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Uplift response of circular plates as symmetrical anchor plates in loose sand

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Abstract. Uplift response of symmetrical circular anchor plates has been evaluated in physical model tests and numerical simulation using Plaxis. The behavior of circular anchor plates during uplift test was studied by experimental data and finite element analyses in loose sand. Validation of the analysis model was also carried out with 50 mm, 75 mm and 100 mm diameter of circular plates in loose sand. Agreement between the uplift responses from the physical model tests and finite element modeling using PLAXIS 2D, based on 100 mm computed maximum displacements was excellent for circular anchor plates. Numerical analysis using circular anchor plates was conducted based on hardening soil model (HSM). The research has showed that the finite element results gives higher than the experimental findings in the loose sand.

Keywords: uplift response; symmetrical anchor plate; circular plate; loose sand; numerical modelling; plaxis; FEM; Hardening Soil Model (HSM)

1. Introduction

The design of many structures need to foundation systems to resist vertical or horizontal uplift loads. As part of a larger effort to improve the performance of foundation systems, the development of guidelines for anchor system design and installation. The different structures like transmission towers, tunnels, sea walls, buried pipelines; retaining wall and etc are subjected to considerable uplift forces. In such cases, an absorbing and economic design solution may be obtained through the use of tension members. These elements, which are related to as anchors, are generally fixed to the structure and embedded in the ground to effective depth so that they can resist uplifting forces, will safety.

Many researchers have investigated the influence of different parameters on the uplift response of horizontal anchors in sand. Researchers such as Mors (1959), Giffels *et al*. (1960), Balla (1961), Turner (1962), Ireland (1963), Sutherland (1965), Mariupolskii (1965), Kananyan (1966), Baker and Konder (1966), Adams and Hayes (1967), Andreadis *et al*. (1981), Dickin (1988), Frydman and Shaham (1989), Remeshbabu (1998), Krishna (2000), Fargic and Marovic (2003), Merfield

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and Sloan (2006), Dickin and Laman (2007), Kumar and Bhoi (2008), Kuzer and Kumar (2009), Liu *et al*. (2012), Kame *et al*. (2012) and Zhang *et al*. (2012) were concerned with the general solution especially for an ultimate uplift capacity based on experimental works in sand. Also, many numerical studies have been carried out on the behavior of symmetrical anchor plates such as Meyerhof and Adams (1968) until the most recent analysis such as Kuzar ad Kumar (2009) are reviewed. This analysis was pioneered by Vesic (1971), Sarac (1989) and Smith (1998), Fargic and Marovic (2003), Merfield and Sloan (2006), Dickin and Laman (2007), Kumar and Kouzer (2008), Kuzer and Kumar (2009), Bowling (2012) and Kame *et al*. (2012). Increasing use of symmetrical anchor plates to resist uplift response may be achieved by increasing the size and depth of an anchor or the improvement of soil in which these anchors are embedded, or both.

In summary, most of the existing works in the literature are mainly focused on the capacity of symmetrical anchor plates embedded in normal soils with a horizontal ground surface. However, a few researches have been reported in the area of anchor plates embedded in different soil densities. On the other hand, to the knowledge of the authors, hardly any effort has been made so far to evaluate the performance of symmetrical circular anchor plate located on different soil densities. Therefore, the effect of soil densities on stability and rupture surface of the soil and, hence, the symmetrical anchor plate capacity is not clear. The current research describes insight into the effect of loose sand on the response of horizontal circular anchor plates that are embedded adjacent to a soil surface. The main objectives of the work are to study the loose sand for enhancing the ultimate uplift response of symmetrical circular anchor plate along with the influence of embedment depth, soil density, failure mechanism and break-out factors.

2. Model tests

2.1 Laboratory model tests

2.2 Model box

The cohesionless soil placement is particularly important such that during uplift tests. Similar cohesionless soil unit weights are obtained as a basis for comparing the influence of uplift parameters on the symmetrical anchor plate capacity. A sand unit weight at a value of 14.99 kN/m^3 was decided for sand in loose packing whereas sand at a loose condition was obtained using cohesionless soil raining method. Trial tests were run in order to predict the particular conditions that had to exist before the target unit weight can be achieved. For cohesionless soil in loose condition, trial tests indicated that there was a limiting sand thickness before a change in sand unit weight across the sand thickness becomes significant as the thickness increases. The standard cohesionless soil thickness was taken as 50 mm since this thickness gave a consistent value of unit weight when the cohesion less soil was rained from a certain height measured from the top of cohesion less soil layer. Regarding the sand-raining test, a range of failing cohesion less soil heights were employed in order to obtain the height required for the desired unit weight. The influence of cohesionless soil thickness on the unit weight was also found to be true for cohesion less soil samples that had to be compacted in order to achieve the desired unit weight.

Uplift tests were carried out in two test boxes covering two areas. The first test box is used for failure tests that carried out in a box covering an area of 600 mm × 250 mm and 450 mm deep with side glass walls to enable observation of cohesion less soil movement and its behavior. The second test box is used for uplift tests that carried out in a box covering an area of 1000 mm × 500

Fig. 1 First test box for failure test

mm and 1200 mm deep. Fig. 1 shows the first box for failure test. The loading frames were designed to suit the requirements of the tests.

2.3 Test materials

Several tests done to determine the properties of sand samples during experimental work. The tests included were:

(1) Particle size distribution using dry sieve method (BS 1377: Part 2: 1990)
(2) Maximum and minimum unit weight using vibratory table method (ASTM standards on soil compaction, 1993 Edition, Test designation D4254-91 and D4253-93)
(3) Direct shear test using small shear box (BS 1377: Part 7: 1990)
(4) Particle density using small pyknometer method (BS 1377: Part 2: 1990)

2.4 Particle size distribution

Particle size distribution test was done according to BS 1377: Part 2: 1990 using dry sieve method. This method covers the quantitative determination of the particle size distribution in a cohesionless soil down to the fine-sand size. For each sand type, 3 dry sieve tests were done with sieve sizes as follows:

Aperture size: 2.36 mm, 1.18 mm, 0.6 mm, 0.3 mm, 0.212 mm, 0.15 mm and 0.075 mm.

The sieve sizes used were considered adequate to cover the range of the sand type used for the experimental work. The sand sample is passed through the series of standard test sieves having successfully smaller sieve size. The weight of sand retained is each sieve is determined and the cumulative percentage by weight passing each sieve is calculated. Particle size distribution for sand type is presented as a curve on a semi-logarithmic plot, the ordinates being the percentage by weight of the particles smaller than the size given by the abscissa. The particle size distribution is shown below in Fig. 2.

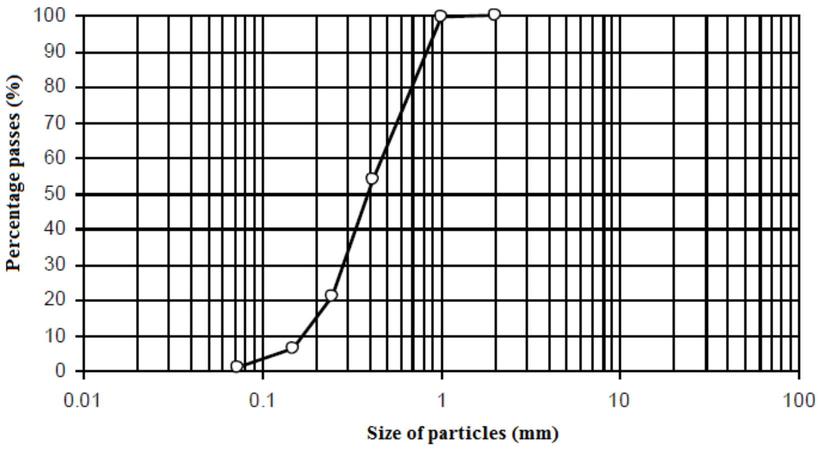

Fig. 2 Particle size distributions for sand sample

Table 1 Particle size properties of sand sample

Particle size properties of sand analysis	
	Particle size (mm)
D_{10}	0.17
D_{30}	0.32
D_{60}	0.55
C_u	2.8
C_c	1
Percent of middle sand	45.7%
Percent of fine sand	54.3%
Percentage of coarse sand	0%

Sand with particle sizes ranging from 0.2 to 0.6 mm is defined as medium sand whereas particle sizes of 0.6 to 2 mm are considered as coarse sand. The soils used were therefore classified as uniform medium sand with D_{50} = 0.50 mm. The sand properties from particle size distribution analysis are summarized below

2.5 Minimum sand unit weight

The minimum unit weight represents the loosest condition of a sand free draining soil that can be attained by laboratory procedure. The procedure essentially prevents bulking and minimizes particle segregation. In general, the particular procedure consist of determining the unit weight of oven-dried soil placed into a container of know volume in such a manner that it minimizes compaction of the soil.

The maximum unit weight of a given free draining soil is determined by placing an oven dried soil in a mold, and applying a dead weight to the surface of the soil. The mold, soil and dead weight are then vibrated vertically using an electromagnetic vibrating table for a certain period. The maximum unit weight is obtained by dividing the oven dried soil mass by its volume.

Relative unit weight expresses the degree of compactness of a cohesionless soil with respect to the loosest and densest condition as defined by laboratory steps.

Only when viewed against the possible range of variation, in terms of the relative unit weight can be the dry unit weight is related to the compaction effort used to place the soil in a compacted fill and the stress- strain tendencies of the soil when subjected to external loading. It is therefore generally recognized that the relative unit weight is a good indicator of the state of compactness of a given soil mass.

The maximum and minimum unit weight for sand sample was obtained through test with designation D4254-91 as recommended in ASTM standards on soil compaction (1993). Soil test in ASTM standards on soil compaction (1993) with designation D4254-91 recommends the standard test method for obtaining the minimum index density/ unit weight and calculation of relative density. Three alternative procedures were suggested to determine the minimum index density/unit weight, as follows:

(1) Test methods A - using a funnel pouring device or a hand scoop the deposit material in the mould,
(2) Test method B - depositing material into a mould by extracting a soil filled tube,
(3) Test method C - depositing material by inverting a graduated cylinder.

Test method A is the preferred procedure to be used in conjunction with ASTM standards on soil compaction (1993) designation D4253 whereas Test Methods B and C are for testing used in conjunction with special studies.

ASTM standards on soil compaction (1993) test designation D4253 are the recommended standard test methods for obtaining maximum index density and unit weight using a vibratory table. Four alternative procedures are suggested to determine the maximum index density and unit weight.

(1) Test method 1A - using oven dried soil and an electromagnetic vertically vibrating table.
(2) Test method 1B - using wet soil and an electromagnetic vertically vibrating table.
(3) Test method 2A - using oven dried soil and an eccentric or cam-driven vertically vibrating table.
(4) Test method 2B - using wet soil and an eccentric or cam-driven vertically vibrating table.

For the purpose of this study, test method 1A was used based upon available equipment. Results of maximum and minimum unit weight are presented below in Table 2 for sand sample.

2.6 Direct shear test using small shear box

The direct shear test includes the testing of a square prism of soil that is laterally restrained and

Table 2 Results of standard test methods for minimum unit weights of sand using a vibratory table

Uniform sand	First sample	Second sample	Third sample	Average unit weight
Minimum unit weight, γ_{min} (kN/m^3)	15.04	14.91	15.02	14.99

Table 3 Normal stress acting on sand sample during direct shear test

	1st sample	2nd sample	3rd sample	4th sample
Normal stress (kN/m^2)	27	54	82	112

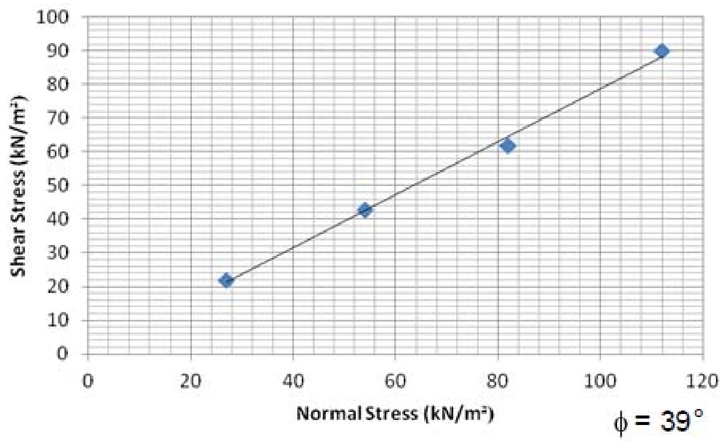

Fig. 3 Variation of shear stress τ, versus normal stress σ for direct shear tests

Table 4 Summary of soil properties and results of Ø from direct shear tests (BS 1377, Part 7: 1990)

	Loose sand
Unit weight (kN/m^3)	14.99
Ø (°) in plane strain condition	39
Ø (°) in 3-dimentional condition	38

sheared along a mechanically involved horizontal plane while subjected to a pressure applied along normal to the shearing plane. The shearing resistance is measured at regular intervals using at data logger and test were carried out with four samples using different normal pressures until the shearing resistance reaches a maximum value for which the soil can sustain. The relationship between measured shear stress failure and normal applied stress obtained would enable the effective shear strength parameters c and internal friction Ø be derived.

The normal stress acting on uniform sand in the direct shear test conducted for this research is given in Table 3.

Results of the direct shear tests conducted on uniform sand are given in Fig. 3. Derivation of the internal friction Ø is obtained from the slope variation of the shear stress τ with the normal stress σ.

A summary of the sand properties and results for the direct shear tests are presented in Table 4.

2.7 Particle density using small pyknomoeter method

The term particle density is used instead of the term specific gravity in this updated version of this particular standard (BS 1377, Part 2: 1990). There are basically three procedures subscribed, namely:

(1) Gas jar method: suitable for most soils involving those containing gravel-sized particles.
(2) Small pyknometer method: the more definitive method for soils consisting of clay, silt and sand-sized particles.
(3) Pyknometer method: suitable for most soils up to medium gravel size and the least accurate compared to the previous two methods.

The test for particle density was conducted using the small pyknometer method for the purpose of this research as the facilities were readily available and the method is suitable. Tests were done on three samples for sand and the average value for particle density is used. Result of the test can be found in Table 5.

Table 5 Summary of particle density test on uniform sand

Bottle No.	1st sample	2nd sample	3rd sample
Mass of bottle (w_1) gm	27.30	30.45	29.70
Mass of bottle+ sand (w_2) gm	34.75	39.10	38.64
Mass of bottle + sand + water (w_3) gm	82.35	86.45	85.30
Mass of bottle + water (w_4) gm	77.64	81.50	79.80
Mass of sand (w_2-w_1) gm	7.49	8.63	8.96
Mass of water whose volume is equivalent to sand (w_4-w_1)-(w_3-w_2) gm	2.78	3.70	3.46
Particle density of sand $\rho_s = \frac{w2 - w1}{(w4 - w1) - (w3 - w2)}$	2.64	2.63	2.64
Average particle density, ρ_s		2.64	

Fig. 4 Symmetrical circular anchor plates at UTM

Table 6 Summary of soil properties for tests previously undertaken

Conditions	Uniform sand
Loose	Ø = 38° G_s= 2.64 γ_{min} = 14.99 kN/m^3

Fig. 5 The setup of uplift test in UTM

2.8 Summary of soil properties

A summary of soil properties for all tests conducted as described beforehand is given below:

2.9 Model symmetrical anchor plate

Uplift test of symmetrical anchor plate geometry of the model circular plates which has been used anchorage. Model anchors with 10mm thick rigid plates are obtained. Experiments have done with diameters of 5.0, 7.5 and 10 cm that including different anchor plates as illustrated in Fig. 4.

2.10 Experimental test

The uplift test was conducted in the geotechnical laboratory in Universiti Teknologi Malaysia. The main treatment to be observed during experimental test is stress-displacement relationship during symmetrical anchor plate breakout. The test set-up uplift test steps are described in the following sections.

A schematic experimental set up is shown in Fig. 5 in the soil laboratory. The test boxes were used to contain cohesionless soil as embedment pattern. The model symmetrical anchor plates are connected to a pulling tendon cable for uplifting. A quasi static rate of pullout of approximately 1.5 mm/min was used for every test. This is to ensure that the symmetrical anchor plates surrounding element will have ample time to redistribute during uplift. Uplift capacity was measured by load cell attached to the pulling tendon cable during uplift test. A linear variable displacement transducer (LVDT) was placed at the top of the symmetrical anchor plate holder to measure the vertical displacement so as to predict the amount of symmetrical anchor plate movement required to mobilize the ultimate uplift capacity. A motor was connected to the pulling tendon cable via tendon steel cables. Datalogger was used to record data read from the load cell and LVDT.

2.11 Uplift test procedure

The uplift test takes into account only the net uplift capacity of the symmetrical anchor plates. This would mean that only the symmetrical anchor plates are involved in the analysis of symmetrical anchor plates uplift capacity. The test procedure for model symmetrical anchor plates tested in uplift included the following steps:

(1) Symmetrical anchor plate models to be tested are attached accordingly to the tendon cable which is then connected to the load cell holder. All apparatus included in the test are controlled for default before movement of symmetrical anchor plates in the test boxes. These controls includes as:
   (a) Inspection of test frame to ensure rigidity
   (b) Inspection of pulling tendon cable to ensure that it has not worn out
   (c) The test boxes are empty and free of cohesion less soil particles
   (d) The tendon cable connected to the load cell holder in firmly in place.
(2) Symmetrical anchor plate model to be tested is lowered slowly into the test box at the intended location marked before hand to ensure that vertical pullout is axially loaded. Symmetrical anchor plate is controlled again for vertically using the spirit level.
(3) Cohesionless soil is then placed in the box according to the placement method described beforehand.
(4) After the required height is reached, the surface layer is then flush and the load cell and LVDT are then placed into position.
(5) Calibration of load cell and LVDT had been done earlier such that only measurement of net uplift response and vertical displacement is fed to the datalogger.
(6) The datalogger is then started to take readings at certain intervals.
(7) Symmetrical anchor plate is considered to have undergone failure when a peak value of uplift response is deemed to have reached.
(8) The cohesionless soil used for testing is then weighted and calculated for its unit weight.

Test would have to repeat when the desired unit weight was not achieved. This involves disturbance to any part of the experimental set up during testing, human errors and power shortage that caused discontinuity of test being conducted. All factors should indicate that the data obtained from the test was reliable before the data is accepted for analysis.

The test procedure was considered adequate to cover the range of parameters under study and to systematically isolate the effects of a certain parameter on the uplift capacity. This would enable critical of the experiments and numerical simulations conducted and provided a basis for

Fig. 6 Set-up of failure mechanism in UTM

comparison.

2.12 Failure mechanism

The failure mechanism tests were performed in Fig. 6. In these tests, patterns were made on the extreme uplift loads and embedment ratio. The aim of these tests was to show the behavior of failure mechanism of loose sand and dense around symmetrical anchor plates due to uplift test.

The properties of test were applied to unit of weight 14.99 kN/m^3 obtaining loose sand. Every 50 mm vertical intervals were involved 4 mm dyed in such a way that sand was placed on front face of failure box in need to visual line. Loading was applied to the circular anchor plates through loading cable with a constant rate of low in loose sand. The failure pattern was shown during the testing. The symmetrical anchor plates were made to move until sufficient distance was achieved, to ensure the failure pattern was showed.

2.13 Breakout factor

The main parameters of collapse load which may act on soil parameters are those due to unit weight of sand, internal friction, symmetrical anchor plate's embedded depth and size of symmetrical anchor plates. In full scale model analysis, equation of those parameters may be expressed in dimensionless quantity stated as below

$$f_1(P, L, D, \varnothing, \gamma) = 0 \tag{3}$$

f_1 may be expressed as f_2,where

$$f_2(\pi_1, \pi_2, \pi_3) = 0 \tag{4}$$

Since the Ø is dimensionless unit, thus

$$\pi_1 = \text{Ø} \tag{5}$$

Then

$$P = f(L, D, \gamma)$$

$$P = L^{\propto} D^{\beta} \lambda^{c}$$

$$MLT^{-2} = (L)^{\propto} (L)^{\beta} (ML^{-2}T^{-2})^{c}$$

$$\propto = 1, \beta = 2, c = 1$$

Then

$$\begin{aligned} P &= LD^2\gamma \\ \pi_2 &= P / LD^2\gamma \end{aligned} \tag{6}$$

L and *D* have the same dimensional form, so

$$\pi_3 = L / D \tag{7}$$

Thus

$$f_1(\text{Ø}, P / LD^2\gamma, L / D) = 0 \tag{8}$$

$$\begin{aligned} \frac{P}{LD^2\gamma} &= f\left(\text{Ø}, \frac{L}{D}\right) \\ P &= f\left(\text{Ø}, \frac{L}{D}\right) \times LD^2\lambda \end{aligned} \tag{9}$$

Where *P* is ultimate uplift load obtained from test, *D* is width of anchor plate, *H* is embedded depth of anchor plate, γ is dry unit weight, Ø is internal friction angle and *L*/*D* is embedment ratio. Internal friction angle is constraint for the test.

3. Numerical simulation models

A series of two-dimensional finite element analyses (FEA) on a prototype symmetrical anchor plate - sand system was performed in order to assess the experimental model tests results and find out the deformations behavior within the sand body. The analysis was performed under the finite element program, Plaxis package (professional version 8, Bringkgreve and Vermeer, 1998). Plaxis is geotechnical software that can be analyzed the soil problems. In general, the initial conditions comprise the initial groundwater conditions, the initial geometry configuration and the initial effective stress state. The sand layer in this research was dry, so there was no need to enter ground water condition. The analysis has done by means of Hardening Soil Model (HSM). The geometry of the prototype anchor plate-box system was supposed to be the same as the experimental model. The same gradient of model test and the material of steel plate for symmetrical anchor plate and sand were used in the prototype research. Table 7 and Table 8 illustrate the sand, geogrid and plate properties used.

Table 7 Material properties used in Plaxis

Parameter value	Loose packing
Cohesion, c (kPa)	0.5
Residual angle of internal friction (°)	38
Angle of dilatancy (Ψ°)	8
Unit weight, γ (kN/m^3)	14.99
Secant stiffness, E_{50} (kN/m^2)	20000
Initial stiffness, E_{OED} (kN/m^2)	20000
Unloading/reloading stiffness, E_{UR} (kN/m^2)	60000
Poisson's ratio	0.2
Power for stiffness stress dependency, (m)	0.5
At rest earth pressure coefficient, K_0	0.38
R_{inter}	0.9

Table 8 Steel plate properties

Type	Steel plates
EI	163 kNm²/m
EA	3.4×10^5 kN/m

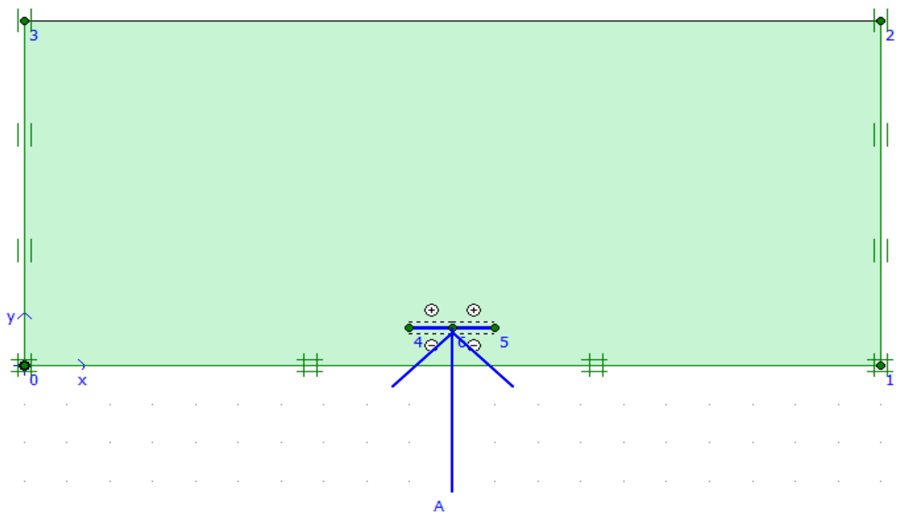

Fig. 7 The model geometry on prototype in Plaxis

A variety of sand models are made in the computer code chosen for this research. It was used the Hardening Soil Model (HSM) criteria to model the sand for its analysis, practical importance and the availability of the parameters needed. The interaction between the symmetrical anchor plates, geogrid and sand was modeled by means of interface elements, which enabled for the specification of a decreased wall friction compared to the soil friction. The parameters used for numerical simulation are depicted in Tables 7 to 8. The model geometry based on finite element method by means Plaxis verified for the analysis is shown in Fig. 7. The left vertical line of the geometry model was constrained horizontally but the bottom horizontal boundary was constrained in both the horizontal and vertical directions. The prescribed load was loaded on increments accompanied using iterative analysis up to failure. The boundary conditions presented that the vertical boundary is free vertically and constrained horizontally until the bottom horizontal boundary is completely fixed. The program can be the automatic produce of elements for the sand and three node tensile elements for the symmetrical anchor plate. The analyzed geometry and produced mesh and related boundary conditions are shows in Fig. 7.

4. Results

This part presents the results of uplift experiments and models conducted for the uplift test. Uplift force-displacement relationship of symmetrical anchor plates when subjected to uplift were recorded and subsequently analyzed based on peak uplift resistance of every particular test and simulation models by finite element method using PLAXIS. The discussion involves the numerical and experimental aspects of net symmetrical anchor plate capacity during uplift test and symmetrical anchor plate displacement on loose sand. A rational basis for the behavior of symmetrical anchor plates in studied in soil failure mechanism studies conducted to obtain evidence on shape and extent of soil shape failure when subjected to varying parameters. Sand was used as an embedment medium in this research. A loose packing was achieved by sand raining methods. Results obtained from test conducted from numerical and experimental results will be discussed in this part. The effect of embedment ratio, break-out factor, and failure mechanism patterns of models are detailed on loose sand in numerical and experimental tests. The results were collected and presented in many curves. The failure mechanism patterns of models in loose sand and dense sand observed based on experimental and numerical analysis in this part. A summary of uplift test result in presented in Table 9 for symmetrical circular anchor plates based on loose sand in the simulation and experimental work.

The discussion of uplift capacity would deal with the parameters of symmetrical anchor plate's sizes, sand packing and embedment ratio separately. This is to enable an impartial and focused review of the effects of each parameter on the symmetrical anchor plate during uplift in loose sand.

With reference to Fig. 8, symmetrical anchor plates experienced an increase in uplift capacity

Table 9 Summary of uplift capacity result (L/D = 4)

Symmetrical Anchor Plate	Uplift capacity in loose sand (N)	
Types	Loose sand	
	Lab	Plaxis
Circular		
Diameter = 5 cm	150	184
Diameter = 7.5 cm	339	414
Diameter = 10 cm	644	788

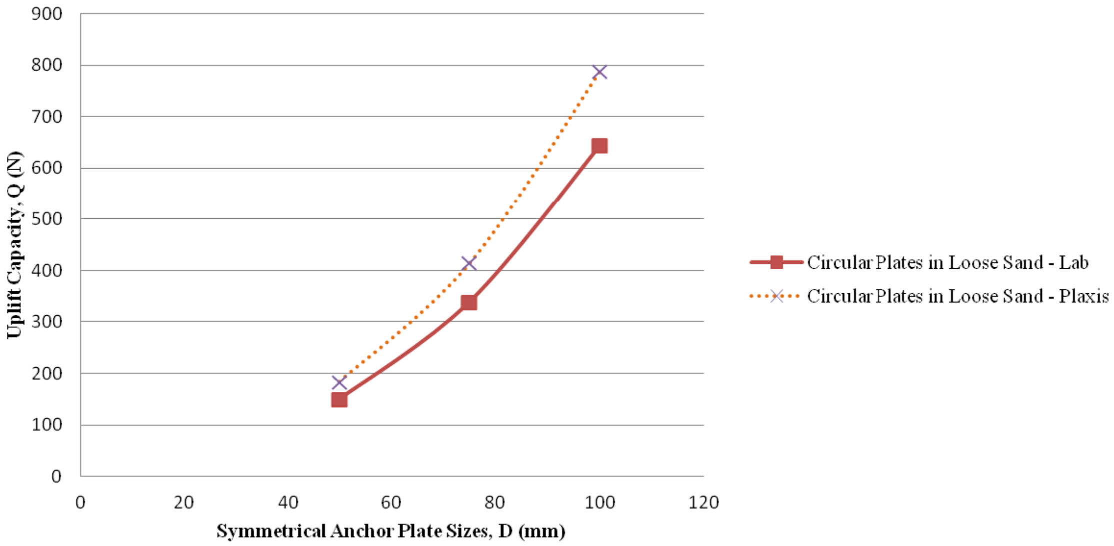

Fig. 8 Variation in uplift capacity *Q* with symmetrical anchor plate size *D* for circular

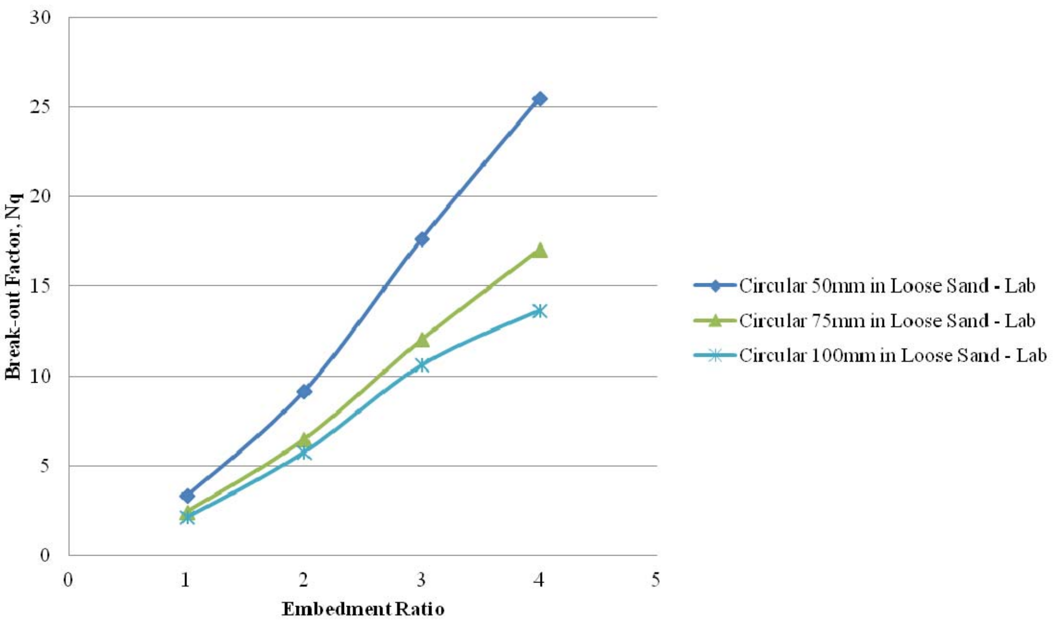

Fig. 9 Variation of break-out factor Nq with embedment ratio *L*/*D* for symmetrical circular anchor plates in both loose conditions in laboratory

for every increase of symmetrical circular anchor plate's size. From Fig. 8 as shown below, the significant trend to note is a decrease of percentage increases in uplift capacity with symmetrical anchor plate's size for test conducted. This is related to the trend of the percentage increase in

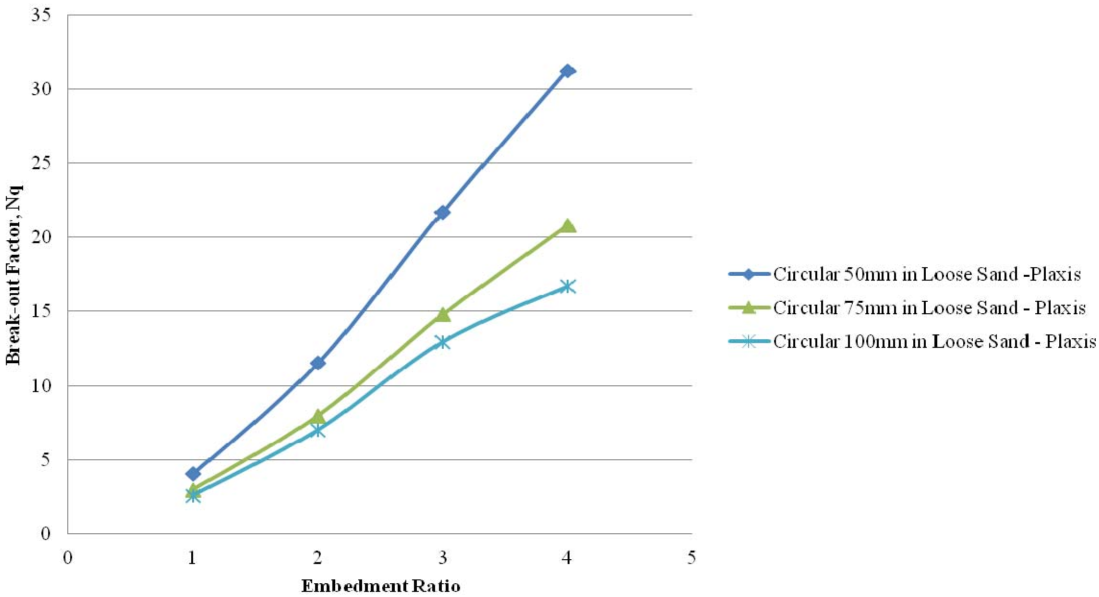

Fig. 10 Variation of break-out factor Nq with embedment ratio *L*/*D* for symmetrical circular anchor plates in both loose conditions in Plaxis

Fig. 11 Initial state of sand before commencement of uplift in loose sand

symmetrical anchor plate's size with increasing depth to be discussed subsequently in previous sections.

With regard to Fig. 9, symmetrical anchor plates experienced an increase in uplift capacity for every increase of embedment ratio in symmetrical anchor plate. As seen from Fig. 10, symmetrical anchor plates in maximum embedment ratio, *L*/*D* = 4, had higher uplift capacities than symmetrical anchor plates in minimum embedment ratio such as *L*/*D* = 1.

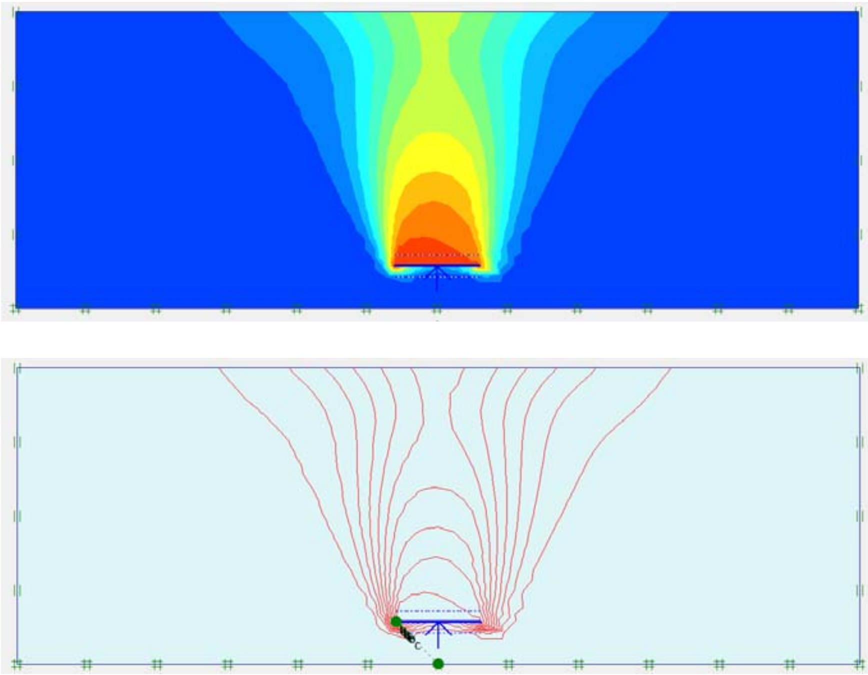

Fig. 12 State of sand after commencement of uplift in loose sand

4.1 Failure mechanism studies

Studies on uplift failure mechanism have shown that symmetrical anchor plates fail with a curved shear surface. An example of this is shown from Figs. 11 to 12. The figures illustrate the shear failure mechanism during uplift for symmetrical anchor plates in loose sand. The condition of sand surrounding the symmetrical circular anchor plates before uplift is illustrated in Fig. 11. At the actual moment when the uplift capacity was reached as illustrated in Fig. 12, the deformation experienced by the sand indicated a proponent failure surface. A certain degree of collapse was observed to have occurred near the symmetrical circular anchor plates. With further uplift movement, the failure surface was seen to be defined more prominently. The final failure surface is seen much clearer in Fig. 12 where a curved shaped localized failure was observed to have occurred when the symmetrical anchor plate was pulled out at constant rate. A contributing factor towards the formation of the curved shaped failure would be the collapse of soil around the symmetrical anchor plate to fill in the void formed near the symmetrical anchor plate's bottom.

5. Discussion

This part presents a comparison of theoretical and experimental values for the experimental and numerical programme conducted. The literature review has explained previous theoretical research results, which were dedicated to the limiting ultimate uplift capacity of symmetrical anchor plates, their breakout factor and failure zones. Researchers such as Balla (1961), Meyerhof and Adams

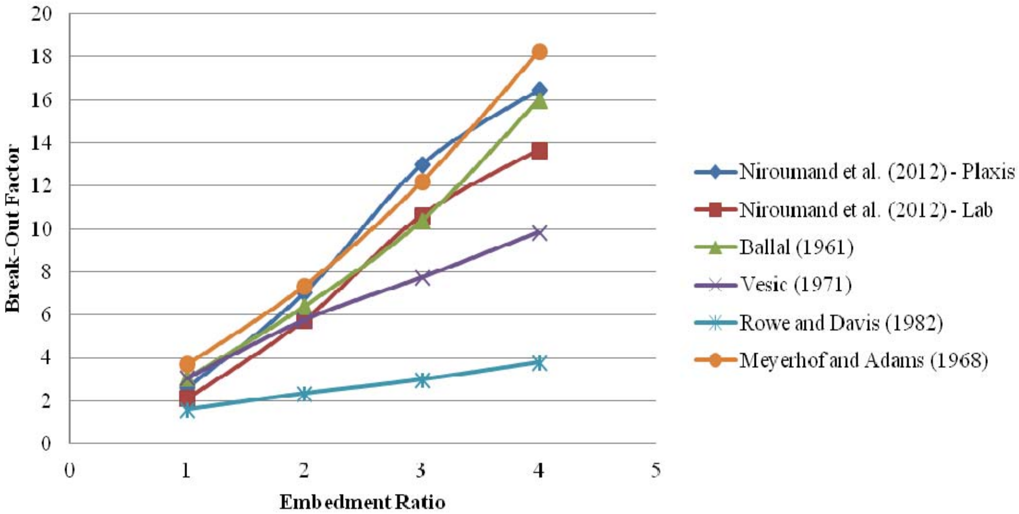

Fig. 13 Comparison of break-out factor between experimental results and theoretical and numerical prediction for circular anchor plates in loose packing

(1968), Vesic (1971), Murray and Geddes (1987), Sarac (1989) and Smith (1989), Fargic and Marovic (2003), Merfield and Sloan (2006), Dickin and Laman (2007), Kuzer and Kumar (2009) and Adhami *et al*. (2012) determine their parametric relationship for ultimate uplift capacity of anchor plates and their breakout factor.

Researchers like, Balla (1961), Meyerhof and Adams (1968), Vesic (1971), Murray and Geddes (1987), dedicated their works in proposing the theories of horizontal anchor plate subjected to uplift loads. This part presents a comparison of existing theories for current research conducted. Fig. 13 illustrate a comparison of theoretical and experimental values as forwarded by various researchers such as Balla (1961), Meyerhof and Adams (1968), Vesic (1971), Murray and Geddes (1987) and current research. The difference between each theoretical prediction lies in the value of the breack-out factor in uplift or the like. Balla (1961), Meyerhof and Adams (1968), Vesic (1971), Murray and Geddes (1987) proposed theoretical values based on the curved failure model by the method of analytical and experimental evaluation in loose sand. Fig. 16 illustrates comparison of theoretical break-out factor values and current results based on experimental and numerical analysis. The overall trend indicate that for the series of tests and models conducted, experimental and numerical values are in close agreement and similar to values of Balla (1961) for circular plates.

6. Conclusions

A parametric research was conducted to obtain knowledge on symmetrical anchor plates, soil conditions and behavior of the symmetrical anchor plates during uplift. Although it is not dedicated to any specific practical conditions in engineering practice, it is useful to study it various effecting factors that influence the symmetrical anchor plate's capacity when subjected to uplift forces. The failure shape for symmetrical anchor plates with embedment ratio *L*/*D* up to 4 is cylindrical despite variation in size, density in sand when subjected to uplift loads.

In the selection of symmetrical anchor plate's depth to achieve an economic anchor plate uplift design, the size and depth are important parameters to be taken into consideration. It would therefore be more economical and rational to increase the uplift capacity of symmetrical anchor plates by increasing symmetrical anchor plate's depth. Increasing symmetrical anchor plate's depth would help to increase uplift capacity more significantly compared to symmetrical anchor plate's size by the increase the anchor plate contact area with the sand.

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