipe moving in direction of least resistance. In most cases this will be the resistance of the remoulded soil above the pipe and so therefore the pipe will tend to move out of the ground and be prone to damage. As the soil surface may not be 'flat' it is likely that the trench within which the pipe is placed may contain a number of slight imperfections, either hills or valleys. It is highly probable that the buckle may initiate at the imperfections within the trench profile. The buckling problem described here is called upheaval buckling, and a useful review of this instability problem, as well as the related lateral buckling problem, is given in [1].

Upheaval buckling can be prevented by increasing the depth of the cover, but there are economic and physical constraints to doing this. Guijt [2] outlines a number of other strategies that can be employed, including the reduction of the wall thickness by adopting different design approaches, increasing the lay tension (*i.e.* pretensioning the pipe) or by using a process of intermittent rock dumping (outlined in [3]). Alternatively Finch [4] outlines research aimed at optimising the soil-pipeline axial friction so as to reduce the possibility of buckling. If upheaval buckling does occur then remediation is usually through expensive processes such as rock-dumping over the exposed sections of pipe.

The upheaval buckling problem described above is not a new one. It has been researched for a number of years, principally because the costs are large when remedial action is required. The main approach has been to investigate the resistance of the soil to the upward movement of the pipe. The problem is assumed to be two-dimensional (2-D) and the experiments are conducted under conditions of plane strain. A piece of pipe would typically be buried in the soil, with the loads and displacements being measured as the pipe is pulled from the soil. By considering a 'vertical slip' model, or variations on it, expressions can be determined from a theoretical basis and calibrated against the experiments [5]. For example the following expression [4,6] gives the drained soil resistance per unit length, F_u/L_p, of a 2-D circular pipe moving upwards through a frictional soil:

$$\frac{F_u/L_p}{\gamma' HD} = 1 + f_d\left(\frac{H}{D}\right) \qquad (1)$$

where H is the depth of cover, D is the diameter of the pipe, γ' is the effective (buoyant) unit weight of the soil and f_d is a dimensionless factor that can be determined by experimental methods. Such factors have been proposed by a wide variety of researchers for example Schaminee *et al.* [6] describe a large experimental programme aimed at determining the factor f_d. Similar expressions have been proposed for clay soils, but the experimental data is sparser. The problem is more complex, as the clay spoil that is used to cover the pipeline is often blocky and not a homogenous material [7].

There are two major drawbacks to tackling this problem in the manner described above:

a) The assumption of plane strain conditions means that three-dimensional (3-D) effects are not considered in the experimental approaches, nor in the analytical/theoretical models. Such effects might include dynamic considerations as the buckle occurs, at which stage there will be a complex interaction between the pipe and the soil as the buckle propagates and the pipe moves rapidly through the soil.

b) Realistic installation methods are not usually replicated. It is typical, in say centrifuge testing, for the pipeline to be placed within the soil sample as it is being prepared [8,9]. The soil resistance is therefore likely to be overestimated in the case of i) sand, where loosening would occur on trenching in the field, and, ii) clay, where the soil would undergo remoulding and re-consolidation in the field.

Whilst upheaval buckling is a well recognised problem [10,11] a secondary problem of pipeline 'floatation' has more recently come to the fore [4,12]. The evidence suggests that floatation may occur either during the trenching operation or shortly afterwards once the pipeline begins operation. Cathie *et al.* [12] describe post installation surveys of a pipeline in the Northern North Sea where large lengths of the pipe are recorded as having risen out of a backfilled trench. It appears that once a section of pipeline starts rising to the surface an 'unzipping' process occurs whereby large lengths of the pipe move to the surface. The remedial action, of either rock-dumping on the exposed sections or retrenching, becomes very expensive. The unzipping mechanism is exacerbated by the fact that the pipeline is designed with a low specific gravity; buoyancy is increased further as the hot product is pumped through it. There is a paucity of data on pipeline floatation although Sumer *et al.* [13] have studied the problem of pipeline floatation due to fluid pressures induced in the seabed by surface waves and currents. Cathie *et al.* [12] outline a proposed mechanism for pipeline floatation and identify key characteristics of the soil profile that should serve as a warning indicator. There is however no rigorous framework that can be applied in the design of the pipelines to mitigate this effect.

RESEARCH PROJECT

A project sponsored by EPSRC and Technip Offshore UK Ltd has recently commenced to investigate the issues of pipeline unburial (upheaval buckling and floatation) in sand using both 2-D and 3-D testing. This will allow a thorough investigation of interaction effects, 3-D effects and highlight any shortcomings of the 2-D approach. The primary motivation for doing this are two case histories [10,12] where designs based on industry standard procedures have proved to be unsatisfactory and have led to expensive remedial action.

To the authors' knowledge, 3-D experiments have not been employed extensively for studying the upheaval buckling case, the floatation case or the combination of both. The 2-D approach is usually favoured since full scale sections of pipes can be tested in experiments where the soil stresses are accurately modelled. In order to be economical, 3-D tests require the pipeline model to be scaled down considerably, and therefore careful attention to the effects of stress level on the pipe response is required. One approach, such as that adopted by Moradi and Craig [14] and Moradi [15], is to carry out experiments in a centrifuge. They investigated the upheaval buckling behaviour of a 6mm diameter tube 1800mm long at radial accelerations equivalent to 20g and 40g. The buckling was induced successfully by pumping hot water through the pipe. Whilst their experiments were successful, it is anticipated that 1-g laboratory experiments will allow a broader range of parameters to be studied and more detailed measurements to be obtained, with a view to recommending design guidelines. Maltby and Calladine [16,17] performed 1-g experiments involving a pipe of diameter 6mm and length 5m in dry sand where the buckling was induced by means of both an increased internal pressure and an axial force applied to the end of the pipe. It is anticipated that the 3-D experiments in this research will target pipe diameters up to 50mm.

The related problem of the lateral buckling of a pipeline resting on the soil surface (or partially embedded) has been studied in detail by a number of researchers including Miles and Calladine [18] and Brennodden *et al.* [19]. The lateral instability problem has been investigated both as a 3-D problem and also, more recently, by looking at the response of a plane strain pipe section to lateral resistance (*i.e.* equivalent to the typical approaches taken for the upheaval buckling problem). Numerical models that can be used for the lateral buckling problem include those described by Wagner *et al*. [20] and Wolfram *et al.* [21]. Zhang *et al.* [22] discuss the development of a macroscopic plasticity model treating the pipeline as a foundation under combined loading. Such a model can readily be embedded within a numerical code describing the structural response of the pipeline, so that the resulting pipeline behaviour can be calculated.

Initial 2-D Testing in Loose Sand

The first phase of testing has been to perform plane strain pullout tests on sections of pipelines at a range of diameters varying from D_0 up to $8.8D_0$ where $D_0 = 25\,\text{mm}$. The larger pipe approaches the size that is typical for full scale pipes in the field. The design and dimensions of the testing equipment and perspective image of the device is shown in Figure 1. In the following the depth of embedment (*H*) is measured to the centre of the pipe, which has a diameter *D*. A conventional laboratory sand was used and was placed into the tank in a loose state. The model pipeline was lowered into the tank to the prescribed depth and the tank was filled with sand to the required level. The decision to carry out tests in loose sand was so as to replicate the field condition, where either jetting or trenching of the pipelines results in the sand over the pipe being very loose.

The results of an initial experiment in loose dry sand, replicating drained conditions, is shown in Figure 2. There is an initially stiff response to about 85N, after which the load drops off as the pipe is pulled from the ground. This response is characteristic of all the tests carried out for the drained case. The apparent noise in the response is related to "stick-slip" behaviour and interlocking of the sand grains as the pipe is pulled from the ground. More details of the response can be determined from the analysis of photographs taken at various stages during the test (Figure 3). At the deepest level, the mechanism of failure is related to flow around the pipe. Directly beneath the pipe, a cavity forms with a slope at the angle of repose, equal to ϕ', the critical state friction angle of the soil. As the pipe is pulled upwards this cavity is enlarged, (with the load also increasing) until the slope becomes unstable and a flow failure occurs (at which stage the load relaxes). Interestingly this stick-slip behaviour seems to disappear once the failure mechanism propagates to the surface.

A detailed correlation of load-displacement response with the mechanisms of failure is shown in Figure 4. The images shown are in fact derived from a set of three successive photos. By using an optical differencing routine, the areas of greatest movement appear darker, and the areas of no movement appear lighter. This clearly reveals the different mechanisms of failure, and in particular it is noted that images 1 to 6 relate to a deep flow failure, images 7 to 10 relate to a vertical slip model (as discussed earlier) and images 11 to 13 relate to a near surface slip and flow mechanism. Further analysis of the photographic images can be carried out by using particle image velocimetry techniques (such as described in [23]). Figure 5 shows a preliminary PIV analysis illustrating the four distinct mechanisms:

a) initial compression above the pipeline,
b) flow around the pipeline,
c) vertical slip failure,
d) slip and flow near the surface.

The testing programme has investigated effects of scale as this will impact on the selection of pipeline diameter for the larger scale 3-D testing. Figure 6 compares the results from three tests using pipelines of different diameter with an initial embedment depth of *H*/*D* = 3.5. The displacements are normalised by dividing by the diameter, whereas the loads are normalised using the following equation:

$$N_u = \frac{F_u}{\gamma' H_{initial} D L_p} \qquad (2)$$

Broadly, the three curves follow the same trend, indicating that global behaviour is similar. The main differences are in terms of the initial peak load and the level of the stick-slip behaviour exhibited as the pipeline is pulled through the sand. A key parameter to match between prototype and model is the peak load (i.e. N_{upeak}). Figure 7 shows the results from a variety of tests for different initial embedment depths and pipeline diameters. It is clear from the figure that the normalised peak load is higher for the pipeline of diameter D_0 than for the other diameters. The optimum pipeline diameter for the 3-D model tests appears to be $2D_0$ where achieving a representative uplift resistance could be balanced against minimising the loads required to cause buckling of a length of pipe. This is further shown in Figure 8 where the peak results are plotted against the embedment ratio and compared with results found in the literature. In particular there is a good match between the results for pipes of diameter $2D_0$ and $4D_0$ compared with the results from Dickin [5]. There is also a good match, though on the conservative side, with a calculation carried out using the vertical slip model, making appropriate assumptions.

Initial Three-Dimensional Testing in Loose Sand

To establish a preliminary understanding of the buckling behaviour in 3-D some simple experiments involving buckling of a buried pipe with pin-ended connections were carried out. The tests were performed using an aluminium tube of length 1.84m, outside diameter 12.8mm and normal wall thickness 1.86mm. The experimental set-up is shown in Figure 9. The tube was pinned at either end in such a way as to ensure that buckling would occur in the vertical plane, and a small initial vertical imperfection (approximately 0.5mm deflection) was introduced into the pipe so that buckling would occur upwards only. The pipe was loaded via an actuating device at one end with both load and axial displacement of the pipe measured at this point. Some corrections are required to the load and displacement data to account for the flexibility of the loading system and the friction of the loading rod against its PTFE collar as it enters the test-box. The soil used was a conventional laboratory sand placed in a loose state to the height required for particular tests. A range of tests were performed for different H/D ratios, and typical axial load-displacement curves are shown in Figure 10. The lower curve represents the pipe buckling in free air, and the ultimate load should be approximately equal to the Euler buckling load of a pinned-pinned strut ($P_{cr} = \pi^2 EI / L_p^2 = 202N$). In practice the results from the tests give a buckling load slightly above the Euler load, which is related to differences in the wall thickness along the length of the pipe. The upper curve relates to a test where the embedment ratio, *H*/*D*, was equal to 4.7 at the start of the test, thus representing a deeply buried pipe. In this case the peak load is substantially larger than the Euler load, and the increase can only be due to the presence of the soil above the model pipe. There is an initially stiff response as the axial load in the pipe increases. At a displacement of approximately 2.0mm the load drops suddenly, associated with an increase in the axial displacement. This is the point at which the pipe has buckled. The remainder of the curve shows the resistance of the pipe moving through the soil in the buckled shape. Whilst this is substantially below the peak load for the pre-buckled pipe, it is still larger than the Euler buckling load.

Figure 11 shows a collection from the peak loads attained in tests at different embedments. Up until *H*/*D* = 8 there appears to be a linear trend between the peak load and the embedment ratio. Further work is required to understand this relationship and also to relate it to the data presented in Figure 8. If the pipe is deeply buried (*H*/*D* > 5 for instance) then some interesting observations are made. Figure 12 shows the axial load-displacement response for a pipe buried to *H*/*D* of 9.4. In this particular experiment the hinges were left unburied and qualitative observations were made as to the behaviour of the hinges. As the load increases, both hinges are initially observed to remain horizontal. Once the buckling load has been reached (at $P \approx 2000$N in the figure) the hinge closest to the actuating device begins to rotate, whilst the hinge at the far end remains horizontal. This behaviour indicates that the pipe is effectively buckling in a pinned-fixed manner. Eventually the buckle propagates through the length of the pipe and at an axial displacement of 12mm it is observed that the second hinge begins to rotate. At this stage the pipe starts buckling as a pinned-pinned strut. Further work is required to understand this progressive propagation of the buckling, and therefore ascertain an appropriate effective length that can be used in simple Euler buckling calculations. Clearly the use of load control (rather than displacement control) would lead to a rapid and catastrophic ‘unzipping’ failure rather than the gradual progressive failure exhibited in Figures 10 and 12.

A further set of experiments were carried out to investigate the effect of displacement controlled cycling, see Figure 13. The lower curve represents cycles of the pipe in air and it is clear that the response is essentially elastic. The upper two curves represent pipes buried at different depths, and the same qualitative response for both is found. An initial forward axial displacement is applied to the pipe and the loads measured are similar to the monotonic tests shown in Figure 10. In each test the pipe buckled before the cyclic displacements were applied. After buckling, and on further unloading and loading, the pipe ratchets itself out of the soil and the resistance appears to be related to that of a pipe moving upward through the soil mass. Note that during the initial loading, even though the pipe has not buckled, the central section of the pipe might undergo small vertical movements. On unloading these vertical movements may translate into an increased permanent vertical offset that in turn may affect the pipe’s response to cyclic loading. Further experimental work is being carried out in order to quantify the magnitude of the initial loading/displacement, and the size of any vertical offsets accumulated during pre-buckle cycling, before the response of the pipe to axial loading is compromised.

Proposed Programme of Research

The work described in the preceding sections represents an initial programme of research into the response of pipelines. It is part of a much larger programme of research and the following outlines briefly the next stages.

a) 2-D testing: Further 2-D tests are being carried out to understand more about the failure mechanisms for pipelines in loose sand at different embedments. The experiments described above were conducted in dry sand and so represent the fully drained condition. Clearly offshore pipelines are buried in a saturated material and the response of that material may vary from drained through to undrained depending on the loading rate, defined by the uplift speed. There may also be effects of liquefaction to be understood as well. Therefore the next stage in the investigation is to explore the response of the pipeline sections to different rates of loading in saturated sand. The drainage characteristics of the soil in the experiments will depend on the rate of loading, the grain size of the soil, the diameter of the pipeline and the viscosity of the pore fluid. The effect of the different parameters on the drainage might be understood by looking at a non-dimensional velocity of the pipeline;

$$v_n = \frac{vD}{c_v} \tag{3}$$

where v is the actual velocity, D is the diameter of the pipeline and c_v is the coefficient of one-dimensional consolidation of the soil. By changing the variables on the right hand side of this expression it is possible to explore a wide range of v_n. This expression has actually been applied to the response of foundations and t-bar penetrometers in clays and silts. In these cases if v_n is less than 0.1 the response can be characterised as drained whilst if v_n is greater than 30 the response can be categorised as undrained [24,25]. Work will be required to verify whether these ranges are applicable to the pipeline application. To explore a range of responses the main variation will be in the grain size of the soil. Experiments will be carried out using two different soils; coarse grained sand where drained to partially drained responses might be observed and fine grained sand where it is hoped that partially drained and close to undrained responses might be induced.

b) 3-D testing: The main phase of the work involves a more thorough investigation of the 3-D response and linking it to the 2-D responses as characterised in a) above. The initial 3-D experiments described in the preceding text constituted a pilot study and only involved dry sand. As with the 2-D tests, it will be necessary to explore the response of the pipeline embedded in saturated sand. Again experiments will be carried out in soils of two different grain sizes to observe the effect of rate on the response. Preliminary experiments will be carried out in the small test-box already constructed. However an important part of this research project will be that the main set of 3-D experiments will be performed at a much larger scale. To this end a flume/tank is being constructed to enable soil samples up to 8m by 0.6m by 0.8m to be prepared. This will allow model pipelines of length 8m to be tested within the laboratory. It is proposed to carry out experiments on pipes of diameter 25mm and 50mm so as to mitigate the effects of scale.

The pipes will be trenched into the soil sample to replicate the field condition, after which a variety of different tests will be undertaken.

i). The first series of tests will involve lifting a section of pipeline vertically, at different velocities and embedment depths, through the soil and measuring both the load required to do so and the deflection profile along the length of the pipeline.

ii). The second series of tests will involve applying an axial load to the pipeline to induce a buckling response. Again the deflection profile along the length of the pipeline will be measured.

iii). The final series of experiments will involve inducing a buckle in the pipeline through thermal or other means. As before, the deflection profile will be measured along the length of the pipeline.

It is expected that the experimental work described in this section will be conducted during 2006 and in the early part of 2007.

CONCLUSIONS

This paper has described some recent studies of pipeline behaviour in loose sand at the University of Oxford. Firstly, the response of segments of pipeline to uplift under plane strain conditions in loose dry sand was described. The results were compared to previous experimental work reported in the literature, showing favourable agreement. Photographic evidence of the failure mechanism at different embedment depths was also presented. Some preliminary three-dimensional buckling experiments, showing the relationship between axial load-displacement response and embedment depth, have also been presented. In subsequent work it will be important to relate the 2-D test results to the 3-D results through theoretically-based relationships. Finally, the paper has described the framework of a large research project planned for 2006 and 2007 aimed at identifying the influence of 3-D effects on the buckling behaviour of buried pipelines.

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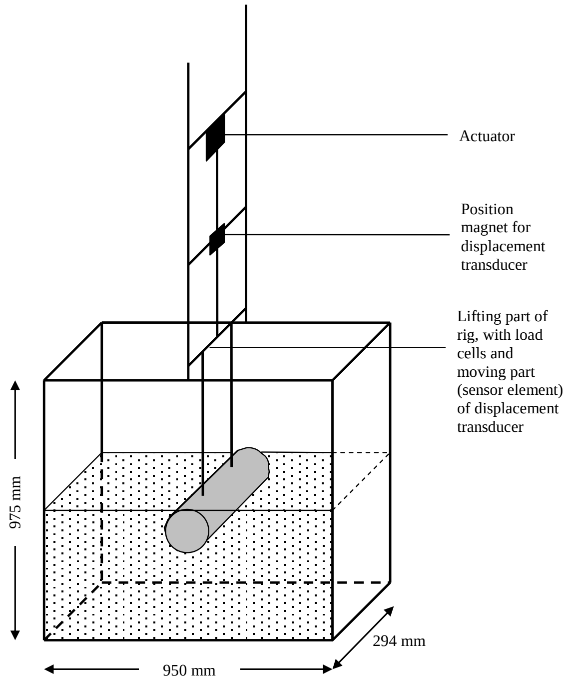

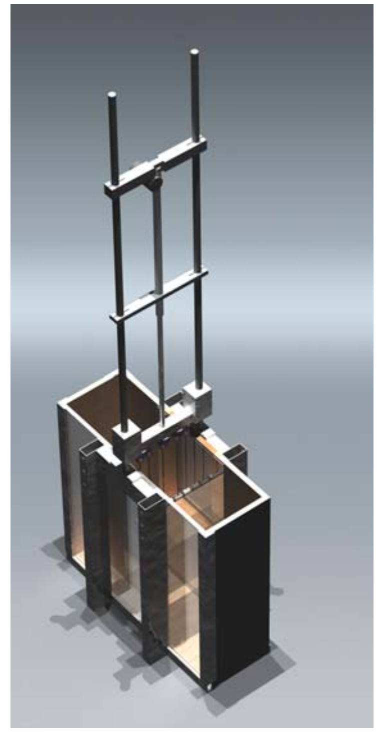

Figure 1 : Plane strain pipeline testing equipment

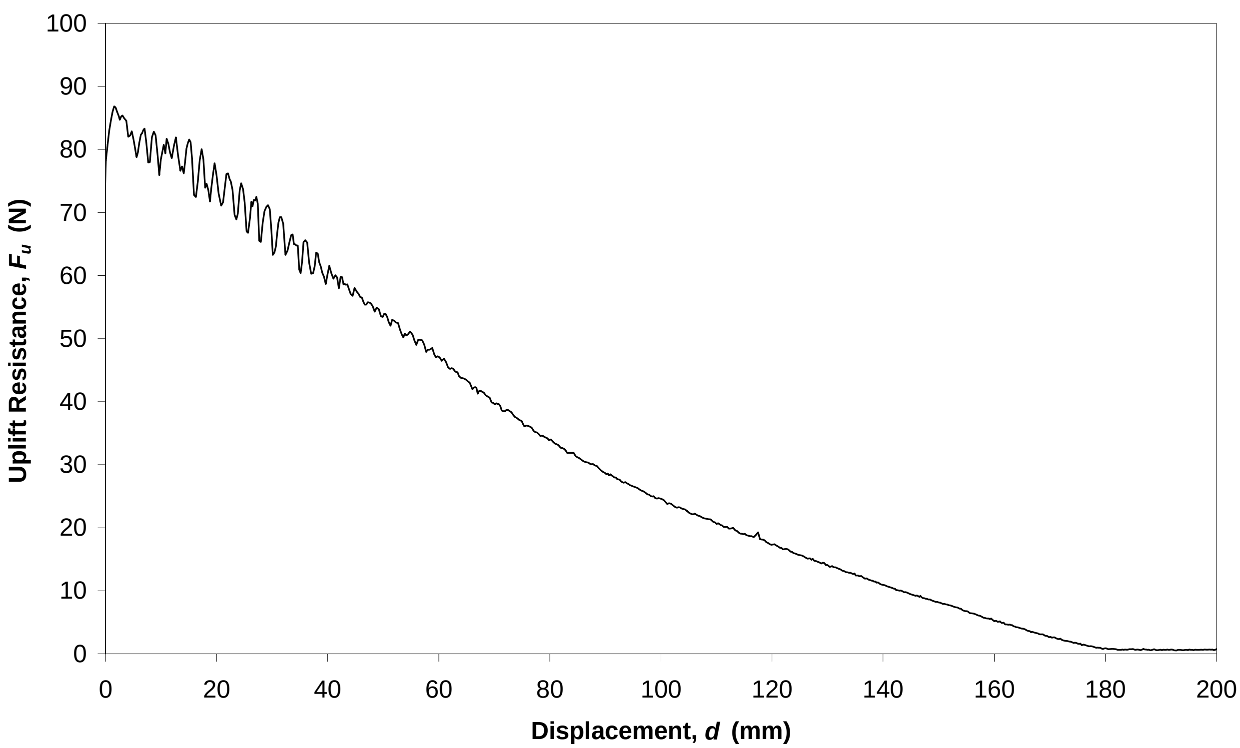

Figure 2 : A typical pullout result (D/D_0 = 2, H/D = 3.5).

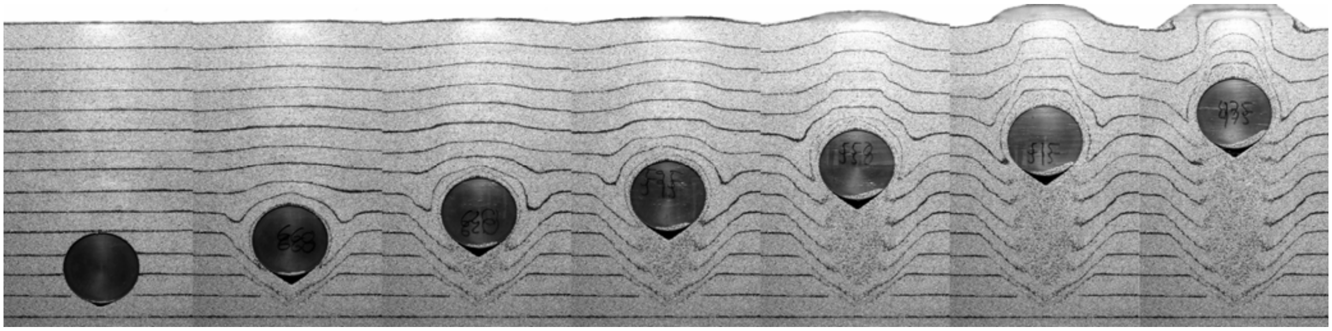

Figure 3 : Photographs of the pipeline as it is pulled through the sand

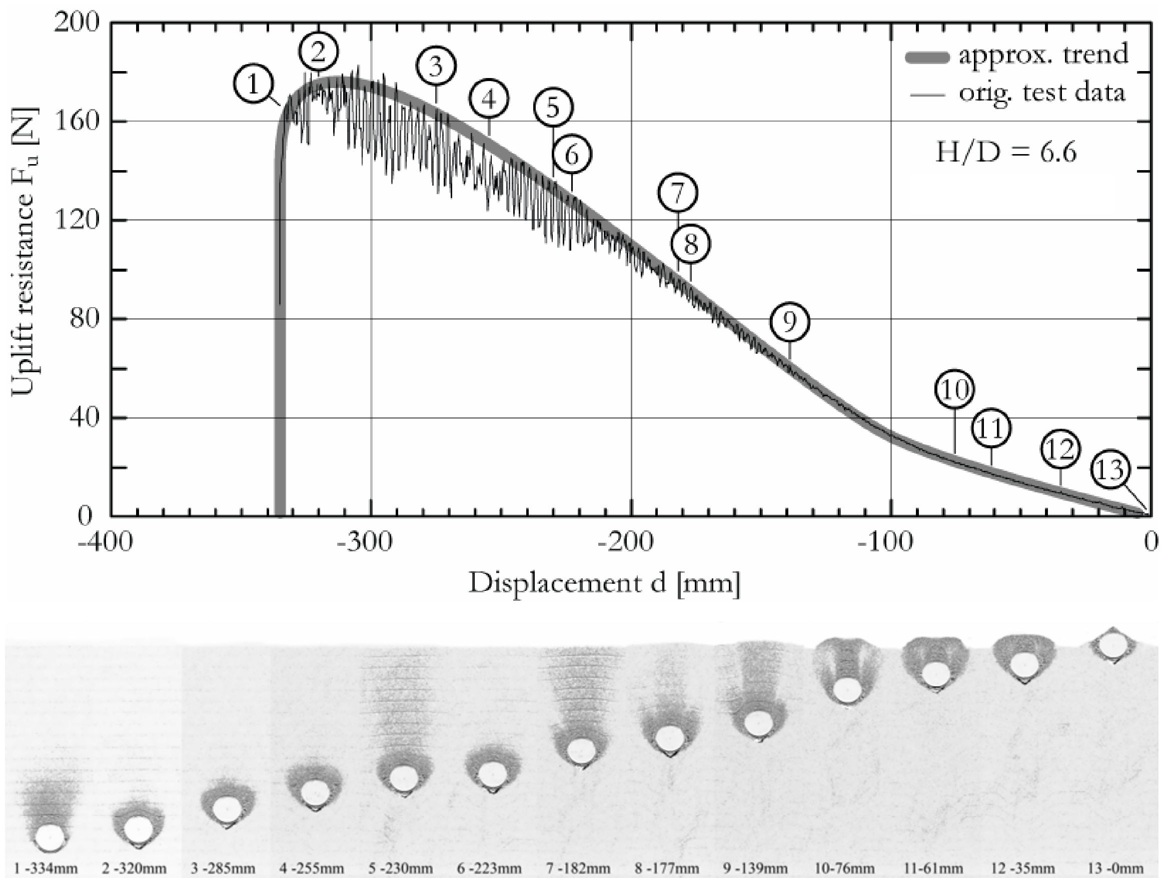

Figure 4 : Correlation of pullout test result with different mechanisms (D/D_0 = 2).

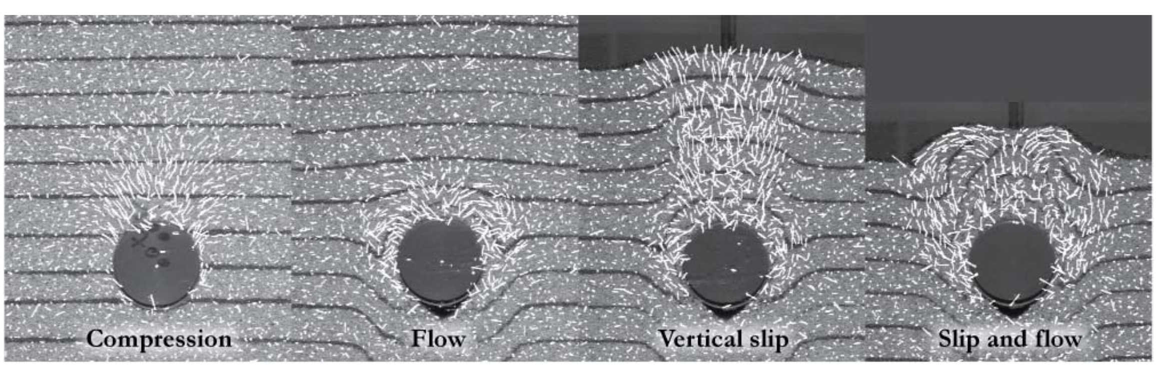

Figure 5 : Preliminary Particle Image Velocimetry (PIV) analysis of the images.

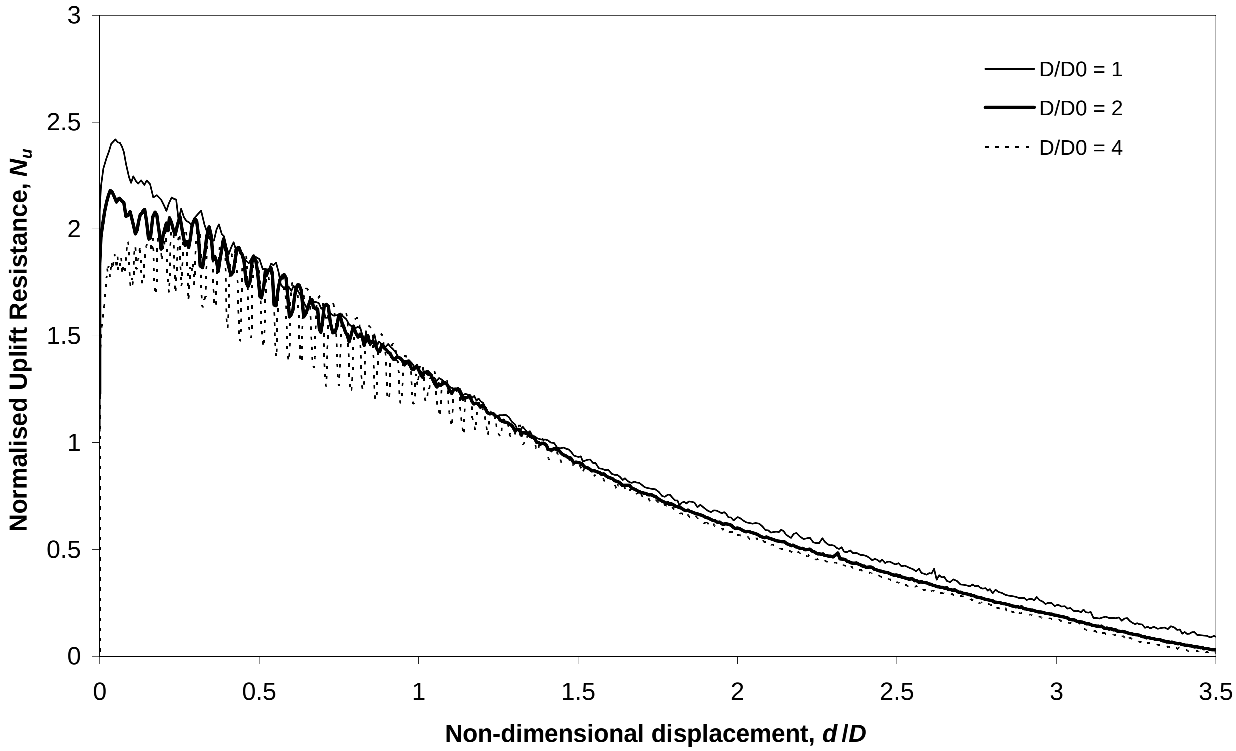

Figure 6 : Comparison of results from tests carried out on pipes of different diameters (H/D = 3.5).

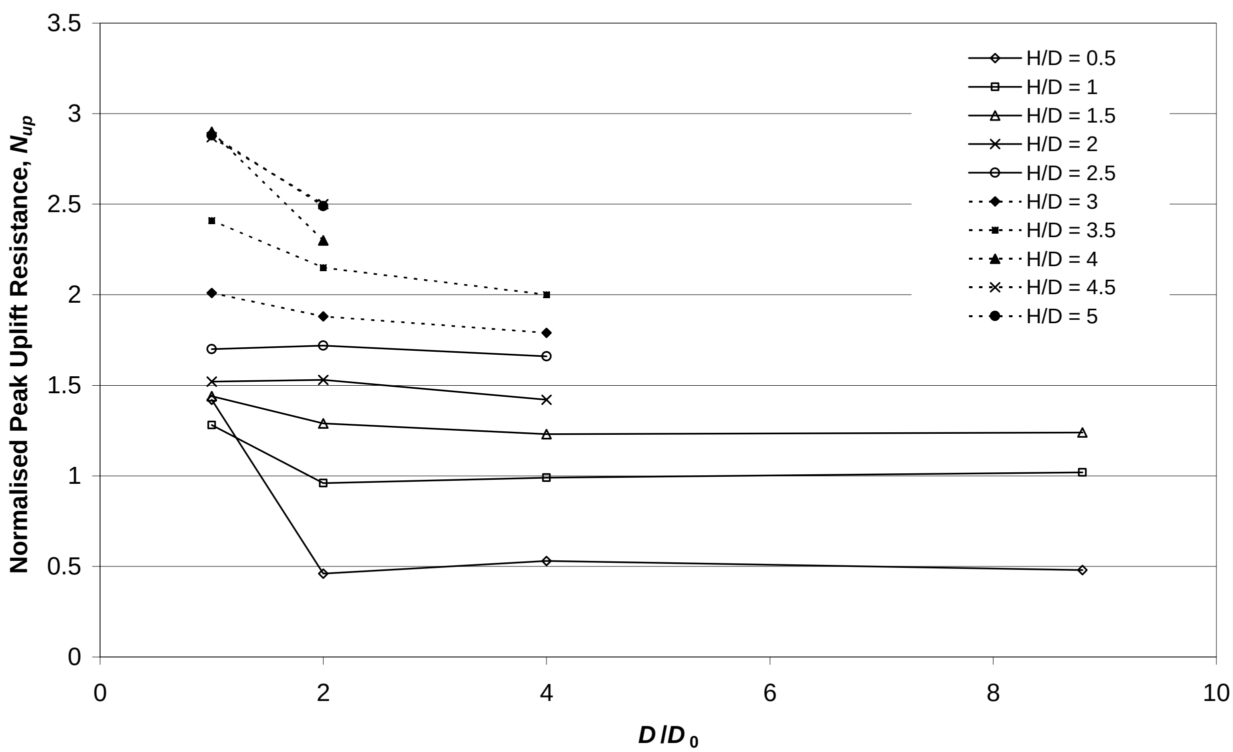

Figure 7 : Peak results as a function of diameter.

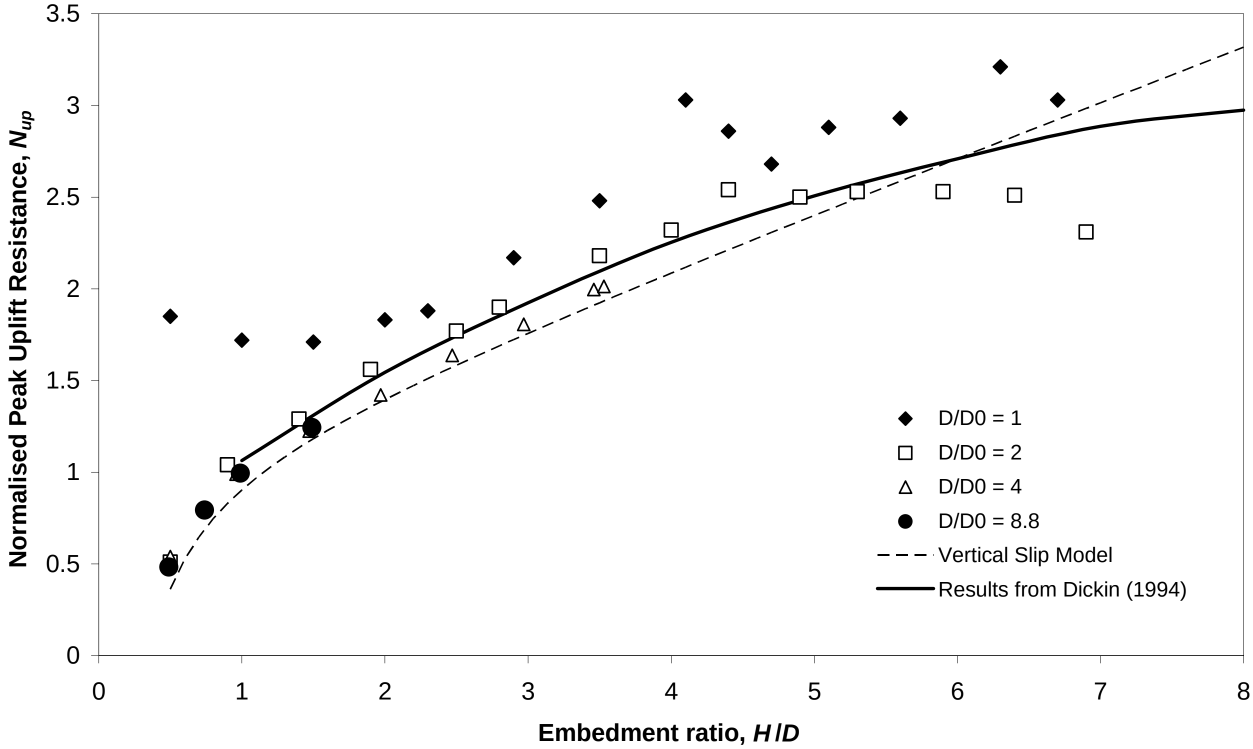

Figure 8 : Peak results compared with previous literature.

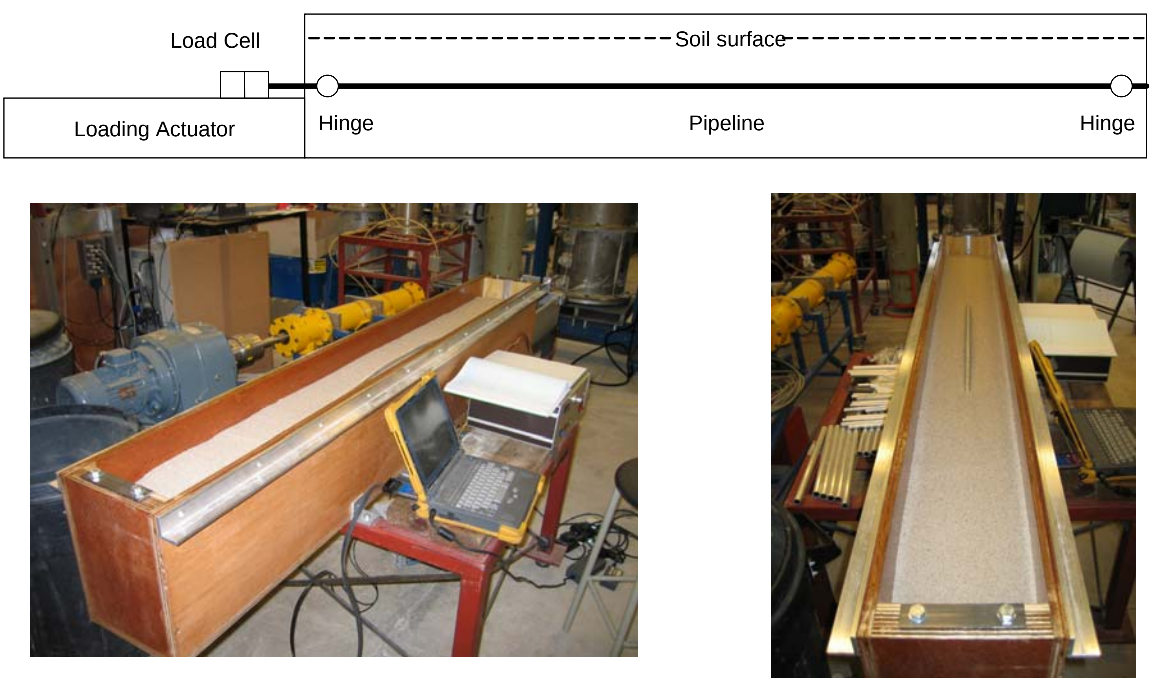

Figure 9 : Set-up for small scale 3-D buckling tests.

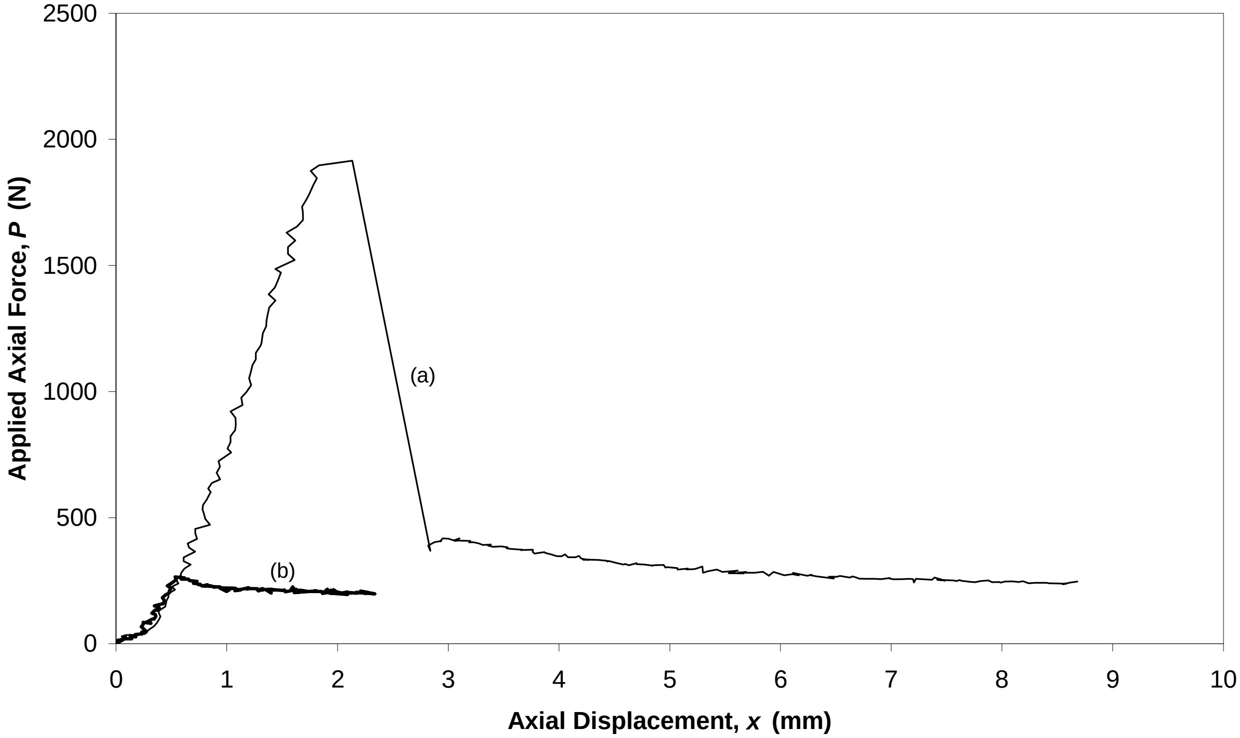

Figure 10 : Two results from the small scale buckling tests: a) *H*/*D* = 4.7, b) no sand.

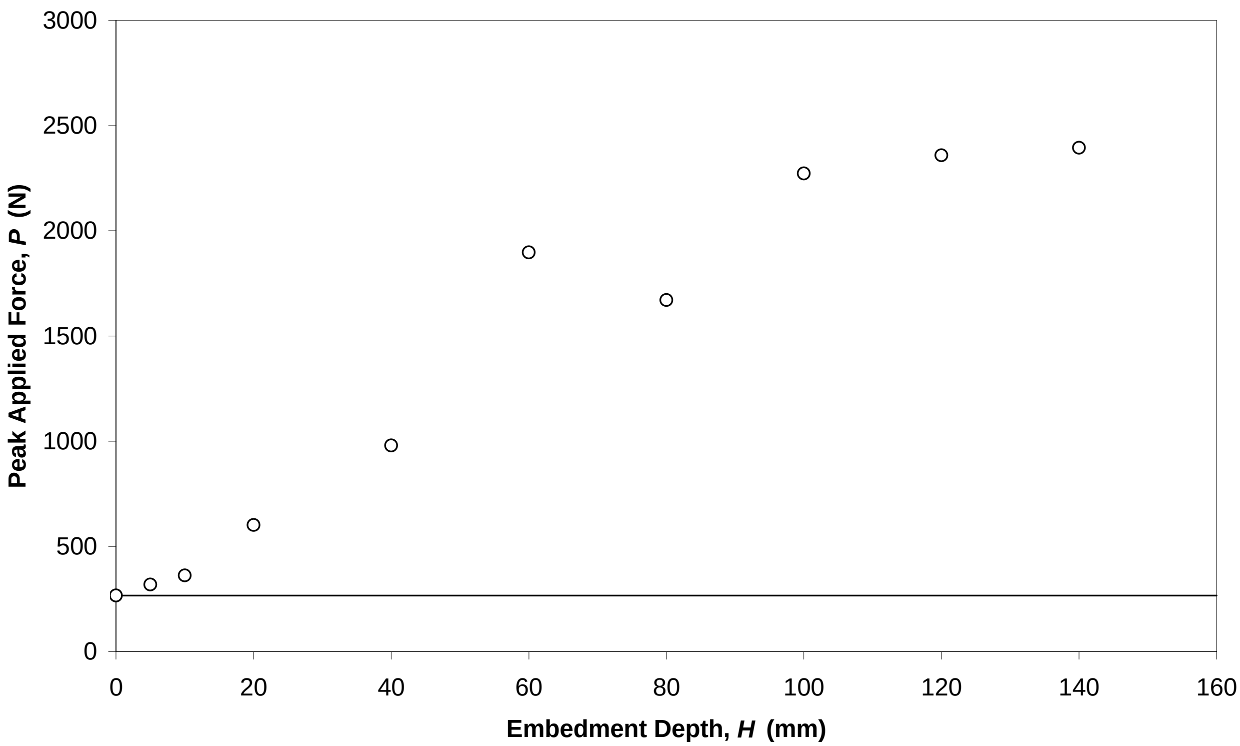

Figure 11 : Summary of preliminary test results for different embedment depths. The flat line represents the buckling load in air.

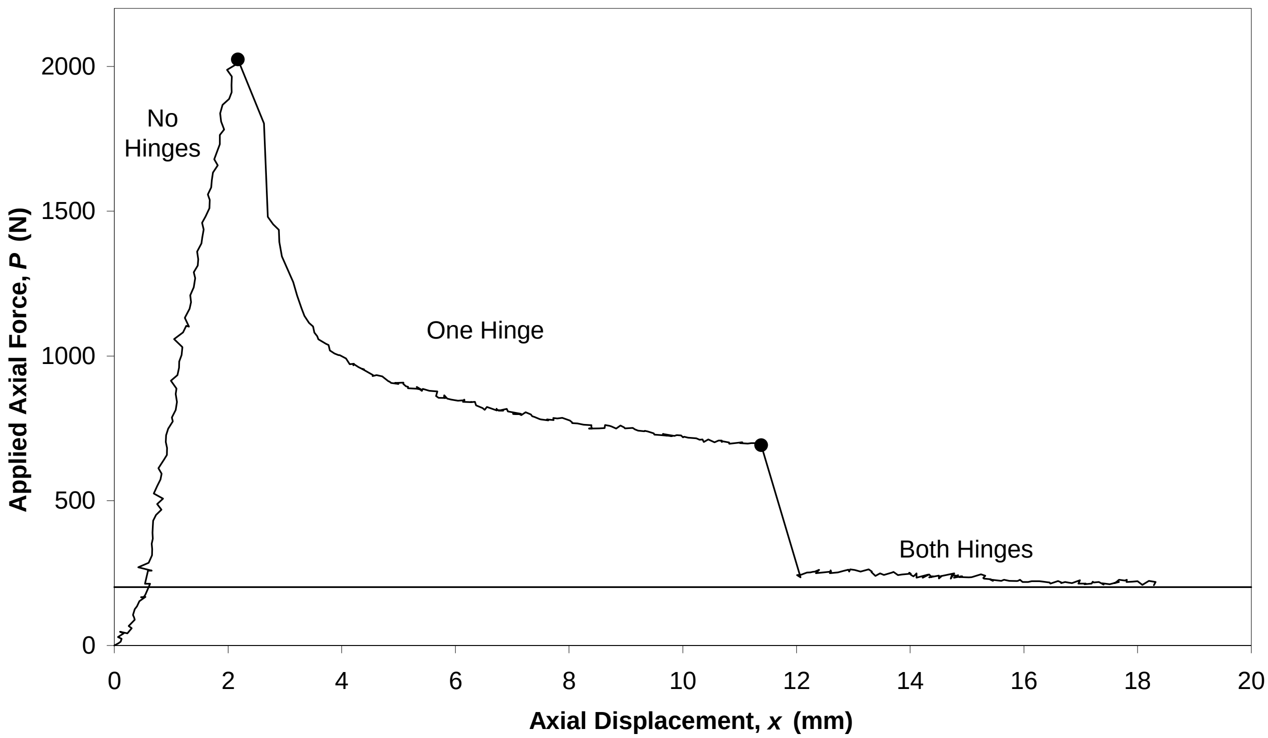

Figure 12 : Observed response for a deeply buried pipeline (*H*/*D* = 9.4). The flat line represents the buckling load in air.

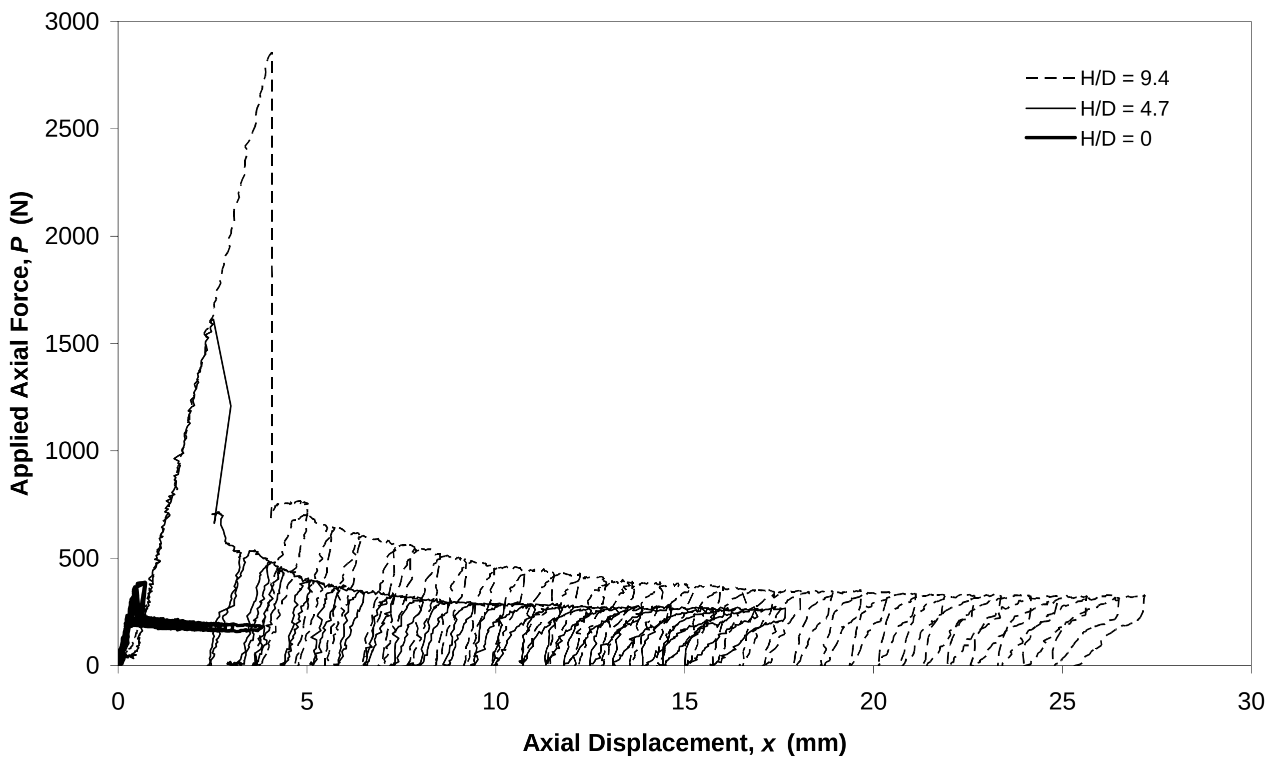

Figure 13 : Observed response for pipelines undergoing displacement controlled cycles.