# A NOVEL TECHNIQUE TO MITIGATE THE EFFECT OF GAPPING ON THE UPLIFT CAPACITY OF OFFSHORE SHALLOW FOUNDATIONS

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**Published in Géotechnique 63(14): 1245-1252.**
**http://dx.doi.org/10.1680/geot.12.P.173**

ABSTRACT

A flexible mat was provided around the periphery of skirted shallow foundations, a so-called 'gap arrestor', to assess the potential to mitigate the effect of gapping on uplift capacity. Results are presented from a series of drum centrifuge tests on skirted foundations with an intact skirt-soil interface, a gapped skirt-soil interface and a gapped interface with gap arrestor, subjected to undrained and sustained uplift. The results are promising, showing that the provision of an effective gap arrestor preserves suction to larger foundation displacements and reduces the rate of displacement under sustained uplift compared with the case of a gapped interface without arrestor.

KEYWORDS: centrifuge modelling; clays; footings/foundations; offshore engineering; suction

Words (intro – conclusions inclusive): 4955. Figures: 9. Tables: 3.

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## INTRODUCTION

Shallow skirted foundations have been used for over two decades by the offshore oil and gas industry - initially for gravity based structures (GBS) and then for a variety of structures such as tension leg platforms (TLP), jackets, storage tanks and subsea frames for pipeline manifolds and oil wellheads. Depending on the application, a skirted foundation may be subjected to uplift from overturning actions or buoyancy forces. Skirted foundations are proven to be capable of taking high magnitudes of tension under short-term and cyclic loads due to the generation of negative excess pore pressure, often referred to as suction (relative to ambient water pressure), between the top plate and the soil plug confined by the skirts. The suction enables mobilisation of a reverse end bearing mechanism, similar to a compression bearing failure mechanism, but in the upward direction.

Reverse end bearing (REB) potential of skirted foundations is widely acknowledged (Puech et al., 1993; Dyvik et al., 1993; Andersen et al., 1993; Watson et al., 2000; Gourvenec et al., 2009; Mana et al., 2012a) and is gradually being accepted by the offshore industry for particular designs. However, general design guidelines and recommended practices (e.g. ISO 2003, API 2011) do not provide specific guidance on how to take account of the several factors that may affect uplift capacity. Standard recommendations involve relying on the frictional resistance mobilised along the skirt-soil interface, which may lead to predicted uplift resistance of an order of magnitude less than available from REB. Designs in which REB have been taken into account have been based on detailed problem specific field investigations and have been restricted to time scales relevant for transient loading (Dyvik et al., 1993; Bye et al., 1995).

Caution over reliance on reverse end bearing capacity has arisen from various uncertainties around the phenomenon, including whether or not full reverse end bearing can be mobilised (i.e. equal to compression capacity), the foundation displacement and duration of time for which suctions can be maintained, and mechanisms that may jeopardise the sustainability of reverse end bearing. Each of these issues have been addressed by previous research projects, results of which have provided increased confidence in relying on reverse end bearing for transient and limited sustained uplift (e.g. Gourvenec et al., 2009; Mana et al., 2012a) but caution over the effect of gapping – which can jeopardise the sustainability of reverse end bearing (Acosta-Martinez et al., 2010; Mana et al., 2012c). Experimental studies comparing the uplift response of skirted foundations with an intact and gapped skirt-soil interface have shown that, while full reverse end bearing can be mobilised with a sealed foundation with an intact skirt-soil interface, undrained uplift capacity is severely affected by loss of suction at small foundation displacements in the case of a gapped interface (Acosta-Martinez et al., 2010; Mana et al., 2012b). Under sustained loading, the rate of displacement can be doubled and the threshold foundation displacement before loss of suction can be halved due to the presence of a gap (Acosta-Martinez et al., 2010; Mana et al., 2012b). There is clearly potential benefit from mitigating the development of a gap along the skirt-soil interface or mitigating the effect of a gap should one form.

Keaveny et al. (1994) first proposed the use of a 'mudliner' to prevent the formation of a tension crack along the active side of an inclined loaded suction caisson anchor. Field tests were carried out on a pair of tangentially connected suction caissons of individual diameter 0.7 m, embedded to a depth of 1.4 m. Details of the geometry of the liner were not reported, but a sketch of the test set up indicated the liner to be square with an edge length of approximately 2.3 m. No details of the material used for the liner were given, but the text described the liner as ‘thin and flexible’. The purpose of the liner was stated as “*to inhibit a tension crack on the active (back) side by preventing a supply of water (and thereby lose suction) between the soil and the model*”. The results showed that the liner fulfilled its purpose and improved the static quasi-horizontal capacity by up to 50 % compared with an equivalent test without a liner, and in which a tension crack formed.

To the authors’ knowledge, the potential of ‘gap arrestors’, as termed in this paper, in preventing the formation of a vertical tension crack along the external skirt-soil interface has not been explored further by other studies available in the public domain for either suction caissons or skirted foundations. The term ‘gap’ is adopted in this paper to indicate a vertical tension crack along the external skirt-soil interface to distinguish between a tension crack across the base of the foundation at skirt tip level. The effect of gapping is arguably more critical for skirted foundations than suction caissons due to smaller embedment of the former and thus the greater potential to short circuit the drainage path to skirt tip level. Gapping is also likely to be most critical under vertical uplift (as opposed to a combination of inclined loading or overturning) as free water at tip level may lead to necking of the soil plug during uplift and near instantaneous loss of suction under the top plate (Mana et al. 2011c). In this paper, the potential of gap arrestors in improving the vertical uplift capacity of shallow skirted foundations is explored.

## DRUM CENTRIFUGE TESTS

All the model tests were conducted in the drum centrifuge facility at the University of Western Australia (UWA) (Stewart et al., 1998). The centrifuge has mainly two parts – an annular channel and a central tool table on which the actuator is mounted. It has a diameter of 1.2 m, channel depth of 0.2 m and channel height of 0.3 m. The signals from the centrifuge are transmitted to the control room and vice versa through high-speed wireless systems (Gaudin et al., 2009). The signals include those for controlling the channel and the actuator, data from the load cell and transducers connected to the models and images from the cameras set inside the centrifuge. All testing reported here was carried out at an acceleration level of 200 *g*, with reference to the mid-depth of the sample.

*Foundation model*

Foundation models of diameter 60 mm, representing a prototype foundation diameter of 12 m at 200 *g*, were used for this study. Two different foundations with skirt depth (d) to diameter (D) ratios of 0.1 and 0.2 were tested The models had a skirt thickness $t_{wall}$ of 0.5 mm, giving a thickness to diameter ratio $t_{wall}/D = 0.008$. **Figure 1**(a) and (b) shows respectively the side elevation of foundation with embedment ratio 0.1 with and with out the gap arrestor and **Figure 1**(c) shows the bottom plan view of the same foundation.

A flexible latex membrane was used to craft the gap arrestor. In order to prevent the membrane folding and creasing during installation, a thin cellulose acetate sheet was glued on the upper face of the latex. The acetate sheet was trimmed to stop short of the foundation skirt to prevent the membrane becoming too stiff and moving with, rather than relative to, the foundation. The underside of the membrane was coated with sand to achieve a rough contact surface with the soil and to make the membrane heavy enough not to float in water. The gap arrestors used in this study were such as to project radially 0.3D from the foundation periphery and were positioned at the same level as the underside of the top plate. The size of the gap arrestor was chosen to be as large as possible while maintaining sufficient stiffness to achieve clean mating with the seabed (without creasing or folding).

The foundations were equipped with two pore pressure transducers (PPTs) and four total pressure transducers (TPTs). Two TPTs and one PPT were set flush with the underside of the foundation top plate to measure the total and pore pressures acting inside the skirt compartment (**Figure 1**(c)). One PPT was attached through a hollow cylindrical tube fixed along the skirt wall to measure the pore water pressures at the skirt tip, and used predominantly to assist with precise installation. Two TPTs were attached flush to the outer sides of the skirt wall, diagonally opposite to each other and close to the skirt tip. The TPTs in the skirts allowed verticality during installation and changes in radial stress during gap formation and subsequent foundation loading to be monitored.

The foundations were equipped with a pneumatically operated valve to enable egress of water from the skirt compartment during installation and sealing of the foundation following installation in-flight. The valve door is connected to a spring, which enables closing of the valve on application of high air pressure and opening on its release. This enabled the closing and opening of the valve when required, without stopping the centrifuge.

*Soil sample preparation and shear strength characterisation*

A lightly over consolidated clay was selected for the study to improve the likelihood of a vertical gap remaining stable once formed, since the ability of a gap to form and stand open increases with increase in the soil strength ratio, $s_u/\gamma'z$ (Chen 1975, Pastor 1978, Britto & Kusakabe, 1982;). Analytical solutions for the depth that an unsupported excavation can remain stable (i.e. remain open or upright) exist for plane strain and axisymmetric boundary conditions, expressed in terms of a stability ratio

$s_u/\gamma z$ (Chen, 1975; Pastor, 1978, Britto & Kusakabe, 1983). While neither case exactly replicates the boundary conditions of this study, the solutions were used to provide a guideline for the targeted shear strength. The theoretical solutions indicated that $s_u/\gamma z > 0.275$ for a plane strain cut or gap (Pastor, 1978) and more critical, 0.2 for an axisymmetric excavation with depth to diameter ratio relevant to this study (Britto & Kusakabe, 1982) For this study, a sample with $s_u/\gamma z$ ($s_u/\gamma' z$ in the case of the saturated, inundated samples in this study) > 0.275 was targeted, the highest of the theoretical predictions, to ensure the gaps would form and remain open.

For preparing the soil samples, commercially available kaolin powder was used. The properties of this kaolin are well established (Acosta-Martinez & Gourvenec, 2006; Chen & Randolph, 2007). It has a liquid limit, LL = 61 %; plastic limit, PL = 27 % and specific gravity, $G_s = 2.6$. The kaolin powder was mixed thoroughly for four to five hours in a vacuum mixer with 120 % (of its weight) of water under a vacuum of approximately 70 kPa. The resulting uniform slurry was then carefully poured in to the drum channel spinning at 30 *g* and allowed to consolidate at 200 *g*. Two layers of 6 mm thick geofabric were provided along the base of the sample and at different locations along the sides, to allow for the easy drainage of water during consolidation. Three top-ups of clay slurry were supplied during the consecutive days after first load, to achieve maximum possible sample height in the channel.

After the final top-up, the whole sample was allowed to consolidate at 200 *g* for four more days to obtain a normally consolidated (NC) sample. The full consolidation of the clay was ensured by conducting undrained T-bar tests (Stewart & Randolph, 1991, 1994). Once the clay was fully consolidated, the top 60 mm was scraped off to obtain a lightly over consolidated (LOC) soil sample. The scraping was performed by attaching a scraping plate to the central tool table, after stopping the centrifuge and then manually rotating it along the sample surface. The plate was gradually advanced in to the clay sample until the desired depth of the LOC sample was achieved. The LOC sample was then reconsolidated for one more day before performing the tests.

The shear strength of the soil was determined by conducting undrained T-bar tests at a rate of 1 mm/s. The chosen velocity resulted in the value of non-dimensional term $vD_{T\text{-}bar}/c_v$ above 60 (where v is the penetration rate, $D_{T\text{-}bar}$ is the diameter of the T-bar cylinder = 5 mm and $c_v$ is the coefficient of consolidation of the clay ~ 0.082 mm$^2$/s) ensuring undrained conditions (Finnie & Randolph, 1994). Two T-bar tests were carried out in the NC deposit and four T-bar tests were carried out in the LOC deposit. The tests were distributed spatially around the channel and temporally during the testing programme to give a fair representation of the shear strength profile of the sample.

A shear strength profile approaching proportionality with depth of the NC sample confirmed that the clay was close to fully consolidated, before the centrifuge was stopped to allow scraping of the upper clay. **Figure 2** shows the shear strength profile of the LOC sample in which the model tests were conducted. The shear strength from T-bar tests is generally calculated as the ratio of measured penetration resistance to a constant value of T-bar factor, $N_{T\text{-}bar}$ = 10.5, corresponding to full flow around the T-bar cylinder (Stewart & Randolph, 1994). The shear strength shown in **Figure 2** was calculated following the procedure proposed by White et al. (2010), adjusting $N_{T\text{-}bar}$ at shallow depths and correcting for buoyancy. The profile can be idealised using a straight line expression

$$s_u = s_{um} + kz \qquad (1)$$

where $s_{um}$ is the shear strength at the mudline, k is the rate of increase in shear strength with depth and z is the prototype depth at which the $s_u$ is measured. Using $s_{um}$

= 6 kPa and k = 0.875 kPa/m gave a representative value of the shear strengths measured in the centrifuge sample.

A profile of buoyant unit weight of the sample was determined from moisture content and unit weight measurements of specimens taken over the depth of the sample. The data indicated that buoyant unit weight increased linearly with depth defined by $\gamma'$ (kN/m$^3$) = 6.54 + 0.0315z. Values of $s_u/\gamma' z$ were estimated as 0.89 and 0.51 at skirt tip level for the foundations with embedment ratio d/D = 0.1 and 0.2 respectively, and therefore a gap may be expected to remain open based on theoretical solutions for excavation stability (Pastor, 1978; Britto & Kusakabe, 1982).

*Testing programme*

In total, 21 tests were carried out in a single channel sample. Undrained uplift to failure and two levels of sustained uplift, defined as a proportion of the peak undrained uplift capacity, were considered for each foundation embedment ratio, with an intact skirt-soil interface, with a gapped skirt-soil interface and with a gapped interface with a gap arrestor. Undrained compression tests were also carried out to compare the bearing capacity in compression with the peak uplift capacity. All the tests were carried out with both foundation models, with d/D = 0.1 and 0.2. A matrix of the testing schedule is presented in **Table 1**.

The foundations were installed in-flight (at 200 *g*) at a rate of 0.1 mm/s, with the drainage valve open to allow water to escape from inside the skirt compartment. Complete installation was confirmed through the sudden increase in the readings from the load cell and the PPT and TPTs underneath the top plate. Uniformity in the installation process between the tests was achieved by penetrating the foundation until the load cell readings reached similar magnitudes. For the tests with gap arrestors attached to the foundation, the gap arrestors were deployed along with the foundation.

After installation, the drainage valve in the foundation top plate was closed and a waiting period of five minutes (equivalent to prototype time of 138 days) was allowed before loading. A fixed waiting period was adopted to maintain a uniform time interval between installation and loading in all the tests and to allow a reasonable degree of consolidation of the soil around the skirt wall disturbed during installation. The degree of consolidation adjacent to the skirt wall was calculated from cylindrical cavity expansion solutions derived for consolidation around thin-walled piles and caissons (Randolph, 2003). Taking a representative coefficient of consolidation for swelling for kaolin $c_v$ = 0.24 mm/s (Randolph & Hope, 2004) and an equivalent diameter of the foundation of $D_{eq} = 2\sqrt{(Dt_{wall}/2)}$ (assuming that only about half of the soil displaced by the skirts is pushed outwards) gives a dimensionless time factor $T = c_v t/D_{eq}^2 = 1.2$ for a the waiting period t = 300 seconds (5 minutes). Cylindrical cavity expansion solutions for radial drainage indicate approximately 70 % excess pore pressure dissipation adjacent to the skirt wall at T = 1.2.

In the tests requiring generation of a gap, i.e. the 'gapped' tests and 'gap arrestor' tests, the gap was generated immediately after installation during the waiting period. The gaps were generated by moving the foundation horizontally (tangential to the drum channel) at a rate of 4 mm/s through 4 mm (in model scale, i.e. D/15) at the mudline. After the waiting period, the foundations were loaded in compression or uplift. All short-term loading tests were displacement controlled and conducted at the rate of 0.1 mm/s (v), giving the value of dimensionless term $vD/c_v$ around 70, sufficient for ensuring undrained loading conditions (Finnie & Randolph, 1994). The sustained uplift tests were carried out through load-control at loads equivalent to 10 and 40 % of the ultimate undrained uplift capacity. A limiting uplift velocity of 0.01 mm/s was assigned during the load controlled testing in order to prevent sudden pullout of the foundation from the soil once suction was lost.

## RESULTS

*Undrained capacity in compression and uplift with intact skirt-soil interface*

The net capacities in the tests with an intact skirt-soil interface are compared with available theoretical solutions (Martin 2001) and plotted against normalised foundation displacement relative to the installed depth in **Figure 3.** The responses from four tests are presented: undrained compression and uplift of foundations with d/D = 0.1 and 0.2. Net bearing resistance ($q_{net}$) was calculated from the measured bearing resistance ($q_m$), accounting for the correction in overburden pressure according to

$$q_{net} = q_m - \sigma'_{v0} + \frac{W'_{soilplug}}{A_{soilplug}} \qquad (2)$$

where $q_m$ is the measured load resistance divided by foundation cross-sectional area, $\sigma'_{v0}$ is the effective vertical overburden pressure at the skirt tip level, $W'_{soilplug}$ is the submerged weight of the soil plug inside the skirt compartment and $A_{soilplug}$ is the base area of the foundation.

Theoretical predictions of resistance from upper and lower bound solutions for a rough-sided circular skirted foundation in a deposit with shear strength heterogeneity factor, $kD/s_{um}$ = 2 (Martin, 2001), close to the value of $kD/s_{um}$ for the soil sample in which the tests were conducted are plotted for comparison. The theoretical resistance was calculated as the product of the limit value of bearing capacity factor for the initial foundation embedment depth ratio and the local shear strength at current skirt tip level (i.e. the original shear strength at the depth corresponding to current skirt tip level, as shown in **Figure 2**).

In compression, it can be seen that the observed net resistance lies in between the theoretical limits for both the foundations. Comparison of the observed net peak uplift resistance and the steady state compression resistance indicates that near-full reverse end bearing was mobilised for the foundation with d/D = 0.2 (92 % of the compression capacity), but less so for d/D = 0.1 (77 % of the compression capacity). Peak uplift resistance for the foundation with d/D = 0.2 was mobilised at an equivalent displacement to that at which steady state compression resistance was achieved. In contrast, peak uplift resistance for the foundation with d/D = 0.1 was mobilised at displacement of around two thirds of that corresponding to steady state compression resistance, which is consistent with full reverse end bearing not being achieved.

Failure to mobilise full reverse end bearing for d/D = 0.1 is not consistent compared with previous results in the same centrifuge with the same models in essentially similar deposit that have shown full reverse end bearing mobilised (Mana et al., 2012c). A second undrained uplift test was carried out with the foundation with embedment ratio d/D = 0.1 but an identical result to the first test, as shown in **Figure 3**, was observed.

The reduced peak uplift resistance observed may result partly from overloading during the jacked installation, leading to some remoulding of the soil - a characteristic that has been observed in previous tests (Mana et al., 2012c). The reduced peak uplift resistance may also be partly due to vibration of the tool table leading to minor cyclic loading of the foundation and a weaker reverse end bearing failure mechanism. A slightly lower soil shear strength from T-bar tests and lower installation resistance of both the foundations were observed in this suite of tests compared to a previous suite of tests (Mana et al., 2012c), despite similarity in procedures, which supports the postulation of vibration. The effect of constant amplitude and frequency of vibration of the tool table would be expected to have the most significant effect in uplift and for low foundation embedment ratios due to lower strength and stiffness of the zone of soil involved in the failure mechanism, as is supported by the experimental data. The measured unbalance of the centrifuge channel during testing was within normal tolerance, with fluctuations in acceleration maintained below 0.30 *g*, but the resolution of this unbalance signal was insufficient to capture the vibration caused by slightly damaged bearings. The bearings of the drum centrifuge were replaced shortly after this suite of centrifuge tests were carried out. T-bar data in normally consolidated deposits have since shown increased shear strength compared to that measured in this suite of tests, and also consistent with profiles measured in previous testing programmes.

The purpose of the centrifuge tests was to compare the effect of the skirt-soil interface condition on uplift resistance. Since repeatability of the uplift response of the intact interface was confirmed by the duplicate test, it was considered that the reduced peak value of reverse end bearing would not adversely affect comparison of the effect of the interface conditions on foundation response.

*Effect of gap and gap arrestors on short-term uplift response*

The installation and uplift resistance measured by the load cell, for foundations with an intact interface, gapped interface and gapped interface with gap arrestor, are shown in **Figure 4** and **Figure 5** for foundation embedment ratios of 0.1 and 0.2 respectively. The measured installation resistance was similar in the different tests, although slightly higher installation resistance was observed for the gap arrestor tests due to the gap arrestor touching the soil surface in advance of full penetration of the skirts.

The presence of a gap along the skirt-soil interface (in the absence of a gap arrestor) led to faster loss of suction during uplift, indicated by the dramatic loss of uplift resistance at low foundation displacement. The gap arrestors were clearly effective in their intended function of retarding the immediate loss of suction due to the presence of a gap at the skirt-soil interface. It is interesting to note that for both cases with a gapped skirt-soil interface, suction was lost at a displacement equivalent to 10 % of the respective skirt depth. The relative foundation displacement, given in terms of foundation diameter and skirt depth, at which suction was lost for each foundation and skirt-soil interface condition is summarised in **Table 2**.

The presence of a maintained gap during the tests with a gap arrestor was verified from readings from the total pressure transducers on the outer face of the skirt wall over the test duration. **Figure 6** compares the TPT data from tests with a gapped skirt interface, with and without a gap arrestor for the foundation with d/D = 0.2. The sharply reducing and then constant readings of TPT3 on the active (gap) side and the sharply increasing and then constant readings of TPT4 on the passive side of the foundation indicate that a gap was generated and maintained throughout the test. Since it is clear that a gap was developed between the skirt and soil, the improved suction capacity can be attributed to the gap arrestor retarding the drainage of water into the gap.

A comparison of the pore pressures developed at the underside of the top plate of the foundations with an intact skirt-soil interface, gapped interface and gapped interface with gap arrestor is shown in **Figure 7**. A time history of pore pressures and the pore pressure-displacement responses throughout the entire tests are shown in **Figure 7**a and b respectively. The development of negative pore pressure during uplift in all the three tests is evident, reflecting the trend of the load cell readings shown in **Figure 4**, and indicating that loss of uplift resistance is caused by the loss of suction beneath the foundation top plate.

*Sustained loading*

**Figure 8** and **Figure 9** show the time-displacement responses of foundations with d/D = 0.1 and 0.2 respectively, with an intact skirt-soil interface, gapped interface and gapped interface with gap arrestor, subjected to sustained loads equivalent to 10 % and 40 % of the ultimate undrained uplift capacity, $q_u$, of the foundation with an intact interface. Figures 8(a) and 9(a) show the displacement history over the full period of the test while Figures 8(b) and 9(b) show the detail of the displacement history over the first equivalent prototype year of loading. With one exception (d/D = 0.1, $q/q_u$ = 0.1), the tests were continued to a maximum of 10 years or until just beyond the indicated points where suction was lost.

Initially an 'immediate', rapid foundation displacement was observed resulting from the application of the load. The magnitude of the immediate displacement was a function of the applied load and foundation embedment ratio, and in these tests ranged from 0.1 % to 0.5 % of the foundation diameter. Following the initial rapid displacement, the rate of displacement slowed, governed by time-dependent displacements due to seepage of water into the skirt compartment. The rate of displacement observed was a function of the magnitude of applied load, the foundation embedment ratio and the skirt-soil interface condition. An approximately linear relationship between displacement and duration of loading is evident in 9 out of the 12 tests. For the lower level of vertical load considered, half the tests exhibited a change in rate of displacement within the first year or two of loading, the rate then remained constant until suction was lost or the test was stopped. This change in rate of displacement was not found to be related to a particular skirt-soil interface condition, with each of the three bi-linear results corresponding to a different skirt-soil interface condition.

Higher rates of displacement were observed due to the presence of a gap compared with an intact skirt-soil interface or a gapped interface with arrestor, for each embedment ratio at each load level. The gap arrestor restricted the rate of displacement of the foundation compared with a gapped skirt-soil interface, in some cases to close to that of the foundation with an intact skirt-soil interface. The presence of the gap arrestor also led to displacements of a similar magnitude to those of the foundation with an intact skirt-soil interface to be resisted before suction was lost.

**Table 3** summarises the total relative displacement and duration of sustained uplift at which suction was lost beneath the foundation top plate. At the lower relative load level, $0.1q_u$, suction was preserved for the duration of the tests for the foundations with an intact skirt-soil interface and with a gapped interface with an arrestor for both foundation embedment ratios. In the case of a gapped skirt-soil interface, suction was still maintained for several years: 3.6 years for $d/D = 0.1$ and 8.4 years for $d/D = 0.2$. At the higher load level, $0.4q_u$, suction was lost in all tests. However, the presence of the gap arrestor at least doubled the duration over which suctions were maintained, and provided performance that was essentially equivalent to the foundation with an intact skirt-soil interface. The gap arrestor therefore limited the rate of displacement and sustained suction by preventing drainage of water into the gap.

It is noteworthy that immediate displacement contributed as little as 1.5 % and as much as 25 % to the maximum total displacement that occurred in the foundations with the gapped skirt-soil interface. The proportion of immediate to maximum total displacement was greater at the higher load level and for lower embedment ratio as might be expected.

## CONCLUSIONS

This paper has reported results from an investigation into an approach for reducing the adverse effect of gapping at the skirt-soil interface on the undrained and sustained uplift capacity of shallow skirted foundations. This has become an area of increasing importance due to deep water oil and gas developments, where shallow skirted foundations may be subjected to uplift from overturning moments.

Centrifuge model tests were performed to assess the potential improvement in undrained and sustained vertical uplift response of shallow skirted foundations due to provision of a flexible mat ‘gap arrestor’ around the periphery of the foundation. It was observed that the gap arrestor enabled suction to be maintained for larger foundation displacements under short-term and sustained uplift; under sustained uplift the rate of displacement was reduced and suction was lost at larger foundation displacements when compared to the case of a gapped skirt-soil interface with no gap arrestor. In many cases, the presence of the gap arrestor resulted in a foundation with a gapped skirt-soil interface exhibiting essentially identical resistance to a foundation with an intact skirt-soil interface.

Reverse end bearing potential of shallow skirted foundations is widely acknowledged, but the presence of a crack or gap down the skirt-soil interface may reduce that capacity considerably. The results presented here are promising and demonstrate high potential for the use of gap arrestors to prevent the adverse effects of gapping on the undrained and sustained uplift capacity of shallow skirted foundations.

## ACKOWLEDGEMENTS

The work described in this paper forms part of the activities of the Centre for Offshore Foundation Systems, currently supported as a node of the Australian Research Council Centre of Excellence for Geotechnical Science and Engineering and in partnership with The Lloyd's Register Educational Trust. The work presented in this paper was supported through ARC grant DP0988904. This support is gratefully acknowledged. The technical support provided by drum centrifuge technician, Mr. Bart Thompson, electronics support by Mr. Philip Hortin and Ms. Khin Seint and mechanical support by Mr. David Jones is also gratefully acknowledged.

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## TABLES

Table 1 Matrix of testing schedule; all tests carried out for foundations with d/D = 0.1 and 0.2

| Loading | Skirt-soil interface condition | | |
|---|---|---|---|
| | Intact | Gapped | Gapped with gap arrestor |
| Undrained compression | X | -- | -- |
| Undrained uplift ($q_u$) | X | X | X |
| Sustained uplift, $0.1q_u$ | X | X | X |
| Sustained uplift, $0.4q_u$ | X | X | X |

Table 2 Relative displacement during undrained uplift at which suction pressures lost

| | Skirt-soil interface condition | | |
|---|---|---|---|
| d/D | Intact | Gapped | Gapped with gap arrestor |
| 0.1 | 0.04D | 0.01D | 0.03D |
| | 0.4d | 0.1d | 0.3d |
| 0.2 | 0.08D | 0.02D | 0.05D |
| | 0.4d | 0.1d | 0.25d |

Table 3 Relative total displacement and duration of sustained uplift at which suction pressures lost

| Uplift pressure | Skirt-soil interface condition | | |
|---|---|---|---|
| applied, q | Intact | Gapped | Gapped with gap arrestor |
| d/D = 0.1 | | | |
| $0.1q_u$ | N.A.* | 0.015D | N.A. |
| | > 4 yrs | 3.6 yrs | > 10 yrs |
| $0.4q_u$ | 0.028D | 0.013D | 0.024D |
| | 1 yr | 0.3 yrs | 0.9 yrs |
| d/D = 0.2 | | | |
| $0.1q_u$ | N.A. | 0.068D | N.A. |
| | > 10 yrs | 8.4 yrs | > 10 yrs |
| $0.4q_u$ | 0.086D | 0.055D | 0.072D |
| | 1.7 yrs | 0.80 yrs | 1.5 yrs |

*N.A. indicates suction was not lost during the test

FIGURES

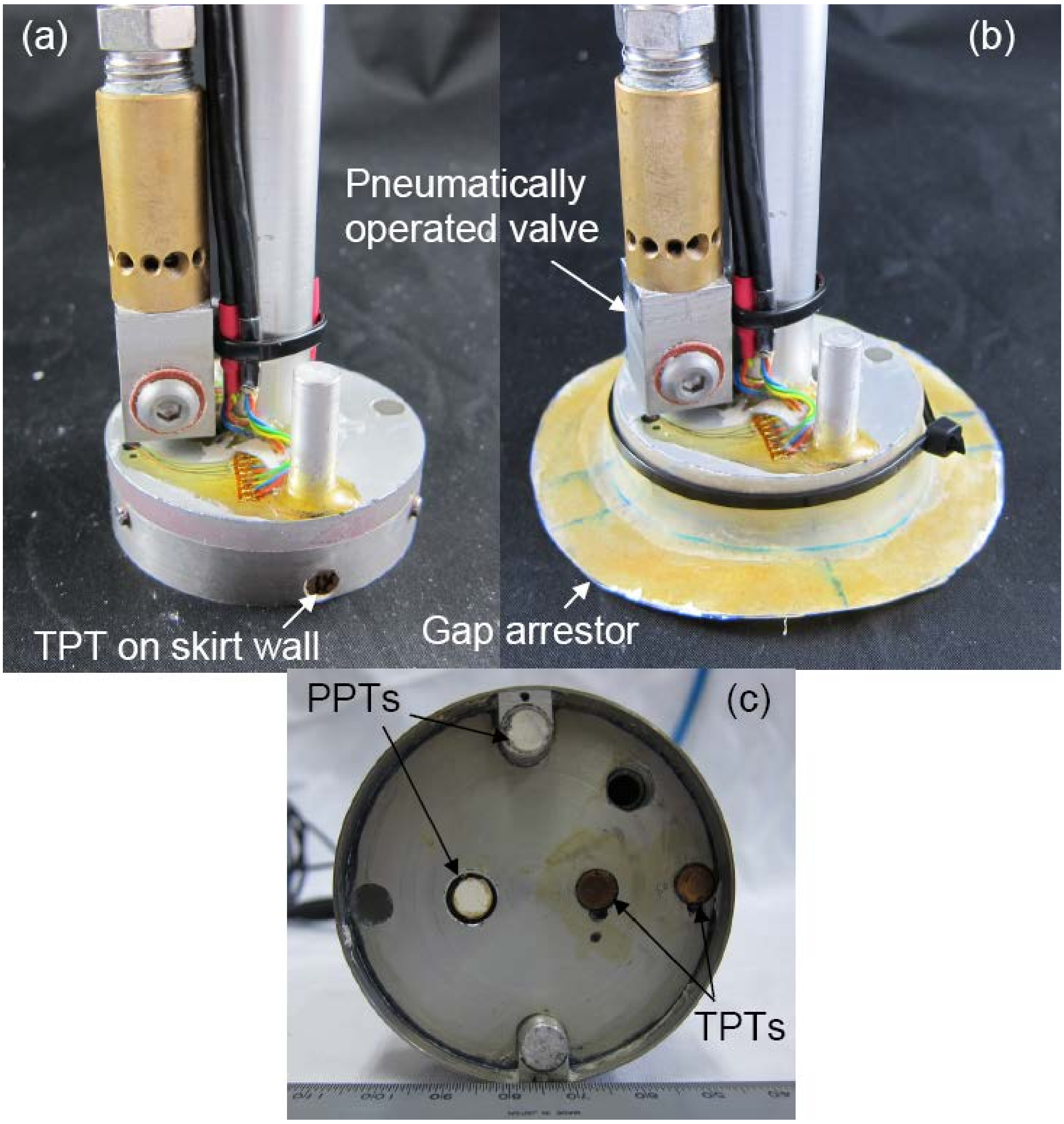



Figure 1. Foundation model with d/D = 0.1 (a) side elevation of foundation models without gap arrestor and (b) with gap arrestor, and (c) bottom plan of skirt compartment

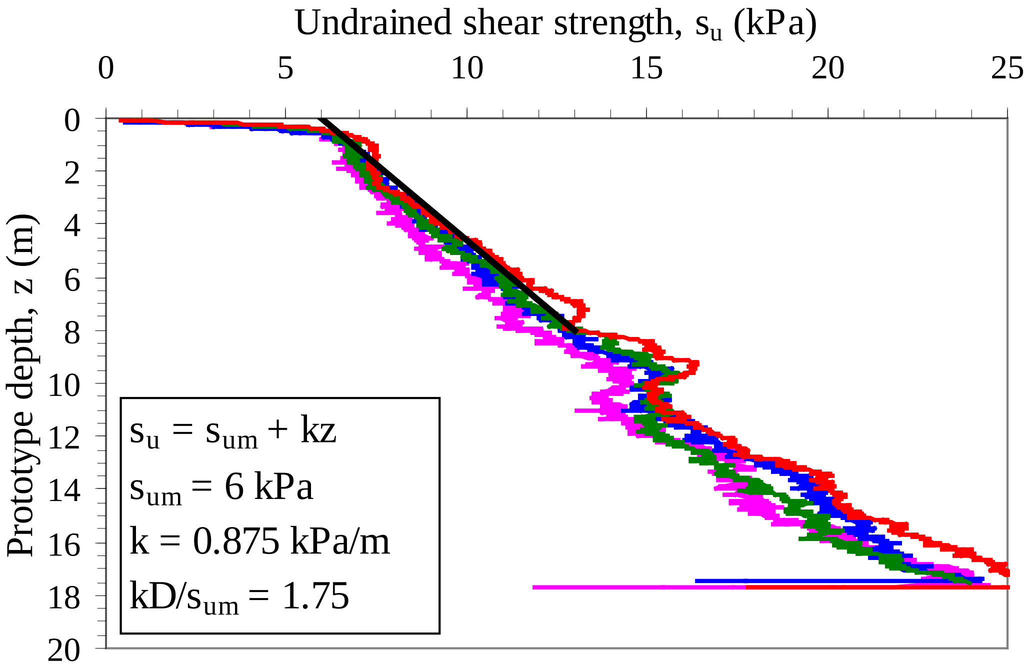



Figure 2. Undrained shear strength of the LOC sample corrected for buoyancy and shallow embedment

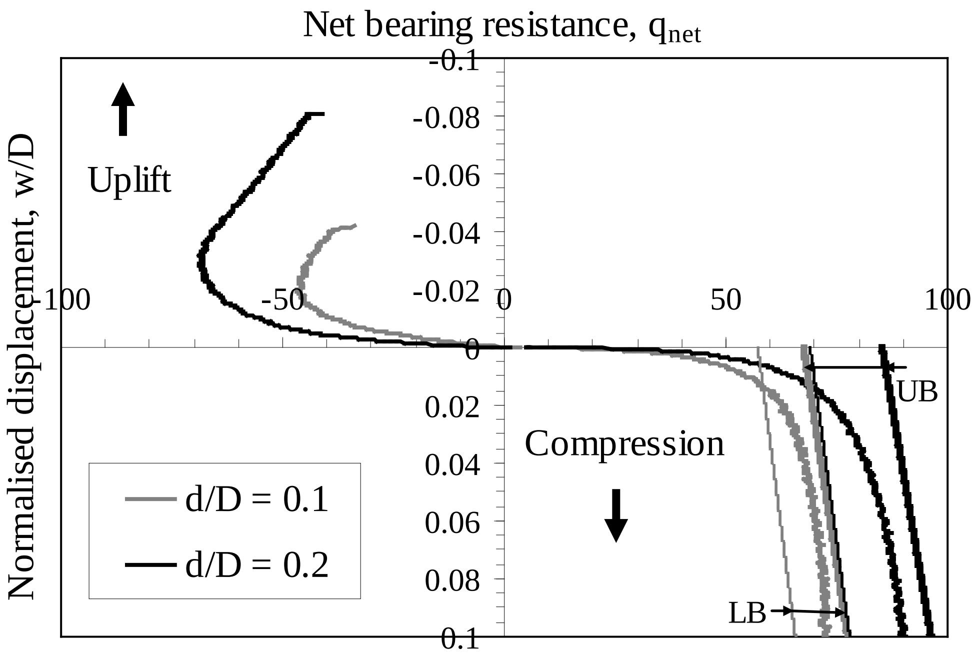



Figure 3. Net bearing resistance for foundations with embedment ratios 0.1 and 0.2 (w is the foundation displacement past installation)

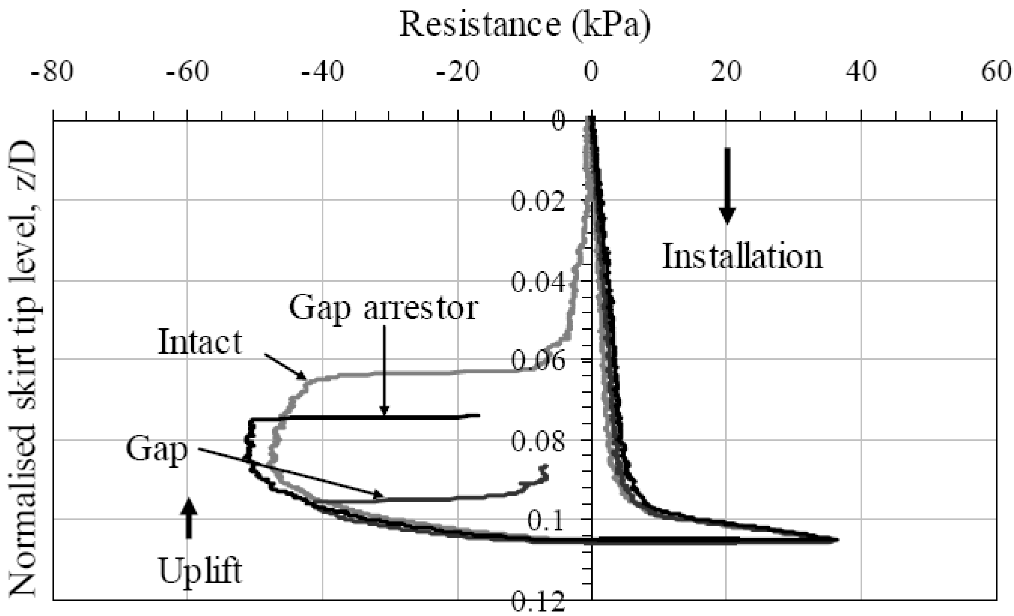



Figure 4. Installation and undrained uplift resistance measured using load cell of foundation with embedment ratio 0.1

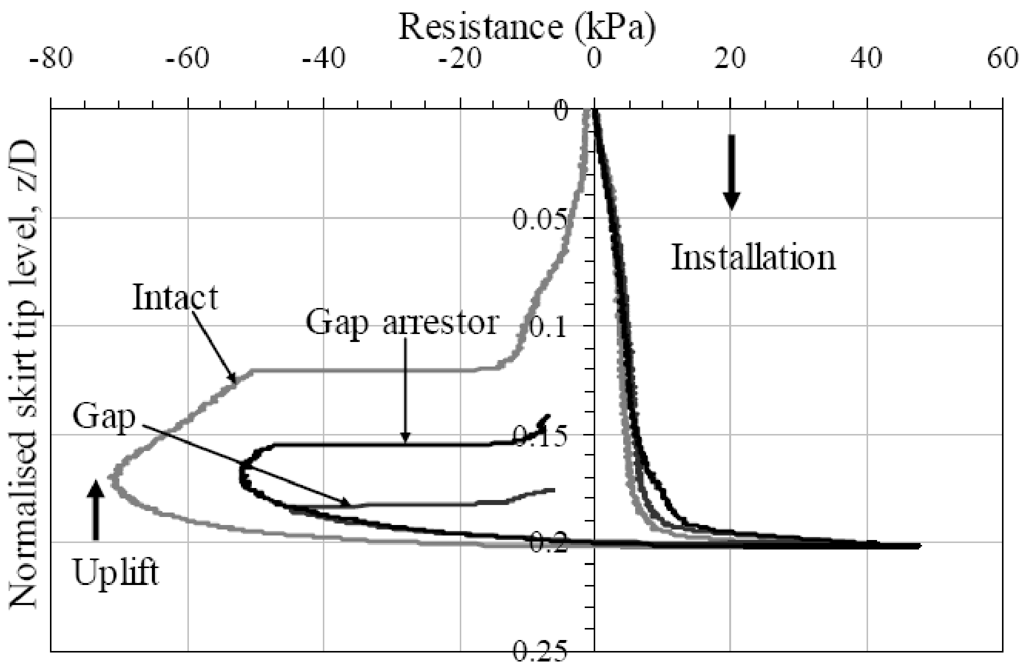
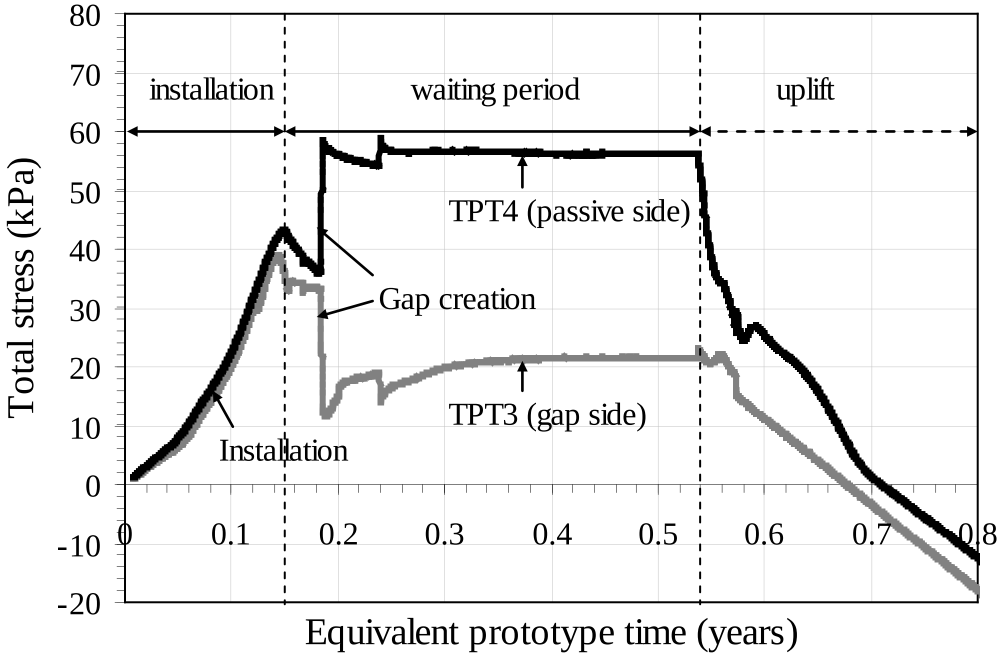
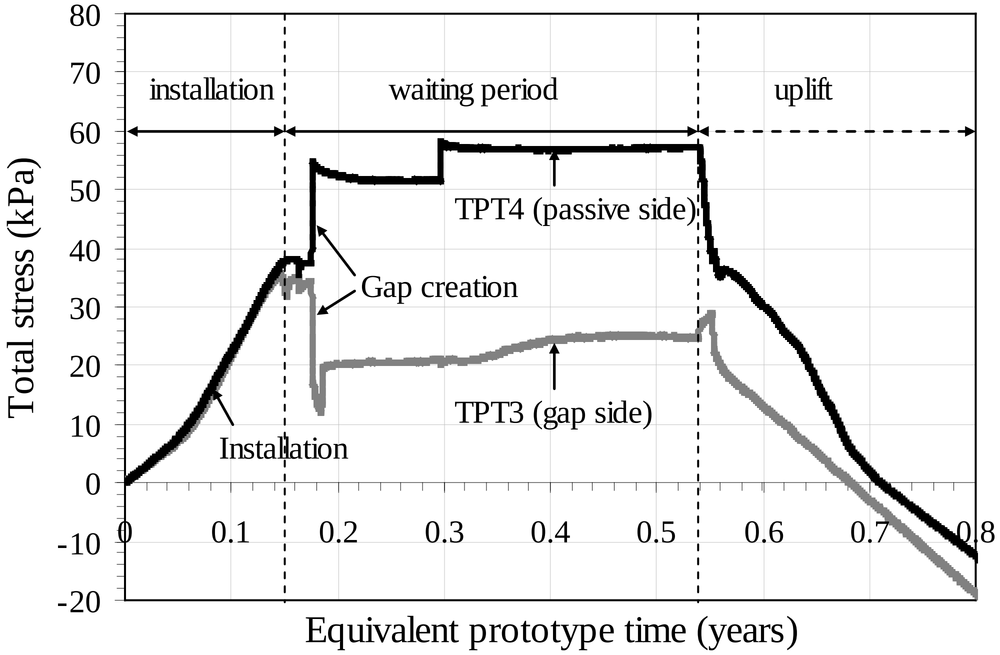



Figure 5. Installation and undrained uplift resistance measured using load cell of foundation with embedment ratio 0.2




(a) Gap arrestor




(b) Gap

Figure 6. Readings from the total pressure transducers on the outer skirt wall surface near the skirt tip during (a) gap arrestor test and (b) gap tests on d/D = 0.2

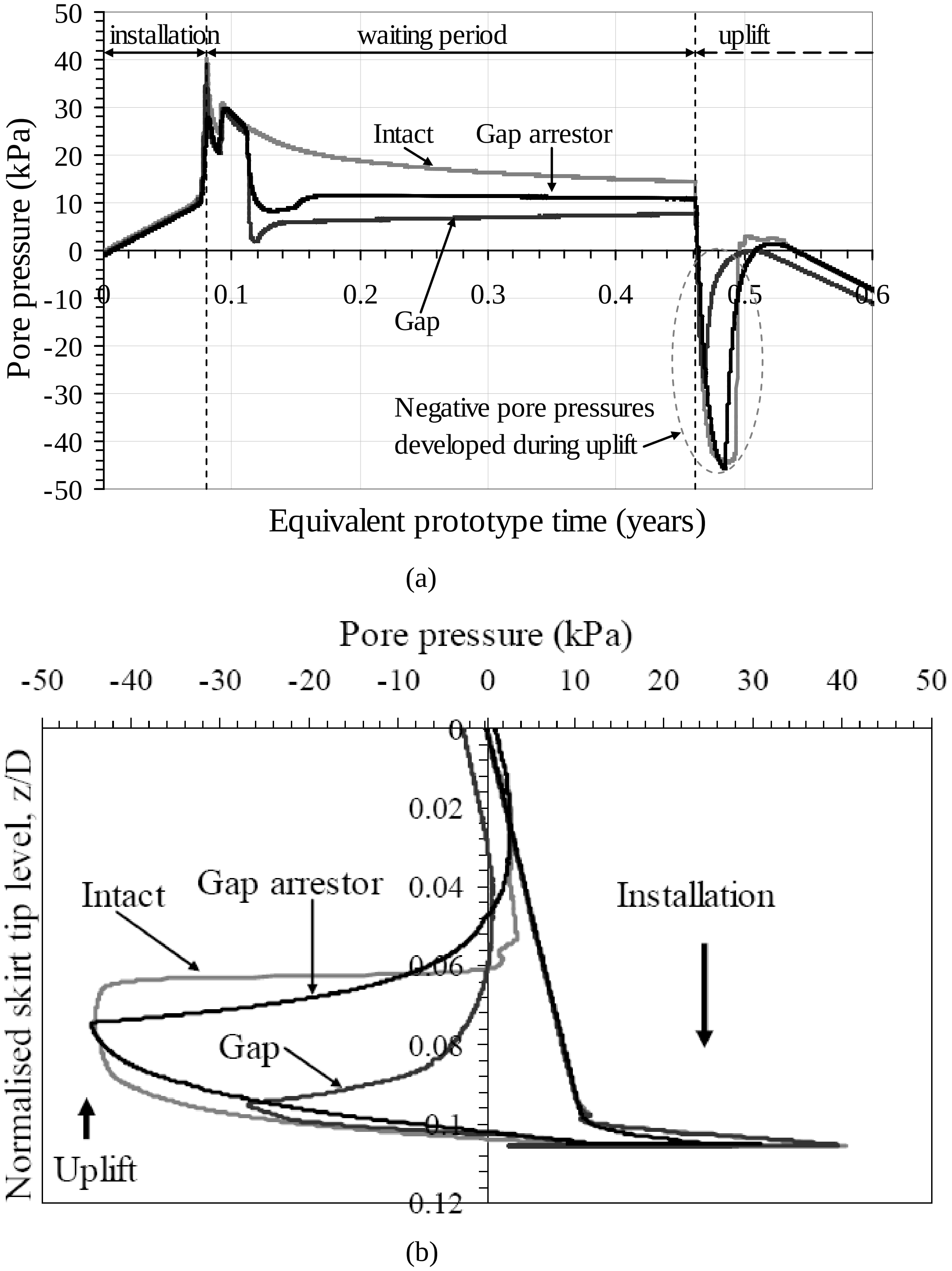
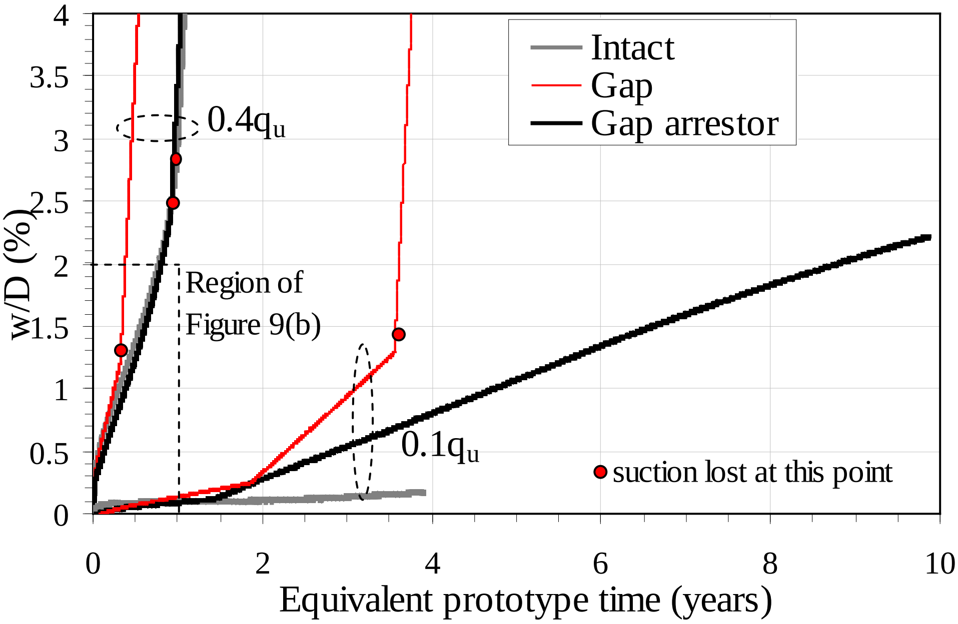
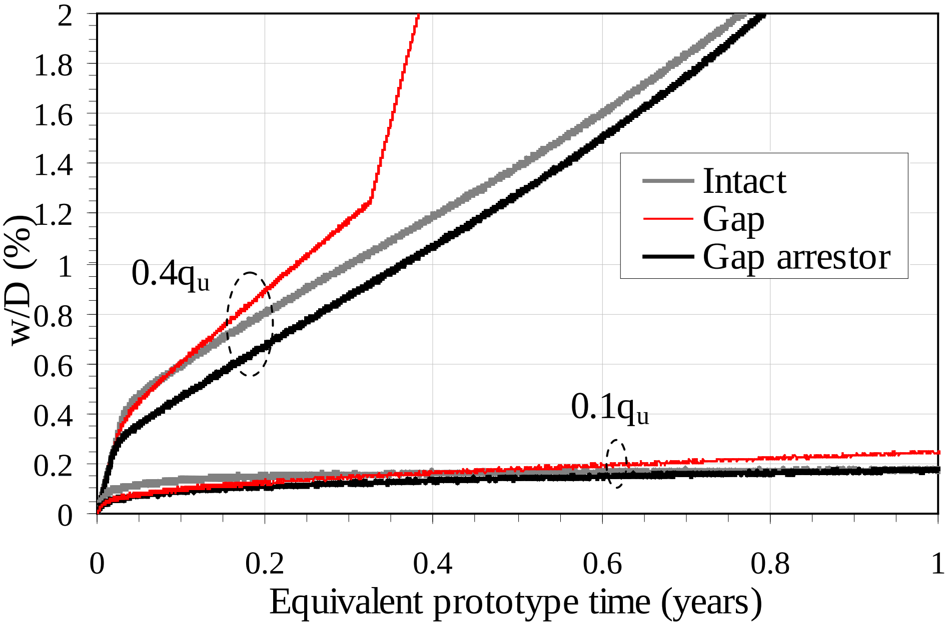



Figure 7. Readings from the pore pressure transducer at the underside of the top plate of foundation with d/D = 0.1 with an intact interface, gapped interface and gapped interface with gap arrestor plotted against (a) testing time and (b) foundation displacement

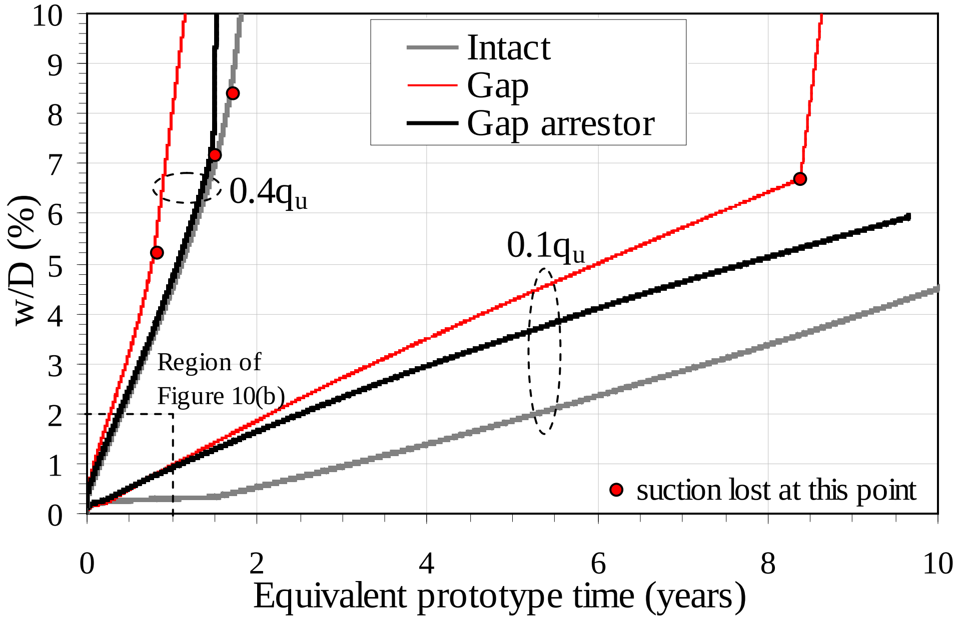



(a)

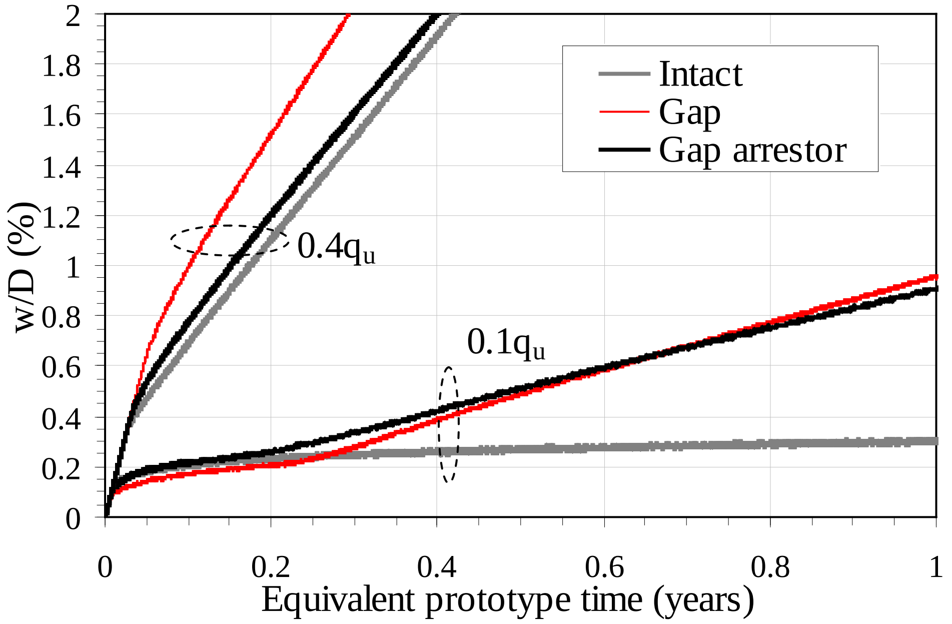



(b)

Figure 8. Time history of total foundation displacement for d/D = 0.1 with an intact interface, gapped interface and gapped interface with gap arrestor under sustained loading(a) over the full duration of loading and (b)over the first year of loading.




(a)




(b)

Figure 9. Time history of total foundation displacement for d/D = 0.2 with an intact interface, gapped interface and gapped interface with gap arrestor under sustained loading (a) over the full duration of loading and (b) over the first year of loading.