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Pullout behaviour of inclined shallow plate anchors in sand

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Abstract: This paper presents an experimental study on the pullout behaviour of inclined shallow plate anchors subject to axial pull in sand. The 1g model tests were performed to examine the effects of anchor inclination and sand-anchor interface conditions on the load-displacement response and the associated failure and deformation mechanisms of plate anchors at various embedment ratios and sand densities. The anchor pullout capacity was found to increase continuously with the load inclination angle to the vertical (α), and the increase was more significant for α from 45° to 90°. The effect of sand-anchor interface conditions was negligible for horizontal plate anchors ($\alpha=0^\circ$) but it became increasingly significant at larger inclination angles. The effects of these two factors both decreased with an increasing embedment ratio. Their influences on the failure and deformation mechanisms were measured and

26 analysed using a digital image correlation (DIC) technique. Based on the test data and
27 results available in the literature, a simple empirical method for the prediction of pullout
28 resistance of inclined plate anchors in sand is calibrated and recommended.

29 **Key words:** inclined anchor, pullout capacity, failure mechanism, model tests, sand.

Draft

30 **Introduction**

31 Plate anchors are often used to provide pullout forces for various onshore and
32 offshore structures such as retaining walls, bridge abutments, sheet piles, transmission
33 towers and mooring systems. The performance of plate anchors, especially the ultimate
34 pullout capacity, is pivotal for the safety and integrity of the structures. Numerous
35 studies on the pullout behaviour of plate anchors in sand appeared in the literature
36 (Choudhary and Dash 2017; Dickin and Leung 1983; Evans and Zhang 2019; Hanna et
37 al. 2015; Jadid et al. 2019; Liu et al. 2011; Merifield and Sloan 2006; Murray and
38 Geddes 1989; Neely et al. 1973; Rowe and Davis 1982; Shahriar et al. 2020). Most of
39 these have concerned with either horizontal or vertical plate anchors. There is far less
40 information available on inclined plate anchors which are more adaptable for resisting
41 overturning movements of structures (Das and Shukla 2013; Ghaly and Clemence
42 1998).

43 It has been shown that the pullout capacity of anchors in sand subject to axial pull is
44 a function of several factors such as anchor geometry, inclination, embedment ratio,
45 sand type and density, sand-anchor interface conditions, and method of installation (Das
46 and Shukla 2013; Merifield and Sloan 2006; Pérez et al. 2017). The load-displacement
47 response and the failure and deformation mechanisms of plate anchors at two extreme
48 conditions (i.e. uplifted horizontal anchors and laterally loaded vertical anchors) have
49 been extensively studied using various advanced theoretical (e.g. limit analysis, finite
50 element method and discrete element method) (Evans and Zhang 2019; Kumar and

51 Sahoo 2012; Merifield and Sloan 2006; Rowe and Davis 1982) and experimental
52 techniques (e.g. Particle Image Velocimetry (PIV)) (Choudhary and Dash 2017; Dickin
53 1988; Dickin and Leung 1983; Liu et al. 2011; Liu et al. 2013; Yue et al. 2020). For
54 inclined anchors in sand, previous experimental investigations mostly focused on the
55 load-displacement response (e.g. the ultimate capacity and failure displacement), which
56 showed that the anchor inclination may significantly influence the ultimate capacity,
57 particularly while the inclination angle to the vertical α is greater than a typical value
58 (e.g. 45°) (Hanna et al. 2015; Harvey and Burley 1973; Meyerhof 1973; Murray and
59 Geddes 1989; Ovesen 1981). On the other hand, relatively few tests have attempted to
60 reveal the underlying failure and deformation mechanisms. Frydman and Shaham
61 (1989) and Ghaly and Clemence (1998) measured the failure mechanisms of inclined
62 anchors in sand by tracing the discontinuities of thin colored sand layers. However,
63 these tests were limited to the cases of anchors inclined at $\alpha \leq 60^\circ$, and the colored-
64 layered sand technique is usually hard to get detailed information about the failure
65 process and the subsurface soil deformation (e.g., soil particle movement and strain
66 localization), particularly between the colored layers. In addition, although the effect of
67 anchor roughness on the pullout behaviour of horizontal and vertical plate anchors has
68 been well recognized in the literature (Merifield and Sloan 2006; Rowe and Davis 1982;
69 Yue et al. 2020), its possible influences on the failure mechanisms of inclined plate
70 anchors in sand have rarely been measured (Bhattacharya and Kumar 2014). It is known
71 that theoretical methods such as limit equilibrium approaches, methods of
72 characteristics and limit theorems for the calculation of anchor resistance heavily rely

upon the agreement between the assumed and the realistic failure and deformation mechanisms (Das and Shukla 2013; Shahriar et al. 2020; White et al. 2008; Yu et al. 2014). Hence, the abovementioned lack of experimental observation about the failure and deformation mechanisms of inclined plate anchors limited the development and validation of relevant theoretical methods.

This paper presents an experimental study on the pullout behaviour of inclined (e.g. $\alpha \in [0^\circ, 90^\circ]$) shallow ($H/h \leq 7$, H is the depth from the bottom edge of plate anchor to the soil surface, h is the height of anchor) plate anchors buried in sand. A total of 64 1g model tests were performed. By incorporating a nondestructive deformation measurement technique (i.e. PIV), the coupling effects of anchor inclination, embedment ratio, sand density and sand-anchor interface conditions on the load-displacement characteristics and the associated failure and deformation mechanisms during the entire loading process are systematically studied. Finally, a simple empirical method for the prediction of ultimate capacity of inclined plate anchors in sand is calibrated and validated based on the present test results and some data available in the literature.

Test setup, materials and procedure

Test setup

Fig. 1 presents a sketch of the test rig that was designed and used in this study. The plate anchor was placed within a cubic chamber. The inside dimensions of the chamber were 1200mm, 252mm and 800mm in length, width and height, respectively. The front

and back walls of the chamber were made from transparent tempered glass (20mm thick), which allows observation of anchor movements and soil deformation during anchor pullout, and they were firmly supported by two pieces of steel plates and a steel base. The model anchors were made from steel plate, which had dimensions of 250mm, 50mm and 10mm in length (L), height (h) and thickness (t), respectively. Two plastic endcaps (about 1mm in thickness) were attached to the model plate anchor to fill the gap between the anchor ends and the glass walls of the chamber. A Nikon D3100 digital still camera was placed in front of the chamber at a distance of 600mm, beside which two spotlights were used to provide stable lighting conditions.

The model anchor was driven by a servo-controlled actuator, which were connected by a steel rod and a pulling chain. The dragging load applied by the actuator was measured by a load cell with 2kN capacity and 0.2N accuracy. The pulling system was supported by a guide rail and a set of adjustable pulleys that were fixed on the steel frame and supporting beam, and the direction of loading in the vertical-horizontal plane (i.e. the inclination angle of loading to the vertical α in the range of 0~90°) was controlled by changing the direction of the steel rod and adjusting the positions of the guide rail and the pulley. Note that openings with a dimension of 100mm (height)×10mm (width) at given depths were reserved in one of the side walls for performing tests with plate anchors inclined at 60° and 90°.

Fig. 1 [Insert near here]

The Geo-PIV technique developed by White et al. (2003) was used to track the

anchor movements and measure the associated soil deformation. The Geo-PIV technique operated by recognizing the natural texture of the sand particle within the divisiory patch. Its location in the subsequent image was recognized when the highest correlation occurred, thereby revealing the displacement of the patch. The real soil displacement was obtained by conducting the translation of the image-space coordinates into object-space coordinates. More details about this technique refer to White et al. (2003) and Yue et al. (2020). Images were taken at a resolution of 4608×3072 pixels, providing a field of view (FOV) in the object-space of about $600\text{mm} \times 600\text{mm}$ in this study. Using the chosen patch size of 32×32 pixels, the precision error was less than 0.04mm according to the empirical equation proposed by White et al. (2003).

Test materials

A uniformly graded silica sand was used in the tests, whose mean grain size d_{50} was 0.4mm , uniformity coefficient C_u was 2.83, coefficient of curvature C_c was 1.32, and maximum and minimum dry densities were 1.745g/cm^3 and 1.385g/cm^3 , respectively. The ratio of the anchor height to the mean grain size (i.e. h/d_{50}) is 125, and this is larger than the minimum ratio that was suggested to avoid possible grain size effects in sand-anchor interaction problems (Athani et al. 2017; Bradshaw et al. 2016; Garnier et al. 2007). The model tests were performed in sand samples of two relative densities, namely $D_r=83\%$ and $D_r=50\%$, and they can be classified as dense and medium-dense sand, respectively, according to ISO 14688-2 (2017). To investigate the anchor surface

roughness effect on the pullout behaviour, plates with two surface conditions, namely
 graved and polished (see Fig. 2), were fabricated, which were named as rough and
 smooth anchors, respectively. Direct shear tests and interface direct shear tests under
 different levels of constant normal stress (i.e. 50-400kPa) were performed to determine
 the internal friction angles of sand φ and the sand-anchor interface friction angles φ_{in}
 at each density as summarized in Table. 1 (Yue et al. 2020). The friction angles (i.e.
 $\varphi_{in}=0.56\sim0.99\varphi$) measured between sand and the anchor of typical surface conditions
 cover a wide range of sand-structure interface friction angles that may exist in practical
 conditions (i.e. $\varphi_{in}=0.3\varphi-1\varphi$) (Ho et al. 2011; Su et al. 2018; Uesugi and Kishida 1986)
 as expected, although the fabricated surface conditions are not perfectly identical to the
 practical plate anchors.

Fig. 2 [Insert near here]

Table. 1 [Insert near here]

Sample preparation and test program

The air pluviation method with a bespoke point pourer was used to prepare sand
 samples (Yue et al. 2020). It was assumed that the samples were uniform following the
 literature (Ansari et al. 2018; Dave and Dasaka 2012; Gade and Dasaka 2016). Different
 sand densities were achieved by changing the deposition intensity (i.e. drop height and
 flow rate). Duplicate tests were performed to calibrate the accuracy and reliability of
 the preparation process. At first, a bedding layer of 300mm (i.e. $6h$) was poured in all
 the tests to minimize the possible bottom boundary effects. Afterwards, the model

157 anchor was set on the surface of the sand bed with its longitudinal axis perpendicular
158 to the front and back walls of the chamber and the tie rod was connected. For the cases
159 of $\alpha=0^\circ$ (i.e. horizontal plate anchor), the plate anchor was placed in the middle of the
160 chamber in the length direction, while it was at a horizontal distance of $16h$ from the
161 right wall of the chamber for other cases (i.e. $\alpha=30^\circ$, 45° , 60° and 90°) as the pulling
162 direction was from left to right. After the installations, sand pouring continued until the
163 target embedment depth was reached. Extra care has been taken while pouring sand
164 around the tie rod, the plate anchor and the walls of the chamber to prepare the samples
165 as uniform as possible, for example, sand beneath the tie rod and the plate were poured
166 by slightly bending the flexible tube on the sand pluviator. Note that there is no
167 surcharge acting on the sand surface in the present tests, and the effects of anchor
168 installation method (e.g. trench-backfilling, pushing-in and suction embedded methods)
169 were out of the scope of this study.

170 The single plate anchor was subsequently pulled in a displacement-controlled
171 manner at a constant rate of 1.6mm/min which is slow enough to ensure that the pullout
172 resistance is not affected by the velocity effect (Athani and Rognon 2019). Images were
173 captured simultaneously with an interval of the 30s until the total axial displacement of
174 the anchor reached or approached to $0.2H$. A total of 64 model tests using 64 individual
175 samples were performed, which includes cases of five inclination angles (i.e. $\alpha=0^\circ$, 30° ,
176 45° , 60° and 90°), four initial embedment ratios ($H/h=1$ to 7 with an interval of 2) and
177 two relative densities of samples ($D_r=83\%$ and 50%) (Table. 2). Note that raw data of
178 tests at $\alpha=90^\circ$ (i.e. vertical plate anchors) are after the companion paper of Yue et al.

179 (2020).

180 **Results and discussion**

181 *Load-displacement curves*

182 Calibration tests were performed to measure the forces caused by the weight of the
 183 model anchor and the loading system and the friction exerted on the ends of the plate
 184 anchors against the glass walls. They were subtracted from the total axial force
 185 measured by the load cell, by which the net pullout force P was obtained. Normalised
 186 load-displacement curves in terms of $N_\gamma = P/\gamma HhL$ (γ is the effective unit weight of sand)
 187 versus δ/H (δ denotes the anchor movement along the loading direction) obtained from
 188 tests at $\alpha=0^\circ$, $\alpha=45^\circ$ and $\alpha=90^\circ$ are plotted in Fig. 3. Note that the friction angle between
 189 the sand and the glass walls may lie in the range of 10° - 15° (Ansari et al. 2018;
 190 Butterfield and Andrawes 1972). The frictional resistance at the sand-glass interface
 191 above the anchor would be different from the true plane strain conditions, and this may
 192 lead to slightly higher pullout resistance (Cheuk et al. 2008).

193 Fig. 3 [Insert near here]

194 In Fig. 3, it is shown that initially the pullout resistance increased almost linearly
 195 with anchor movements. Peak resistance was reached soon after this pseudo-elastic
 196 stage, which is followed by gradual reductions in the resistance with further anchor
 197 movements for the cases at $\alpha=0^\circ$ and 45° . In the contrary, the resistance continued to
 198 increase for the laterally loaded anchors (i.e. $\alpha=90^\circ$), although at very slow speeds.

199 Significant oscillations in the pullout resistance occurred in the post-peak stage for the
 200 cases at $\alpha=0^\circ$, whereas this phenomenon lessened or vanished with increases of the
 201 inclination angle α . Similar features of the load-displacement curves have been
 202 reported in the literature (e.g. (Dickin and Leung 1983; Hossain and Rognon 2020; Liu
 203 et al. 2011; Wu et al. 2019)). The post-peak oscillations in the load-displacement curves
 204 were mainly due to the sudden release of stress as the sand above the plate anchor
 205 intermittently flowed downward to fill the cavity formed below the anchor (Ansari et
 206 al. 2018; Cheuk et al. 2008), and this phenomenon is loading-direction dependent
 207 (Hossain and Rognon 2020; Wu et al. 2019). The magnitude of the oscillations may
 208 also vary with the grain size and the rate of loading (Hossain and Rognon (2020)).

209 For the cases that post-peak reductions were shown (e.g. Fig. 3 (a) and (b) and some
 210 cases in Fig. 3 (c)), peak resistances were defined at points where the slope of the load-
 211 displacement curve reached zero. In other cases, the ultimate resistances were defined
 212 at the points beyond which the load-displacement curve tended to be linear (Choudhary
 213 and Dash 2017; Dickin and Leung 1983; Murray and Geddes 1989). All the failure
 214 points are marked by circles in Fig. 3. Peak pullout factors $N_{\gamma-p}$ (referred to as the break-
 215 out factor) and corresponding failure displacements δ_f (referred to as the mobilisation
 216 displacement) measured in the tests are summarized in Table. 2. It is shown that δ_f lay
 217 in a range of 1.73%-12.94% H in the medium-dense tests and 0.95%-11.5% H in the
 218 dense tests. The value of δ_f became larger with increases of sand density, load
 219 inclination to the vertical and anchor embedment ratio, whereas it was marginally
 220 influenced by the anchor roughness. Mobilisation displacements that were measured in

this study are in agreement with test results reported in the literature (e.g. (Choudhary and Dash 2018; Liu et al. 2011; Murray and Geddes 1989)). Nevertheless, it has been shown that the geometry scale effects may influence the formation and propagation of the shear zones, thereby the mobilization displacement (Palmer et al. 2003). Hence, care should be exercised while extrapolating the mobilization displacement of a prototype plate anchor based on the data that were obtained in the scaled model tests.

Table. 2 [Insert near here]

The variations of $N_{\gamma-p}$ with the inclination angle α at different embedment ratios are plotted in Fig. 4. It is shown that $N_{\gamma-p}$ becomes larger at greater load inclinations with respect to the vertical, in particular, while $\alpha > 45^\circ$. This observation is in line with the literature (e.g. (Bhattacharya and Kumar 2014; Frydman and Shaham 1989; Murray and Geddes 1989)). The sand-anchor interface conditions influenced the value of $N_{\gamma-p}$ for inclined shallow anchors, for example, the rough anchors at $\alpha=90^\circ$ had higher peak pullout resistances than the smooth at identical conditions by 2.8% to 34.4%. The difference between the rough and the smooth anchors reduces with an decreasing value of α and almost vanished at $\alpha=0^\circ$. In addition, the test results indicate that the effects of load inclination and sand-anchor interface conditions on the break-out factors tend to decrease with increasing embedment ratios (Fig. 5).

Fig. 4 [Insert near here]

Fig. 5 [Insert near here]

Failure mechanisms at peak resistance

The failure mechanism associated with the development of soil resistance are of great importance in the calculation of anchor pullout resistance. To show those information, results of cumulative displacement fields at the mobilisation displacements are plotted in Figs. 6 and 7, in which the middle locations of the observed shear planes are outlined by red dash lines (i.e. lines a_1 , a_2 , p_1 , and p_2). It is shown that the slip surfaces roughly coincide with the contour lines at which the cumulative displacement is half the value of the total displacement at the same depth.

Fig. 6 [Insert near here]

Fig. 7 [Insert near here]

Three failure mechanisms are shown in Figs. 6 and 7, including the wedge-type general shear failure mechanism (i.e. the shear zones shear out at the soil surface), the flow-type local failure mechanism (i.e. sand particles flow around the plate anchor) (see Figs. 6 (d-1) to (d-4)) and the transition mechanism between them (e.g. Figs. 6 (c-1) and (c-2)). The results indicate that the failure mechanism to occur depends on anchor embedment ratio and sand density, and this is consistent with other experimental observations on horizontally or vertically loaded plate anchors (e.g. (Choudhary and Dash 2017; Liu et al. 2011)). In the meanwhile, in the medium-dense tests at $H/h=7$, the deformation pattern was localised for α from 0° to 60° , whereas it extended to the surface for $\alpha=90^\circ$. This suggests that the critical embedment ratio, beyond which a local shear failure dominated, may increase with the load inclination to the vertical. This phenomenon can be more clearly showed by plotting the instantaneous particle

movements and shear strain field at the peak resistance (e.g. Fig. 8, taking tests with the smooth anchor at $H/h=7$ with $D_r=50\%$ as an example). It is shown that sand particles above the horizontal anchor (i.e. $\alpha=0^\circ$) flowed downwards along two sides of the plate and into the cavity formed beneath, which was driven by gravity. In contrast, the surcharge above an inclined plate anchor tended to suppress the soil particles in the front move to backward, and this effect increased for plate anchors with greater inclination angles to the vertical. As a result, a general shear failure was observed for the vertical plate anchor at $H/h=7$ in the medium-dense tests, whereas a localized failure pattern was observed in other cases at the same embedment ratio. In addition, the geometric center of the plate was shallower with an increasing α as the same buried depth was controlled as the vertical distance from the bottom edge of plate to the soil surface in this study. This may also contribute to the inclination angle dependency of the critical embedment ratio. The observed variation of the critical embedment ratio with the loading agrees with that reported by Wang and Wu (1980). Nevertheless, it needs to be mentioned that conflict results have also been reported in similar tests, for example, Ghaly and Clemence (1998) reported an opposite dependence of the critical embedment ratio on the load inclination for helical screw anchors and Wu et al. (2019) observed that the critical embedment ratio was independent of the direction of pipe movements in sand.

Fig. 8 [Insert near here]

For a shallow plate anchor at $\alpha=0^\circ$, its peak pullout resistance was mobilised through

the formation of an inverted trapezoidal block which bounded by two outward inclined slip surfaces (i.e. p_1 and p_2). These two slip surfaces were symmetrical with respect to the axis of the load direction and almost linear in the 1g model tests. These observations are in agreement with the literature (e.g. (Liu et al. 2011; Liu et al. 2013)). Following Cheuk et al. (2008), the mean outward inclination angle to the vertical (θ) was measured by drawing a straight line passing through the end of the plate anchor and the point at which the cumulative displacement is half the value of the maximum at the soil surface. Figs. 6 and 7 show that the value of θ varied with anchor embedment ratio and sand density, which is mainly because of the dependency of dilatancy behaviour of sand on the sand state (i.e. stress level and relative density) (Bolton 1986; Roy et al. 2018). The mean values of θ measured by the PIV analysis (i.e. 21.4° in the dense tests and 11.3° in the medium-dense tests) are slightly greater than the peak dilation angles of sand that are predicted by the empirical stress-dilatancy correlations of Bolton (1986, 1987) (see Table. 1). As a result, underprediction to the peak uplift resistance may be caused while using the estimated dilation angle of sand to establish the geometry of the failure mechanism of shallow strip anchors in limit equilibrium analysis (White et al. 2008).

The slip surfaces in front of the plate anchor (or the boundaries of the passive shear zones, i.e. p_1 and p_2) became no longer symmetrical as the load inclination deviated from the vertical direction (Figs. 6 and 7). For the surface anchors (i.e. $H/h=1$), p_1 and p_2 ahead of an inclined anchor can still be approximated by linear lines. The apex angle β_2 (i.e. the angle between p_2 and the horizontal soil surface on the left side of p_2) changed from $\pi/2-\psi$ at $\alpha=0^\circ$ to be $\pi/4-\phi/2$ at $\alpha=90^\circ$, and the apex angle β_1 (i.e. the

angle between p_1 and the soil surface on the left side of p_1) was greater than β_2 by approximately 1~2 times of the dilation angle of sand. With increases of the embedment ratio, p_1 and p_2 ahead of inclined anchors became increasingly curved and the apex angles β_1 and β_2 became larger. In general, the shape of p_1 and p_2 in front of an inclined plate anchor can be closely represented by a proper combination of segments of logarithmic spiral curves and straight lines. For example, for plate anchors at $\alpha=90^\circ$ with a general shear failure, the shape of p_2 can be roughly represented by a logarithmic spiral with the pole on the top edge of the plate anchor at the height range of $0-2h$ from the bottom edge of the anchor, beyond which p_2 developed to the soil surface almost linearly.

For anchors buried at the same embedment ratio, Figs. 6 and 7 show that a greater volume of soil was mobilised by shallow anchors at greater inclination angles to the vertical, leading to a longer total length of the slip surfaces (or greater shear resistances) and greater weight of soil to resist anchor movement. This phenomenon is more significant for anchors with inclination angles larger than 45° . The width between the intersection points of p_1 and p_2 with the soil surface slightly increased with anchor inclination. The apex angles β_1 and β_2 became smaller at greater inclination angles to the vertical. The above features of the failure mechanism explained the variation of the break-out factor with the inclination angle that was shown in Fig. 4.

In addition, it is noted that, while the inclination angle was larger than 45° , a range of sand above the plate anchor collapsed into the cavity that formed behind the anchor.

This can be more clearly seen from the instantaneous displacement vectors superposed on the instantaneous deformation fields in Fig. 9. The range of this active failure zone, bounded by slip surfaces of a_1 and a_2 , extended with increases of the inclination angle (Figs. 6 and 7). For shallow anchors at $\alpha=90^\circ$, a_1 stemmed from the bottom edge of the plate anchor and propagated upwards at an angle to the vertical of $45^\circ-\phi/2$ to the height of the top edge, and a_1 and a_2 then developed towards the soil surface almost vertically. The active soil pressure acting on the rear of the inclined plate anchor with $\alpha \geq \phi$ applied an opposite effect on the anchor pullout resistance.

Fig. 9 [Insert near here]

At relatively deep depths, a localized triangular soil wedge was formed ahead of a horizontal plate anchor, which had base angles of $\pi/4+\phi/2$ (Figs. 6 (d-1)) (i.e. local shear failure mechanism). With an increasing value of α , the local shear pattern become rotational with its centre being close to the top edge of the plate anchor. For example, in the medium-dense tests at $\alpha=60^\circ$ and $H/h=7$, the failure mechanism resembled an inclined ellipse with its long axis (around $2h$) perpendicular to the load direction, and it started from the bottom edge of the plate anchor (e.g. Figs. 6 (d-4)). The rotary motion of the local shear pattern mainly because of the stress and strength gradients along the depth (Yu et al. 2015). This observation agrees with that observed by Dickin and Leung (1985) ahead of a deep vertical plate anchor but it differs from the shape of closed bulb that was observed by Ghaly and Clemence (1998) ahead of a deep helical screw anchor in sand.

Effects of anchor roughness on failure mechanism

It is known that the sand-steel interface frictional strength is heavily dependent on the relative interface roughness (i.e. anchor surface roughness/ d_{50}), relative displacement and sand properties (Han et al. 2018; Ho et al. 2011; Lings and Dietz 2005; Uesugi and Kishida 1986). As such, higher interface frictional strength was measured with the rough anchor than the smooth at identical sand conditions (see Table 1). Influences of the interface conditions on the mobilisation of soil resistance due to anchor movements can be illustrated by plotting the cumulative velocity vectors (e.g. Fig. 10).

For a vertically loaded horizontal plate anchor that was embedded in a uniform sand sample with a flat surface (e.g. $\alpha=0^\circ$ in this study), sand particles immediately above the plate move upwards predominantly (namely, no horizontal relative displacement occurs at the sand-anchor interface, e.g. Fig. 10 (a-1) and (a-2)), thus the interface conditions barely involved in the mobilisation of soil resistance in this case. This is consistent with the widely accepted conclusion that the anchor roughness applies an insignificant effect on the uplift behaviour of plate anchors in sand (Bhattacharya and Kumar 2014; Evans and Zhang 2019; Merifield and Sloan 2006; Rowe and Davis 1982). However, for inclined plate anchors (or anchors encounter unsymmetrical soil resistance to the load direction), the interface friction involves balancing the force component parallel to the interface, thus affecting the movements of sand particles in front of the anchor. This effect enhanced with increases in the inclination angle. Figs. 10 (b-1) and (b-2) compare the cumulative velocity vectors of sand particles at the

370 mobilisation displacements around the rough and the smooth anchors at $\alpha=90^\circ$. It is
371 shown that sand particles in front of the anchor move upwards at an angle to the plate
372 anchor. The shear stresses generated along the rough anchor forced the passive failure
373 zone more downwards than that in front of the smooth anchor. This can be clearly seen
374 by plotting the total vertical displacement of soil at depths of the top and bottom edges
375 of the plate at the mobilization displacement (see Fig. 11, taking the test at $H/h=5$ with
376 $D_r=50\%$ as an example). Thus, a larger soil mass in front of the plate anchor was
377 mobilised by the rough anchor than the smooth as summarised in Fig. 12. The anchor
378 roughness effect (or interface friction effect) on the failure mechanism reduces with
379 increases of the embedment ratio and vanishes at sufficiently deep depths (Yue et al.
380 2020). In addition, the shape of the active failure zone in rear of a plate anchor was less
381 affected by the anchor roughness, regardless of anchor inclination. The above
382 mechanisms explained the observed effects of the anchor roughness on the break-out
383 factors of inclined shallow plate anchors (i.e. Fig. 4).

384 Fig. 10 [Insert near here]

385 Fig. 11 [Insert near here]

386 Fig. 12 [Insert near here]

387 *Deformation mechanisms*

388 The deformation mechanism associated with the development of anchor resistance is
389 analysed by plotting the instantaneous shear strain contours superimposed with the
390 instantaneous displacement vectors, taking the results obtained in the dense tests with

the rough anchor at $H/h=5$ as an example (i.e. Fig. 13). The test results suggest that the deformation mechanism during anchor pullout consisted of three key stages: (i) mobilisation of peak (or failure) resistance (e.g. Fig. 13 (a)), (ii) formation of post-peak shear bands (e.g. Fig. 13 (b)) and (iii) flow around (e.g. Fig. 13 (c)). Similar stages have been identified in uplift tests of shallow pipes in sand, e.g. by Cheuk et al. (2008).

Fig. 13 [Insert near here]

Initially, the surrounding soil responded elastically. With further anchor movements, shear bands developed, stemming from the bottom and the top edges of the plate anchors. It is shown that the shear bands did not necessarily reach the ground surface at the mobilisation displacements for moderately shallow anchors. The level of shear strain along the shear bands distributed nonuniformly, reducing with the distance to the anchor, which indicates the progressive nature of the failure process. In other words, the mobilized shear strength of sand along the shear bands may already reduce to the critical state value at locations near the anchor while it is close to the peak value at a far distance for the anchor in dilative sand. In addition, no obvious sand infilling below the horizontal plate anchor was observed at the mobilisation displacement, whereas soil behind the anchor collapsed before the mobilisation displacement was reached for inclined anchors (e.g. Fig. 13). This phenomenon was primarily because larger mobilisation displacements were required for anchors at greater inclinations to the vertical and the inclined anchors provided less shielding effect for preventing the gravity-driven sand collapse behind the plate anchor.

412 With further increases of the anchor movement (i.e. in the post-peak phase), the range
413 of mobilized sand shrank, namely the shape of the failure wedge in front of the anchor,
414 bounded by the post-peak shear bands, became narrower (e.g. Fig. 13 a-2, b-2, c-2).
415 The shear zones were still inclined and curved outward around the soil surface as the
416 dilatancy of sand increased with decreases of the stress level. The shrinkage of the
417 failure wedge was due mainly to the strain-dependent behaviour of sand (e.g. softening
418 of sand strength and dilatancy) as the shear strain develops and the onset or acceleration
419 of sand infilling behind the plate. The sand infilling, shrinkage of the failure zone and
420 reduction in the mobilized shear strength on the shear planes are three main mechanisms
421 leading to the post-peak softening of the load-displacement response for the non-
422 vertical plate anchors in Fig. 3. No such post-peak softening in the load-displacement
423 response was observed in tests at $\alpha=90^\circ$ because of the rotation of the vertical plate
424 anchor and sand flush into the gaps between the plate ends and the sidewalls of the
425 chamber at large displacements, and more detailed discussion on this refers to the
426 companion paper (Yue et al. 2020).

427 At sufficiently large displacements, the deformation mechanism transitioned from
428 the heave of a soil block to a primarily flow-around, which has also been observed in
429 uplift tests of pipes in sand by Cheuk et al. (2008). The transition between the
430 deformation mechanisms seems to be dependent on the inclination angle. The flow-
431 around mechanism was localized and symmetrical with respect to the axis of the load
432 direction for the horizontal anchors (Fig. 13 (a-3)), whereas it was rotational for the
433 inclined plate anchors. The rotational failure pattern stemmed from the bottom edge of

the plate, and the mobilized sand particles flowed backwards from the top of the plate (Fig. 13 (b-3)). However, this mechanism was not observed around the vertical plate anchor with displacements up to $0.2H$. This might also contribute to the continuous increases of the soil resistance in the post-peak stage for the vertical plate anchors (Fig. 3).

Prediction of pullout resistance of inclined anchors

Fig. 4 showed that the break-out factor of a shallow strip plate anchor in sand is a function of several factors such as inclination angle, embedment ratio, sand density and sand-anchor interface conditions. The slip surfaces of inclined anchors are complex and cannot be represented by a simple geometrical shape, leading to difficulties in calculations of the pullout resistance. Alternatively, Meyerhof (1973) suggested an empirical relationship for anchor resistance prediction of inclined plate anchors. Based on experimental results, Das and Puri (1989) modified this approach to be Eq. (1) to improve its prediction accuracy.

$$N_{\gamma-p}^{\alpha} = N_{\gamma-p}^{0^{\circ}} + (N_{\gamma-p}^{90^{\circ}} - N_{\gamma-p}^{0^{\circ}})(\alpha / 90^{\circ})^n \quad (1)$$

where $N_{\gamma-p}^{0^{\circ}}$, $N_{\gamma-p}^{\alpha}$ and $N_{\gamma-p}^{90^{\circ}}$ are break-out factors of anchors at $\alpha=0^{\circ}$, α and 90° , respectively. n is a constant coefficient.

Das and Puri (1989) found that Eq. (1) with $n=2$ fits well with their experimental data of inclined square anchors in clay. The reliability of this approach for inclined strip plate anchors in clays was confirmed by Merifield et al. (2005) and Yu et al. (2011).

454 Taking this one step further, the relevance and accuracy of Eq. (1) for inclined plate
 455 anchors in sand are examined in this study. At first, measured break-out factors (i.e.
 456 Table. 2) are plotted in the normalized form of $(N_{\gamma-p}^{\alpha} - N_{\gamma-p}^{0^{\circ}}) / (N_{\gamma-p}^{90^{\circ}} - N_{\gamma-p}^{0^{\circ}})$ versus
 457 $\alpha / 90^{\circ}$ in Fig. 14. The test data were fitted by Eq. (1) with several typical values of n
 458 (e.g. $n=2, 2.5, 3$ and 3.5). Fig. 14 shows that $n \approx 2$ and $n \approx 3.5$ roughly represent the
 459 upper and lower limits of the normalized break-out factors, respectively. This value
 460 range of n is typically greater than that for anchors in clays, which means that the
 461 variation of $N_{\gamma-p}^{\alpha}$ with α in sands is slower at small inclination angles but quicker at
 462 large inclinations than that in clays.

463 Fig. 14 [Insert near here]

464 According to the calibration in Fig. 14, Eq. (1) with $n=2.5$ and $n=3.0$ was used to
 465 describe the inclination dependency of anchor break-out factors in a wider range of
 466 conditions. Simulation results obtained by Bhattacharya and Kumar (2014) using the
 467 lower bound theorem of the limit analysis in combination with finite elements and linear
 468 optimization and experimental results obtained by Murray and Geddes (1989) were
 469 selected for further validations. In the simulations of Bhattacharya and Kumar (2014),
 470 strip plate anchors in a wide range of sand properties have been considered, including
 471 $\varphi=30^{\circ}, 35^{\circ}, 40^{\circ}, 45^{\circ}, H/h=3-10, \alpha=0^{\circ}, 30^{\circ}, 45^{\circ}, 60^{\circ}, 90^{\circ}$, and $\varphi_{in}=0^{\circ}$ and $\varphi_{in}=\varphi$. In the
 472 experimental study of Murray and Geddes (1989), 33 model tests have been performed
 473 using square and rectangular plate anchors of $L/h=1, 2, 5$ at embedment ratios ranging
 474 from 2 to 10 and inclination angles ranging from 0° to 90° . Predicated and

simulated/measured break-out factors are compared in Figs. 15 and 16. The dash lines in Figs. 15 and 16 indicate the difference between predicted and simulated/measured break-out factors of $\pm 10\%$.

Fig. 15 [Insert near here]

Fig. 16 [Insert near here]

Figs. 15 and 16 show that Eq. (1) with $n=2.5$ gives a mean overprediction of the break-out factors less than 5% with the coefficient of variation (COV) ranging from 0.04 to 0.15, whereas predictions made by Eq. (1) with $n=3.0$ tend to be more conservative (or more safety) and closer to the simulated/measured results. Closer inspection of the results indicates that the value of n was marginally affected by other factors such as sand density, embedment ratio, anchor roughness and anchor length-width ratio. The close agreement between the prediction and the simulation/measurement suggests that Eq. (1) with a suitable value of n can well capture the variation of anchor break-out factor with inclination angle. Based on the database used, $n=3.0$ is recommended for inclined plate anchors in sand. Furthermore, $N_{\gamma-p}^{0^\circ}$ and $N_{\gamma-p}^{90^\circ}$ are also needed to theoretically predict the break-out factors of plate anchor inclined at $0^\circ < \alpha < 90^\circ$ in sand. According to the observed failure mechanisms, $N_{\gamma-p}^{0^\circ}$ might be estimated by the limit equilibrium solution of White et al (2008), and a simple mechanism-based solution for $N_{\gamma-p}^{90^\circ}$ will be reported in a forthcoming paper.

Conclusions

The pullout behaviour of inclined shallow plate anchors in sand has been investigated by performing a series of model tests. The effects of sand-anchor interface condition (smooth and rough), sand relative density ($D_r=50\%$ and 83%), anchor inclination to the vertical ($0\sim 90^\circ$), and embedment ratio ($H/h=1\sim 7$) on the load-displacement response and the associated failure and deformation mechanisms were studied. The main findings from the tests are summarized as follows:

- The break-out factor $N_{\gamma-p}$ of shallow plate anchors is a function of inclination angle, embedment ratio, sand density and sand-anchor interface conditions. The effect of anchor inclination on $N_{\gamma-p}$ was significant, particularly while $\alpha > 45^\circ$. The effect of sand-anchor interface conditions on $N_{\gamma-p}$ was negligible at $\alpha = 0^\circ$ but it enhanced with an increasing value of α . The effects of these two factors reduced with increases of the embedment ratio and became marginal for deep anchors.
- Post-peak softening was observed for horizontal and moderately inclined plate anchors, whereas the anchor resistance increased continuously with movements up to $0.2H$ for the vertical anchor. Post-peak oscillations in the load-displacement curves were significant for slightly inclined plate anchors, and this phenomenon gradually faded away with increases of the load inclination to the vertical.
- Peak pullout resistance was reached at axial anchor displacements in a range of $1.73\%-12.94\%H$ for the medium-dense tests and $0.95\%-11.5\%H$ for the dense tests, respectively.

- The failure mechanism of a shallow horizontal plate anchor resembled an inverted trapezoidal soil block bounded by a pair of distributed shear zones. The shear zones inclined outwards at an average angle to the vertical slightly larger than the dilation angle of sand. The shape of the shear zones became unsymmetrical for inclined plate anchors, and it can be closely represented by a proper combination of segments of logarithmic spiral curves and straight lines.

- A flow-type local shear failure mechanism was observed for deep anchors. At $\alpha=0^\circ$, sand particles flowed around two sides, which formed a triangular soil wedge above the horizontal plate anchor. For inclined plate anchors, the local shear failure pattern was rotational, stemming from the bottom edge towards the top side of the anchor. The embedment ratio beyond which the general shear failure converted to the local shear failure increased with sand relative density and load inclination to the vertical.

- Beyond the mobilisation displacements, the soil wedge became narrower with developments of post-peak shear bands. At sufficiently large anchor displacements, the deformation mechanism transitioned from the heave of a soil block to a primarily flow-around.

Finally, based on the test results and data collected from the literature, an empirical relationship was calibrated and recommended for prediction of pullout resistance of inclined plate anchors in sand.

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686 **Table 1** Properties of the test materials

Sample	Relative density (%)	D_r	Sand internal friction angle ϕ (°)	Dilation angle ψ (°)	Sand-anchor interface friction	
					angle ϕ_m (°)	
					Rough	Smooth
Dense sample	83%		40.3	19.7*	38.8	22.4
Medium-dense sample	50%		36.6	9.4*	36.3	21.1

687 * Estimated by the empirical correlation of Bolton (1986, 1987) under similar stress
688 levels in the model tests.

689 **Table 2** Summary of model test results

Test No.	α (°)	Anchor surface condition	H/h	D_r	N_{r-p}	δ_f (mm)	δ_f/H (%)	Failure mode
1	0	Smooth	1	50%	1.46	1.30	2.60	General
2	30				1.63	1.30	2.60	
3	45				2.06	1.43	2.86	
4	60				2.70	3.22	6.44	
5	90*				3.93	4.65	9.30	
6	0	Rough	1.57		1.23	2.46		
7	30		1.96		1.22	2.44		
8	45		2.23		1.49	2.98		
9	60		3.10		3.50	7.00		
10	90*		5.28		5.65	11.30		
11	0	Smooth	3	2.58	2.62	1.73	General	
12	30			2.73	2.65	1.77		
13	45			3.52	2.72	1.81		
14	60			4.38	8.70	5.80		
15	90*			9.11	14.40	9.60		
16	0	Rough	3	2.78	3.20	2.13	General	
17	30			3.18	3.40	2.27		
18	45			3.99	4.22	2.81		
19	60			4.89	9.50	6.33		
20	90*			10.59	17.70	11.80		
21	0	Smooth	5		3.83	9.85	3.92	Transition
22	30				4.32	10.16	4.06	
23	45				5.21	10.50	4.20	General
24	60				6.40	14.70	5.88	

25	90*			10.48	25.75	10.30	
26	0			3.93	10.80	4.32	Transition
27	30			4.77	9.47	3.79	
28	45	Rough		5.67	11.16	4.46	
29	60			6.81	14.10	5.64	General
30	90*			13.30	32.35	12.94	
31	0			5.79	14.30	4.09	
32	30			6.56	12.50	3.57	Local
33	45	Smooth		7.00	14.34	4.10	
34	60			8.90	16.88	4.82	
35	90*		7	13.60	35.00	10.00	transition
36	0			5.73	13.50	3.86	
37	30			6.64	14.17	4.05	Local
38	45	Rough		7.28	15.70	4.49	
39	60			8.82	17.25	4.93	
40	90*			13.99	37.45	10.71	transition
41	0			1.54	0.70	1.40	
42	45	Smooth		2.35	0.77	1.54	
43	90*		1	5.27	3.90	7.80	General
44	0			1.63	0.75	1.50	
45	45	Rough		2.52	0.93	1.86	
46	90*			6.13	5.05	10.10	
47	0			3.04	1.42	0.95	
48	45	Smooth		5.11	2.59	1.73	
49	90*		3	9.39	6.95	4.63	General
50	0			2.86	1.70	1.13	
51	45	Rough		5.30	2.76	1.84	
52	90*		83%	10.70	8.60	5.73	
53	0			5.54	2.60	1.04	
54	45	Smooth		6.48	5.50	2.20	
55	90*		5	12.44	24.30	9.72	General
56	0			6.12	3.20	1.28	
57	45	Rough		7.22	5.25	2.10	
58	90*			14.35	28.75	11.50	
59	0			7.32	3.70	1.06	
60	45	Smooth		9.09	7.95	2.27	
61	90*		7	15.60	32.20	9.20	General
62	0			7.54	3.70	1.06	
63	45	Rough		9.26	8.19	2.34	
64	90*			17.37	35.51	10.15	

690 * Data after the companion paper (Yue et al. 2020).

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Fig. 1. Schematic diagram of the experimental setup (unit: mm).

Fig. 2. Model plate anchors.

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Fig. 3. Normalized load-displacement curves: (a) $\alpha=0^\circ$ (b) $\alpha=45^\circ$ (c) $\alpha=90^\circ$.

Fig. 4. Break-out factors at various inclination angles: (a) $D_r=50\%$, (b) $D_r=83\%$.

Fig. 5. The ratio of break-out factors between the vertical and the horizontal anchors

Fig. 6. Cumulative displacement fields at mobilisation displacements of the smooth anchor with $D_r=50\%$.

Fig. 7. Cumulative displacement fields at mobilisation displacements of the smooth anchor with $D_r=83\%$.

Fig. 8. Instantaneous shear strain contours and instantaneous displacement vectors at the peak resistance of the smooth anchor at $H/h=7$ with $D_r=50\%$.

Fig. 9. Instantaneous deformation fields and displacement vectors at mobilisation displacements of the rough anchor at $H/h=5$ with $D_r=50\%$.

Fig. 10. Closed-up view of the cumulative velocity vectors at mobilisation displacements for anchors at $H/h=5$ ($D_r=83\%$): (a) $\alpha=0^\circ$, (b) $\alpha=90^\circ$.

Fig. 11. Total vertical displacement of soil at the top and bottom edges of the vertical plate anchor at $H/h=5$ with $D_r=50\%$.

Fig. 12. Shear zones close to the peak resistance with $H/h=5$ ($D_r=83\%$).

Fig. 13. Instantaneous shear strain contours and instantaneous displacement vectors at different anchor displacements in the dense tests at $H/h=5$: (a) $\alpha=0^\circ$ (b) $\alpha=45^\circ$ (c) $\alpha=90^\circ$.

Fig. 14. Variation of $(N_{\gamma-p}^{\alpha} - N_{\gamma-p}^{0^{\circ}}) / (N_{\gamma-p}^{90^{\circ}} - N_{\gamma-p}^{0^{\circ}})$ with $\alpha / 90^{\circ}$.

Fig. 15. Predicted versus simulated (by Bhattacharya and Kumar (2014)) break-out factors of inclined anchors: (a) $n=2.5$, (b) $n=3$.

Fig. 16. Predicted against measured (by Murray and Geddes (1989)) break-out factors of inclined anchors: (a) $n=2.5$, (b) $n=3$.