The effect of perforations on the uplift capacity of skirted foundations on clay

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Abstract: The retrieval of deep water subsea installations resting on soft soil, such as "mudmat" shallow foundations, can be a difficult and costly operation if significant resistance to uplift is experienced. At the mudmat invert, suctions may develop, increasing the uplift resistance to greater than the weight of the mat. In this paper, a series of centrifuge model tests are performed to determine the uplift resistance of rectangular mudmats resting on lightly over-consolidated kaolin clay. The study investigates the influence of perforation, in combination with skirt length and eccentric uplift, on the uplift resistance and suction generation at the foundation invert. The outcomes demonstrate that the central and eccentric uplift of mudmats have different failure mechanisms, resulting in a different distribution of excess pore pressure at the foundation invert. In contrast, perforations do not change the failure mechanism and only alter the magnitude of suction generated. The two different configurations of perforation investigated significantly reduce the suction at the mat invert and the uplift resistance, and may potentially shorten the operating time for centred uplift. The combination of perforation and eccentric uplift has the most beneficial effect on the reduction of the uplift resistance.

Key words: centrifuge modelling; mudmat; clay; perforation; uplift resistance; suction

1. INTRODUCTION

Mudmats are a type of shallow raft foundation used to support various temporary and semi-permanent subsea structures, such as Pipeline End Manifolds and Terminations (PLEMs/PLETs). They are an easily installed and economical solution commonly used in deep water oil and gas developments. In order to provide sufficient resistance to withstand horizontal loads from the thermal expansion of pipelines and jumpers, mudmats are usually designed with skirts, which are embedded into the seabed by a fraction of the mudmat width. Upon completion of the project, and in some instances to comply with environmental regulations, mudmats must be decommissioned and removed from the seabed.

The standard removal procedure for mudmats is to attach cables to the load points on the structures, these cables are then pulled by a lift vessel at sea level to extract the mudmat from the seabed. The uplift forces required for removal from the seabed are resisted not only by the self-weight of the submerged mudmat, but also by the suction forces potentially developing at the mudmat invert. In soft soil with low permeability, such as the clays or silts commonly encountered in deep waters, these suction forces can be equal to twice the submerged weight of the mudmat (Bouwmeester et al., 2009). In extreme cases, the suction forces may be greater than the lifting capacity of the vessel and lead to hazards during removal (Reid, 2007).

Various mitigation measures to reduce the generation of negative pressures (or suction) at mudmat inverts have been investigated, using both in-situ data and laboratory experiments. It was expected that perforations would limit the development of suction at the mudmat invert by shortening the drainage path. Lieng and Bjorgen (1995) reported that even a small perforation (with respect to the total mudmat area) can lead to a significant reduction in peak uplift resistance. During field trials, a reduction of about 50% of the uplift resistance was observed for a perforation ratio of 3.1% (defined as the plan area of perforating holes with respect to the total area). White et al. (2005) demonstrated that a large number of small perforations were more efficient in reducing the uplift ratio than a small number of large perforations. Their results can be used to maximize the ratio of vertical compression to uplift resistance. An alternative mitigation solution involves applying the uplift load with an eccentric movement to facilitate breakaway at the mudmat invert and hence reduce the magnitude of the suction forces generated. From small scale model tests, Reid (2007) reported a reduction up to 66% (compared to the centred uplift resistance) by applying the pull-out load at the edge of the mudmat. Water jetting at the invert is also a proven method to reduce uplift forces for offshore jack-up rigs embedded foundations, as demonstrated by Gaudin et al. (2011). However, the logistics associated with the jetting method are significantly more complex and costly than typical lifting devices.

Chen et al. (2012) presented a comprehensive investigation of the uplift resistance of mudmats, combining the effects of eccentric uplift, loading rate and skirt length in a model test programme performed in a geotechnical centrifuge. Chen et al. (2012) demonstrated that the uplift resistance was directly correlated to the development of suction at the mat invert and that fully undrained conditions (characterised by a full reverse end bearing mechanism) were achieved at normalised uplift velocities three orders of magnitude higher than those usually considered for shallow foundations in compression. This is because a suction relief mechanism develops at the foundation-soil interface during uplift, but while the system is in compression, the pore pressure dissipation mechanism is governed by pore pressures in the far field (Lehane et al., 2008). In contrast, fully drained conditions that would lead to low uplift resistance require uplift rates too slow to be practically undertaken in-situ and partially drained conditions may be prevalent during uplift of prototype mudmats. Accordingly, the prediction of uplift resistance is hindered by difficulties in assessing the relevant drainage conditions and the associated bearing capacity factor. Additional results from Chen et al. (2012), associated with eccentric uplift, indicated that a different failure mechanism was taking place, favouring the suction relief mechanism and hence contributing to a significant reduction in the uplift resistance.

This paper presents a series of model mudmat tests performed in a geotechnical drum centrifuge. The research aims to advance Chen et al. (2012)'s study by linking perforation and uplift eccentricity in order to (i) further understand the mechanism governing suction development at the invert of a perforated mudmat, and (ii) provide recommendations to optimise a retrieval strategy in order to minimise the uplift resistance and the associated risk and cost. In particular, the generation of suction at the mat invert and the uplift force vs. displacement curves were monitored during centrifuge model tests, and were considered as a function of the effective width, the mudmat skirt length and the uplift eccentricity.

2. DETERMINATION OF THE UPLIFT CAPACITY

As detailed in Chen et al. (2012), the ultimate uplift resistance of mudmats is controlled by the operative shear strength of the soil and the failure mechanism during uplift. The failure mechanism can be assumed to be either a reverse end bearing type (Craig and Chua, 1990; Acosta-Martinez et al., 2008; Gourvenec et al., 2009; Randolph et al., 2011; Mana et al., 2012) or a breakout hemispherical type (Yu, 2000; Rattley, 2007) depending on the level of suction mobilised at the mat invert. Following the compression convention, the uplift capacity of mudmats in clay can be expressed as

[1] $$q_u = N_c s_{uop} - \gamma' h$$

where N_c is the bearing capacity factor, s_{uop} the operative shear strength of the soil at the skirt tips, γ' the submerged unit weight of the soil and h the skirt length, which accounts for the embedment of the foundation. The second term on the right hand side of the equation is the correction for overburden. For skirted foundations, the overburden stress is cancelled by the weight of soil column incorporated by the skirts (see Fig. 1). Therefore, the uplift capacity of a skirted mudmat, regardless of the failure mechanism, can be determined by

[2] $$q_u = N_c s_{uop}$$

Rigorous solutions to determine the bearing capacity factor of a strip footing on homogeneous clay under vertical loading have been developed by Prandtl (1921) and Reissner (1924) and yielded a value of N_c = 5.14. In non-homogeneous soil, N_c increases with the soil heterogeneity kB_0/s_{um}, where B_0 is the width of the footing, k the gradient of the soil profile and s_{um} the initial soil undrained strength at the mudline (Davis and Booker, 1973; Randolph et al., 2004), as illustrated in Fig. 1. The full reverse end bearing capacity can be assessed using solutions derived from the undrained compression capacity (eq. [2]), since for fully undrained conditions, uplift and compression capacities are theoretically equal.

The ultimate bearing capacity of two or more parallel strips has also received attention from Martin and Hazell (2005), Gourvenec and Steinepreis (2007) and Bransby et al. (2010), providing insights into the effect of perforations on the bearing capacity of shallow foundations under undrained soil conditions.

The operative shear strength of the soil, s_{uop}, is taken as the shear strength at the skirt tip, s_{uo}, which can be determined using the standard T-bar test (Stewart and Randolph, 1991; 1994), potentially enhanced by soil strain rate effects. Einav and Randolph (2005) and Lehane et al. (2009), among others, reported that the soil strength increases with strain rate by approximately 5%-20% per log cycle of increasing strain rate. This can be expressed as

[3] $$s_{uop} = s_{u0,ref}\left[1 + \mu \log\left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right)\right]$$

where $s_{u0,ref}$ is the soil shear strength at a reference strain rate $\dot{\gamma}_{ref}$ (which can be taken as s_{u0} from the T-bar test) of 0.0001 s^{-1}, and μ is a rate parameter of approximately 0.1 for normally consolidated kaolin clay (Randolph et al., 2005). Atkinson (2000) suggested that the average operational strain rate underneath a rectangular shallow foundation subjected to vertical loading can be approximated as $v/3B_0$ (where v is uplift velocity and B_0 is the width of the mat). Assuming that full contact is maintained between the foundation and the soil during uplift, a similar approach may be assumed for the present scenario.

The uplift force during model tests can therefore be expressed as

[4] $$F_{up} = N_c s_{uop} A + G'$$

where F_{up} represents the peak uplift force and G' the submerged self-weight of the mudmat. Note that in the present study, the gross area A is used to calculate uplift force regardless of the configuration of perforations.

3. EXPERIMENTAL SET-UP

3.1 Facility

The drum centrifuge at Centre for Offshore Foundation Systems (COFS), The University of Western Australia (UWA) was used to carry out the described tests, as it enables multiple mudmat uplift tests to be conducted in one single soil sample. The ring channel of the centrifuge has an outer diameter of 1.2 m, an inner diameter of 0.8 m and a channel height (sample width) of 0.3 m. A servo-controlled actuator was mounted on the central tool table to provide both vertical and radial movements. The tool table can be coupled to the channel or may rotate independently of it, allowing it to be stopped for examination or changing the tool, without affecting the soil sample. A complete technical description of this centrifuge is presented in Stewart et al. (1998). Tests were performed at a centrifuge acceleration of 150g, i.e. all model linear dimensions are scaled by 150 and all loads by 150^2 (see Garnier et al., 2007 for details on similitude principles).

3.2 Model configurations, instrumentation and calculation of effective widths

Three types of model mudmats were fabricated using aluminium plates, with dimensions of 5 mm in thickness (d), 100 mm in length (L_0) and 50 mm in width (B_0). This represents a prototype mudmat 15 m long and 7.5 m wide. The overall dimensions are identical to models tested by Chen et al. (2012).

One non-perforated model (labelled B) and two types of perforated models (labelled P1 and P2) were considered (see Fig. 2). Model P1 featured large perforations with 36 circular holes 6.0 mm in diameter (Fig. 2b). The second model, P2, featured small perforations, comprising 171 circular holes 2.7 mm in diameter (Fig. 2c). Both perforated models had the same perforation ratio, α, of 0.19, defined as the ratio of the area of the holes to the gross area. Each perforated model was made with removable skirts with a length (h) of 0 mm, 5 mm and 10 mm (0 m, 0.75 m and 1.5 m in embedment prototype, respectively), while the non-perforated mudmat models were fabricated with the same skirt lengths for benchmarking. Both model and prototype dimensions are summarised in Table 1.

The models were equipped with three Pore Pressure Transducers (PPTs), as illustrated in Fig. 2 and 3, to monitor variations in pore pressure at the foundation invert. As the PPTs' housing was too large to be fitted between perforations, they were installed in place of a single perforation as illustrated in Fig. 2b and 2c. In order to examine the effect of central and eccentric uplifts, three small holes (illustrated in Fig. 2a) were drilled to allow a vertical ball shaft to be screwed onto the model plate and connected to a loading cell by a tong, (illustrated in Fig. 3). Uplift was applied via the ball shaft and the uplift resistance was measured by a 500 N capacity load cell.

The effective width (W) of each model was defined to represent the average length of drainage paths between perforations (White et al., 2005). For mudmats with circular perforations, the effective strip width W was calculated as (see Figure 4):

[5] $$W = \frac{x + \sqrt{2}(x + d_0) - d_0}{2}$$

where d_0 is the diameter of circular perforations and x represents the shortest drainage path between the perforations, as illustrated in Fig. 4. From eq. [5], it was calculated that W is 6.07 mm (0.91 m in prototype) for mudmat P1 and 3.34 mm (0.5 m in prototype) for mudmat P2. For the non-perforated mudmat B, W is simply taken as the average of length and width, e.g. $(B_0 + L_0)/2$ = 75 mm (11.25 m in prototype).

3.3 Soil sample preparation and characterisation

Two soil samples were prepared for the present study. Kaolin slurry, prepared at a water content of ~ 120% (approximately twice the liquid limit), was poured into the centrifuge channel under an acceleration of 20g, over a preplaced 10 mm thick drainage blanket at the bottom. Self-weight consolidation under two-way drainage was achieved by spinning the centrifuge at 150g for approximately four days. The degree of consolidation was monitored by measuring pore pressure dissipation via PPTs located at the bottom of the channel and settlement of the top surface of the soil. After full consolidation was achieved, a soil layer 5 to 15 mm thick was scraped off the surface to create a lightly over-consolidated soil sample with a flat surface, enabling a good contact between the model and the soil. The final height of both samples was 150 mm (including the drainage layer).

T-bar tests were performed in both soil samples to evaluate the undrained shear strength. Tests were carried out by using a 5 mm diameter, 20 mm long T-bar at a standard penetration rate of 1 mm/s, ensuring undrained soil conditions (Stewart and Randolph 1991; 1994). As a first approximation, a constant bearing factor $N_{T\text{-}bar}$ = 10.5 derived from plastic solution (Randolph and Houlsby, 1984) and experimental calibration (Low et al., 2010) were adopted to convert the measured T-bar resistance into undrained shear strength. Following the procedure proposed by White et al. (2010), lower bearing factors were applied to characterise the T-bar penetration resistance at shallow depths, where full flow of soil around the T-bar cylinder cannot occur. A cyclic test was also included in each penetration test to obtain accurate calibration data for soil penetration resistance (Randolph et al., 2007).

Fig. 5 summarises the corrected undrained shear strength profiles at prototype scale in both soil samples. In general, soil strength profiles in both soil samples exhibited an excellent repeatability, with sample two featuring a more linear increase in strength with depth. The corrected soil strength for both samples can be idealised as bilinear profiles. At shallow depths (z < 0.75 – 0.6 m for samples one and two, respectively), the soil samples were over consolidated following the trimming process and exhibiteded a constant shear strength with depth, with values of s_u ~ 3.26 kPa and s_u ~ 1.68 kPa for samples one and two, respectively. Soil strength at higher depths can be idealised by linear profiles with gradients of k ~ 1.01 kPa/m for sample one and ~ 1.06 kPa/m for sample two, resulting in a heterogeneity ratio of kB_0/s_{um} ~ 2.3 and ~ 4.7, respectively.

3.4 Testing programme and procedure

Nine central uplift tests were performed in soil sample one and nine eccentric uplift tests were performed in sample two, both under a centrifuge acceleration level of 150g, as summarised in Table 2. Model mudmats were installed on the soil surface at 1g and consolidation under the weight of the foundation was achieved at 150g. A constant uplift velocity of v = 3 mm/s was applied to the model once all excess pore pressures at the mat invert were fully dissipated, indicating that full consolidation under self-weight had been achieved. A constant water table of 50 mm above the soil surface was maintained during each test.

4. TEST RESULTS

4.1 Typical measurements of uplift force and pore pressure

Typical central uplift load/displacement and excess pore pressure/displacement curves are presented for tests S1-1 and S1-4 in Fig. 6. The general patterns were consistent with Chen et al. (2012)'s observations that uplift resistance experienced a sudden increase to reach a peak value (F_{up}) over a short distance (w_p), then reduced to a semi-residual value which was slightly higher than the submerged self-weight (G') of the model mudmats due to the soil attached at the model invert. G' differed between tests due to the different skirt lengths and configurations of perforation (see Table 1). G' also changed slightly with uplift displacement due to the changing acceleration level along the radius in the centrifuge (see dashed line in Fig. 6), and this has been accounted for in the analysis.

The excess pore pressure displacement curves exhibit the same pattern as the load displacement curves, indicating a close correlation between pore pressure generation at the foundation invert and the uplift resistance. The negative values indicate the generation of suction at the mudmat invert, with peak values represented by p_1, p_2 and p_3 being coincident with the peak uplift resistance, indicating that uplift resistance is sustained by the development of suction at the mudmat invert. It is noteworthy that the uplift force for perforated mudmats, e.g. S1-4 and the associated suction at the mat invert is less sustainable compared to that for non-perforated mudmats, e.g. S1-1. This is attributed to shortening of the drainage path resulting from perforation and the associated acceleration in the dissipation of pore pressures. More details on the effects of perforations will be provided in the next section.

The peak values of the uplift forces (F_{up}), the peak value of the excess pore pressures monitored by the three PPTs (p_1, p_2 and p_3) and their average values $\overline{p}$ (= $(p_1 + p_2 + p_3)/3$), and the distance travelled to reach the peak uplift force (w_p) for all the eighteen uplift tests are summarised in Table 2 for further interpretation.

The distances required to reach peak uplift forces, $w_{p,}$ are presented in Fig. 7 as a function of the skirt length. The operational distance decreased with decreasing skirt length, regardless of the configuration of perforations, and both perforated mudmats exhibited a significantly lower operational distance during uplift compared to the non-perforated mudmat. Fig. 8 provides some insight into the secant stiffness (E_s) of the soil under vertical uplift, calculated as the normalised peak extraction resistance F_{up}/A divided by the normalised skirt displacement, w_p/B. Mudmats with perforations generated a stiffer response than non-perforated mudmats, while the stiffness for all mats was reduced with increased skirt length (Fig. 8). This occured because mudmats with perforation and shallower skirts generate a much shallower failure mechanism. As shown in Fig. 7, peak uplift force occurred faster for the perforated mudmats for the same skirt length (while uplifted at the same velocity), suggesting that the perforated design could be a promising method for saving uplift expenses by reducing operating time in the field.

4.2 Effect of perforation combined with skirt length

Fig. 9 presents the net peak uplift forces, $F_{up,net}$ ($= F_{up} - G'$), normalised by the gross area (i.e. $F_{up,net} / A$) and the corresponding peak values of average pore pressures ($\bar{p}$) varying with the effective width for central uplift tests. It is evident that the peak uplift force decreases with reducing effective width and shallower skirt embedment. The peak uplift forces for tests on the perforated mudmat (P1) were reduced by almost half compared to the non-perforated mudmat (B), indicating that the perforation had beneficial effects in reducing the uplift resistance of mudmats. For a same perforation ratio of 0.19, the reduction in effective width resulted in a further reduction of uplift resistance of about 30%. For a specific configuration of perforation, the reduction in skirt length resulted in a reduction of the uplift force by up to 50% for the largest effective width. This improvement significantly reduced, however, with reduced effective width. As anticipated, this reduction of peak uplift force was associated with a concomitant reduction in average peak suction, due to the shortening of the drainage paths by either perforations or decreased skirt embedment, which accelerated the dissipation of the negative pore pressure generated by the uplift mechanism.

The net peak uplift forces (normalised by gross area A) are also plotted against the associated average suctions in Fig. 10. Fig. 10 demonstrates that the uplift resistance was essentially sustained by the suction at the foundation invert, independent of the skirt length and the perforation. Consequently, the mudmat failure mode was a reverse end bearing failure mechanism (see illustration in Fig. 10), as observed and described by Craig and Chua (1990), Acosta-Martinez et al. (2008), Gourvenec et al. (2009), Randolph et al. (2011) and Mana et al. (2012) rather than a breakout contraction type mechanism (Yu 2000; Rattley 2007). This demonstrates that fully undrained conditions are experienced by the soil during uplift. Drainage conditions may be assessed by calculating the dimensionless velocity vB_0 / c_v (Finnie and Randolph 1994; Chung et al. 2006), where c_v is the coefficient of consolidation of the soil, typically equal to 1.5 m^2/year for kaolin clay at a stress level of about10 kPa (House et al. 2001).

In the present study, the dimensionless velocity for the non-perforated mudmat was about 3000, where undrained soil conditions for uplift can be assumed according to Chen et al. (2012). The dimensionless velocity for perforated mudmats was approximately one order less than for non-perforated mudmats if normalised by the effective width *W*, i.e. vW / c_v ~ 400 and ~ 200 for P1 and P2, respectively. This potentially lead to partially drained conditions within the soil that would explain the lower uplift capacity. This is however inconsistent with observations from Fig. 10, and will be discussed further in the following paragraphs.

In order to provide further insights into the drainage conditions and failure mechanisms associated with skirt length and the configuration of perforations, the bearing capacity factors for central uplift tests have been calculated from eq. [4] and are summarised in Table 3. Fig. 11 presents the bearing capacity factors for non-perforated mudmats (B) as a function of skirt length in comparison with limit analysis results from Randolph et al.

(2004) and experimental results from Chen et al. (2012). Results from Randolph et al. (2004) are presented for a soil heterogeneity of kB_0/s_{um} = 0, 3 and 10, encompassing the heterogeneity of the soil samples. The bearing capacity factors for non-perforated mats ranged from 6.84 to 8.37 with skirt length ratio (h/B_0) varying from 0 to 0.2. This agrees well with those obtained by Chen et al. (2012) in soil samples of a similar heterogeneity ratio (ranging from 3.38 to 3.61) indicating good repeatability of the present tests. They also compare reasonably well with the limit analysis solutions of Randolph et al. (2004), although there is a trend for an overestimation of the bearing capacity factor for flat foundations (i.e. $h/B_0 = 0$).

Fig. 12 presents the bearing capacity factors for all the three model mudmats as a function of the effective width. There is an evident trend of reduction of bearing factors with reduced effective width. It is also noteworthy that the effect of the embedment, which increases bearing capacity factors (see Randolph et al., 2004), reduced as the effective width decreased. As mentioned previously, the reduction in bearing capacity factors could be attributed to an accelerated dissipation of excess pore pressures with increased occurrence of perforations. However, the load/displacement curves in Fig. 6, and pull-out stiffness in Fig. 8, indicate that perforated mats exhibited a stiffer load displacement response, and a faster generation of suction at the foundation invert. Both observations demonstrate that the drainage conditions for perforated mats were also undrained, and that the reduction in uplift capacity (and associated bearing capacity factors) was essentially due to an earlier onset of suction breakaway at the mat invert caused by the perforations.

No theoretical solutions have been established to determine bearing capacity factors for perforated mudmats. The closest solution is the one developed by Martin and Hazell (2005), who established bearing capacity factors using the method of characteristics for 2D surface multi-strip footings subjected to downward vertical loadings under undrained conditions. Results from Martin and Hazell (2005) are plotted in Fig. 13 for soil heterogeneity ratios ranging from 0 to 5. They indicated a trend of reducing bearing capacity factor with increasing perforation ratio, beyond a value that depends on the strength heterogeneity ratio.

The perforation ratio used by Martin and Hazell (2005) in Fig. 13 is defined under 2D plane strain condition as the ratio of total footing spacing to the total width, so a distinction cannot be made between the perforation ratio and the effective width, as for the 3D models. In order to enable a direct comparison with the experimental results, an equivalent perforation ratio α^* was calculated for mudmats P1 and P2, as illustrated in the inset in Fig. 13. The equivalent perforation ratio α^* was calculated by converting the shaded area A_{sh} into an equivalent width y, resulting in values of 0, 0.28 and 0.23 for mudmat B, P1 and P2, respectively. Bearing capacity factors for the three mudmats are plotted in Fig.13, considering the equivalent perforation ratio, for comparison with results from Martin and Hazell (2005).

Bearing capacity factors for the non-perforated mat agreed reasonably well with results from Martin and Hazell (2005), accounting for the effects of heterogeneity ratio and skirt length. The agreement was also satisfactory for the perforated mat P1, but not for the perforated mat P2, as Martin and Hazell (2005) only modelled two-dimensional strips that cannot account for the effect of different perforated patterns. Nevertheless, the results suggest that Martin and Hazell (2005) might be used as a first approximation to evaluate the effect of perforation on uplift capacity, provided that the effective width is not reduced by more than a factor of 10, compared to a plain foundation of identical overall dimensions.

4.3 Effect of eccentric loading combined with perforation

Fig. 14 presents typical variation of uplift force and pore pressures with uplift displacement for the eccentric uplift test S2-7. The mudmat experienced a rotational soil failure mechanism (as illustrated in Fig. 14) about a point located close the centre of the mudmat. This resulted in positive excess pressures being mobilised at the end farthest from the lifting side (instrumented with PPT1) and negative excess pore pressures at the lifting side (instrumented with PPT3). The peak excess pore pressures at the mat for all the eccentric uplift tests (refer to Table 2) is presented in Fig. 15.

Fig. 15 presents the approximate pore pressure profile along the length of the mudmats (L_0) at failure. Note that PPT3 in test S2-6 ceased to function during the test, so no data is available (Fig. 15c). It can be seen that the perforation did not change the general failure mechanism as detailed above, which remained rotational. The perforation lead to lower suction being generated at the uplift side, but is unlikely to have significantly affected the excess pore pressure on the opposite side, indicating that they are most likely generated by the increase in bearing pressure resulting from the self-weight of the foundation being applied on a smaller section of the mat as it is being uplifted. It is also noteworthy that the centre of rotation of the mudmat moves away from the lifting point with increasing skirt length. As the skirt length increases, a deeper failure mechanism is generated, with breakaway at the mudmat invert occurring later during uplift.

5. DISCUSSION AND RECOMMENDATIONS

Table 4 summarises the ratio of measured uplift resistance to central uplift resistance for all tests. The reported values enable comparison between eccentric and centered uplift tests, and configurations of perforations. For the non-perforated mudmat, the eccentric uplift considered reduced the peak resistance by 66% to 79% compared to central uplift tests, decreasing with increasing skirt length due to the deeper failure mechanisms mobilised with longer skirts. When a large number of small perforations were introduced (mudmat P2), the peak uplift resistance was reduced by about 74% compared to the central uplift of non-perforated mudmats, with less effect from the skirt length. In contrast, a small number of large perforations (mudmat P1) yielded a reduction in uplift capacity of only 45% indicating that the effective width is the relevant parameter when determining the effect of perforation. In summary, the results indicate that eccentric uplift appears to be more efficient in reducing the uplift resistance than perforation ratio (for the range considered in this study), although both reduce the uplift capacity by generating an early breakaway at the foundation invert. Eccentric uplift is indeed more efficient in reducing the uplift capacity, as the breakaway can propagate more rapidly along a larger surface. However the efficiency of eccentric uplift is hindered by higher skirt embedment, whereas the central uplift capacity of perforated mudmat is less affected by skirt lengths.

By combining eccentric uplift and perforations, the mudmats experience the highest reduction in uplift resistance, with a reduction of ~ 76% for mudmat P1 and ~ 93% for mudmat P2 (i.e. the uplift force is only slightly larger than the self-weight of the mat), with skirt length having only a relatively small effect.

6. CONCLUSIONS

A series of centrifuge tests were undertaken to assess the effect of perforations and loading eccentricity on the uplift capacity of subsea mudmats. The results demonstrated that the uplift capacity in all cases is essentially sustained by the generation of suction pressures at the mudmat invert, and that undrained soil conditions prevailed for all tests, regardless of the configuration of perforation. The reduction of uplift capacity, which can reach up to ~ 80%, results from the breakaway of suction at the foundation invert, which can be generated either by perforations or by eccentric uplift. Eccentric uplift was observed to have a much greater effect in reducing the uplift capacity than perforations, although the benefit reduces with increasing skirt embedment.

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LIST OF SYMBOLS

B	non-perforated mudmat
P1	big perforated mudmat
P2	small perforated mudmat
A	gross area

A_{sh}	shaded area
B_0	width
d	thickness
d_0	diameter
c_v	coefficient of consolidation
e	eccentricity
E_s	secant stiffness
F_{up}	peak uplift force
$F_{up,net}$	net peak uplift force
G'	submerged self-weight
h	skirt length or embedment
k	gradient of soil strength
L_0	length
N_c	bearing capacity factor
$N_{\text{T-bar}}$	bearing capacity factor for T-bar
p_1, p_2, p_3	peak suction monitored by PPT1, PPT2 and PPT3
$\bar{p}$	average suction ($=(p_1+p_2+p_3)/3$)
q_u	uplift resistance
s_u	soil undrained shear strength
s_{um}	initial soil strength at mudline
$s_{u\mathrm{o}}$	soil strength at skirt tip
$s_{u\mathrm{o},ref}$	reference soil strength
s_{uop}	operative soil strength at skirt tip

v velocity

x shortest drainage path

y equivalent width

z depth

W effective width

w_p peak uplift distance

α perforation ratio

α^* equivalent perforation width

γ' submerged unit weight of soil

$\dot{\gamma}$ strain rate

$\dot{\gamma}_{ref}$ reference strain rate

μ strain rate parameter

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List of Tables

Table 1. Characteristics of model mudmats.

Mudmat type*	Perforated ratio α (-)	Skirt length h		Perforated diameter d_0		Effective width W		Submerged weight G'	
		Model (mm)	Prototype (m)	Model (mm)	Prototype (m)	Model (mm)	Prototype (m)	Model (N)	Prototype (MN)
B	-	0	0.00	-	-	75.00	11.25	62	1.40
	-	5	0.75	-	-	75.00	11.25	66	1.49
	-	10	1.50	-	-	75.00	11.25	70	1.58
P1	0.19	0	0.00	6.0	0.90	6.07	0.91	52	1.17
	0.19	5	0.75	6.0	0.90	6.07	0.91	56	1.26
	0.19	10	1.50	6.0	0.90	6.07	0.91	60	1.35
P2	0.19	0	0.00	2.7	0.41	3.34	0.50	56	1.26
	0.19	5	0.75	2.7	0.41	3.34	0.50	60	1.35
	0.19	10	1.50	2.7	0.41	3.34	0.50	64	1.44

* B=mudmat without perforations; P1=mudmat with large perforations; P2=mudmat with small perforations

Table 2. Summary of mudmat tests.

Soil sample & test no.	Mudmat type	Skirt length h (mm)	Eccentricity e (mm)	Peak uplift force F_{up} (N)	Peak suction p_1 (kPa)	Peak suction p_2 (kPa)	Peak suction p_3 (kPa)	Average suction $\overline{p}$ (kPa)	Peak uplift distance w_p (mm)
S1-1	B	0	0	199.1	-21.8	-28.5	-25.6	-25.3	1.07
S1-2		5	0	224.0	-33.5	-39.2	-41.5	-38.1	1.51
S1-3		10	0	276.6	-44.2	-44.5	-44.7	-44.5	1.99
S1-4	P1	0	0	127.8	-16.7	-20.3	-19.6	-18.9	0.42
S1-5		5	0	140.5	-11.3	-24.0	-23.8	-19.7	0.63
S1-6		10	0	174.7	-	-	-28.3	-28.3	0.83
S1-7	P2	0	0	88.3	-4.5	-	-11.4	-8.0	0.26
S1-8		5	0	102.2	-6.8	-12.8	-16.5	-12.0	0.31
S1-9		10	0	109.8	-11.2	-12.3	-12.7	-12.1	0.43
S2-1	B	0	40	75.1	9.5	-2.1	-22.1	-14.7	1.82
S2-2		5	40	98.3	12.7	-5.1	-43.0	-11.8	2.90
S2-3		10	40	130.2	14.7	-18.8	-34.8	-13.0	2.80
S2-4	P1	0	40	68.5	5.9	-2.1	-19.1	-5.1	0.42
S2-5		5	40	75.0	8.9	-1.0	-27.3	-6.5	2.04
S2-6		10	40	92.4	21.8	-8.7	-	6.6	2.54
S2-7	P2	0	40	60.5	1.8	0.1	-10.1	-2.7	0.86
S2-8		5	40	64.9	7.4	1.2	-13.5	-1.6	1.25
S2-9		10	40	82.4	18.3	-11.6	-13.3	-2.2	2.22

Table 3. Bearing capacity factors inferred from central uplift tests.

h	h/B_0	N_c		
mm	-	B	P1	P2
0	0	6.84	3.76	1.74
5	0.1	7.88	4.26	2.10
10	0.2	8.37	4.70	2.14

Table 4. Ratio of uplift resistance to central uplift resistance for all mudmat tests.

Mudmat type	B	B	P1	P1	P2	P2
Eccentricity	$e/L_0 = 0$	$e/L_0 = 0.4$	$e/L_0 = 0$	$e/L_0 = 0.4$	$e/L_0 = 0$	$e/L_0 = 0.4$
$h/B_0 = 0$	1.00	0.21	0.55	0.23	0.25	0.06
$h/B_0 = 0.1$	1.00	0.36	0.54	0.24	0.27	0.05
$h/B_0 = 0.2$	1.00	0.44	0.56	0.24	0.26	0.09

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