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Uplift Capacity of Suction Caissons

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Summary: Prediction of the uplift capacity of suction caissons is a critical issue facing design engineers and rational methods are required in order to produce reliable designs. Extensive theoretical investigations have been carried out of suction caissons subjected to vertical or inclined uplift loading for cases where the behaviour of the seabed soil is undrained, partially drained or drained. A brief literature review on foundations subjected to combined loading is included. Different analytical design models are discussed. Simplified methods for the estimation of the uplift capacity are described, based on the results of the finite element study. The simplified methods are then validated by upper bound theoretical solutions and experiment results. The expressions developed in this paper take into account the influence of the aspect ratio of the caisson, the point of application and angle of inclination of the loading, the undrained shear strength of the soil, the soil permeability and load rate.

1 INTRODUCTION

Compliant offshore structures, like mooring systems and tension leg platforms (TLPs), are usually subjected to considerable uplift forces. These structures require foundations that can anchor them to competent strata and it has been common in the past to use piles to provide such a foundation. However, there are some construction difficulties associated with the installation of the long piles usually necessary, particularly in large depths of water and in some soil type. Largely because of these difficulties a new type of foundation, the suction caisson, has been developed and used to provide uplift resistance, depending on the in situ conditions. A suction caisson, open at the bottom and closed at the top, is designed to penetrate to the sea floor by its own weight and sometimes by also creating an inside under-pressure relative to the outside water pressure. The latter is known as the active suction installation method. As soon as there is any tendency to pullout movement, the suction caisson mobilises significant pullout capacity through the development of negative pore water pressure inside the soil plug and at the bottom of the caisson. This is known as the passive suction condition. The main advantages of suction caissons over tension piles are: the ease of installation of the caissons with the active suction arrangement; the mobilization of passive suction forces at the caisson's bottom during uplift; and the possibility of placing additional ballast on the large diameter sealed top to provide increased pullout capacity. To date, the geometry of suction caissons has tended to involve relatively low aspect ratios, with length less than 3 times the diameter.

The main focus of this paper is on a) suction caissons subjected to vertical uplift loading for cases where the behaviour of the seabed soil is undrained, partially drained or drained, and b) suction caissons subjected to inclined uplift loading for cases where the behaviour of the seabed soil is undrained. The two cases are illustrated in Fig.1. The soil types assumed in this investigation are normally consolidated to lightly over-consolidated cohesive soils.

The theoretical investigations described in this paper have been carried out using a 3-D semi-analytical finite element approach (Taiebat, 1999) incorporated in the software package AFENA (Carter et al., 1995). Simplified prediction methods were developed based on the finite element results. Predictions of the ultimate capacity by the simplified methods have been compared to those obtained independently from upper bound techniques (Randolph et al, 1998) and experimental measurements in 76 individual tests from 12 independent studies. The solutions developed in this study are both for quasi-horizontal and quasi-vertical loading. The developed expressions have also taken into account the influence of the aspect ratio of the caisson, the point of application and angle of inclination of the loading, the undrained shear strength of the soil, the soil permeability and the loading rate.

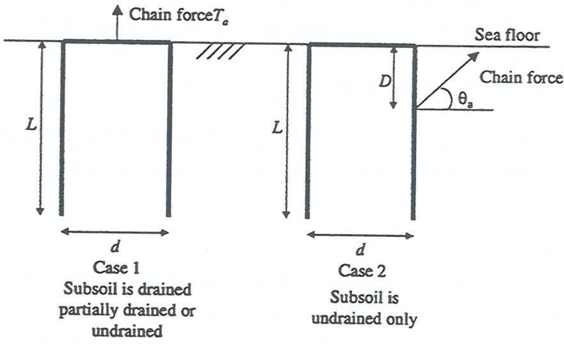

Figure 1 Study cases

2 FOUNDATIONS SUBJECTED TO COMBINED LOADING

The response of a foundation system subjected to combined loading has been a topic of much interest to geotechnical researchers and practitioners, particularly over the last decade. Research in this area has mainly involved investigation of bearing capacity problems. The bearing capacity failure locus for rigid shallow foundations subjected to combined loading has been derived on the basis of experimental results using curve fitting. For example, Butterfield and Gottardi (1995) investigated the behaviour of footings on sand under fully drained or partially drained conditions. Martin (1994) investigated the behaviour of a spudcan footing in undrained clay. In these cases, the footings either had a flat base or a spudcan shape and no consideration was given to the resistance to uplift loading. For these cases, it was found that maximum moments and horizontal loads are sustained in the presence of some vertical compressive load, i.e. typically when $V/V_o \approx 0.4$ to 0.5, where, V is the vertical component of the ultimate load and V_o is the ultimate load for cases of purely vertical loading.

It is evident from many of the previous studies of combined loading that a yield or failure locus relating the vertical (V), moment (M) and horizontal (H) loads at the ultimate condition can often be expressed in the form:

$$f\left(\frac{V}{As_u}, \frac{M}{Ads_u}, \frac{H}{As_u}\right) = 0 \qquad (1)$$

where, A is the plan area of the foundation, d is the diameter of the foundation, s_u is the undrained shear strength of the soil at the base of the foundation. It is usually assumed that associated plasticity provides a reasonable description of undrained failure in the soil, so that this yield surface also describes a plastic potential defining the relative magnitudes of the incremental deformation during elasto-plastic yielding. To apply the normality relationships on the yield surface, the load and displacement definitions must form work conjugate pairs so that the normalised total system work, W, is written as:

$$\frac{W}{Ads_u} = \left(\frac{V}{As_u}\right)\frac{\delta v}{d} + \left(\frac{M}{Ads_u}\right)\delta\theta + \left(\frac{H}{As_u}\right)\frac{\delta h}{d} \qquad (2)$$

where δv, δh and $\delta\theta$ are the incremental vertical and horizontal caisson displacements and its rotation at failure. These incremental displacements are measured at the same point at which the loads are assumed to act.

In a study of multi-footing foundation systems, Murff (1994) suggested a general form of the failure locus, which included some uplift capacity, given as:

$$f = \sqrt{\left(\frac{M}{d}\right)^2 + \Lambda_1 H^2} + \Lambda_2\left[\left(\frac{v}{v_c}\right)^2 - \left(1 + \frac{V_t}{V_c}\right)V + V_t\right] = 0 \qquad (3)$$

in which V_t V_c are the ultimate vertical compression and tension capacities, Λ_1, Λ_2 are constants and d is the footing diameter. It was assumed that this is a form of associated yield or failure surface. However, as indicated by Bransby and Randolph (1997), the locus described by equation (3) gives very poor agreement with numerical predictions of the collapse loads for strip footings when $M = 0$.

The response of a suction caisson (skirted foundation) to combined vertical (V), moment (M) and horizontal (H) loading has been studied for bearing problems by Bransby and Randolph (1997) using a two-dimensional finite element analysis. In this study, the caisson was considered as a long strip footing and one value of the aspect ratio, $L/d = 0.167$, was investigated in detail. On the basis of the plane-strain finite element predictions, they suggested a yield locus as:

$$f = \left(\frac{V}{V_0}\right)^{2.5} - \left(1 - \frac{H}{H_0}\right)^{1/3}\left(1 - \frac{M}{M_0}\right) + \frac{1}{2}\left(\frac{M}{M_0}\right)\left(\frac{H}{H_0}\right)^5 \qquad (4)$$

However, in practice suction caissons are often circular in plan. To obtain a more precise understanding of the behaviour of a circular foundation under vertical, moment and horizontal loading, a three-dimensional analysis is required. Further, the study by Bransby and Randolph (1997) produced a yield locus for problems involving only bearing (compressive) loads. No consideration of the resistance to uplift loading was included.

3 ANALYTICAL DESIGN MODELS

3.1 Bottom Resistance Failure

When a suction caisson is pulled out at a rapid rate and when large deformation takes place, it may be completely pullout out with the soil plug inside. In this case, the bottom resistance (passive suction) and whatever tensile strength of the clay may have are fully mobilized at the bottom of the caisson. The pullout capacity in this case is given by the frictional resistance and the weight of suction caisson, plus the resistance at the bottom of the suction caisson (passive suction) (Fig.2). In this case the ultimate pullout force is given by:

$$P_u = F_s + W + R_b \qquad (5)$$

where, Fs is the skin friction on the wall, W is the underwater weight of the foundation which includes the soil plug, and R_b is the bottom resistance (passive suction).

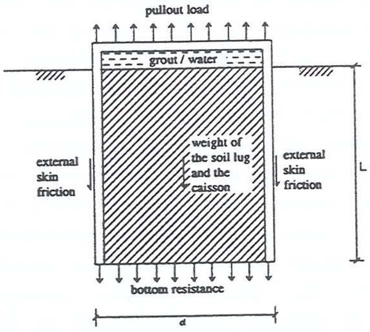

Figure 2 Bottom resistance failure

3.2 Reversed Bearing Capacity Failure

Considering the failure in uplift as a reversed bearing capacity problem is a widely used approach for estimating the pullout capacity of suction caissons. This approach was firstly introduced by Finn & Byrne (1972) after performing laboratory model tests to understand the factors governing the pullout capacity of suction caissons. This idea was then further verified and enhanced by other researchers (e.g., Andersen *et al*, 1993). The failure mechanism of this model is shown in Fig.3.

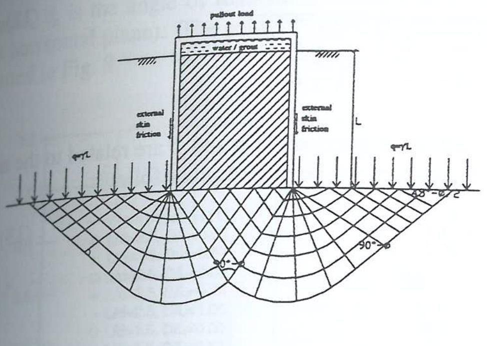

Figure 3 Reversed bearing capacity failure

In this case, the pullout capacity of circular suction caissons under vertical and static loads may be estimated by the following formula:

$$P_u = 1.2AN_c s_u d_c + F_{ts} \tag{6}$$

Where, N_c is the bearing capacity factor with respect to cohesion s_u of the soil. For $\varphi = 0°$ the theoretical value is 5.14 for N_c. d_c is the embedment factor. A is the section area of the caisson. F_{ts} is the friction between the external surface of the caisson wall and the soil.

3.3 Sliding Failure

When a suction caisson is pulled out at an very slow rate, the pullout capacity can be given by the frictional resistance and the weight of suction caisson (Fig.4), such as:

$$P_u = F_s + W_f \tag{7}$$

Where, F_s is the skin friction on the wall, W_f is the submerged weight of the foundation. This failure mode assumes negligible bottom resistance and is more likely to occur under fully drained conditions.

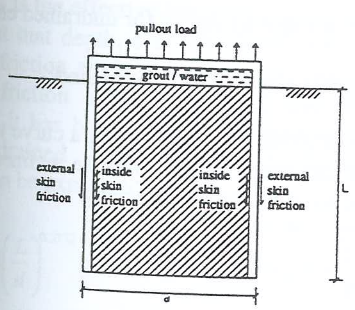

Figure 2 Sliding failure

3.4 Conical Wedge Failure

A possible failure mechanism for suction caisson subjected to inclined uplift loading is illustrated in Fig. 5. r_0 is the radius of the caisson (d is the diameter of the caisson), R is the radius of the deforming wedge, Z_0 is the depth of deforming wedge, h is the depth of the centre of the rotation of the caisson, D is the lug depth, and L is the length of the caisson. Near the surface, a deforming conical wedge forms and is pushed laterally and upwards by the translating and possibly rotating caisson. Below the wedge, the soil is assumed to flow horizontally around the caisson. To accommodate this mechanism, the soil wedge must conform to the caisson at the caisson-soil surface, and must move tangentially to the right soil-caisson interface.

This failure mechanism was firstly developed by Murff & Hamilton (1993) for laterally loaded piles and was then used by Randolph et al (1998) for suction caissons subjected to inclined uplift loading. Similar mechanisms have also been detected in the finite element modelling.

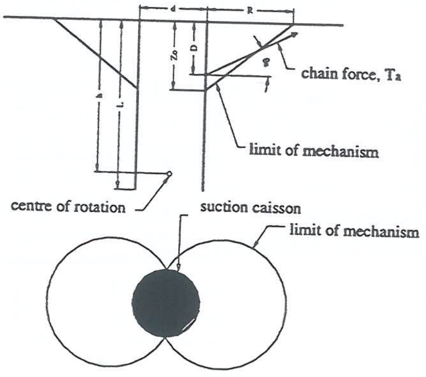

Figure 3 Conical wedge failure

4 THEORETICAL SOLUTIONS

Studies by 2-D and 3-D finite element modelling have been carried out for different aspect ratios of the caissons, different soil strength profiles, different soil permeability and different loading rate. The simplified expressions for the estimation of uplift capacity of suction caisson were then developed by curve fitting the finite element predictions. Details of the finite element study and the detailed derivation of the theoretical solutions have been presented elsewhere in Deng & Carter (1999a) and Deng & Carter (1999b).

4.1 Vertical Uplift Capacity - Undrained

The vertical uplift capacity can be estimated by a modified form of the equations governing the reversed bearing capacity problem. The ultimate uplift capacity can be expressed as the ultimate value of the average uplift traction, p_u, applied at the top over the caisson area, $A = \pi d^2/4$. In this case p_u will be the sum of two terms, one representing the effective overburden pressure at the level of the caisson tip, and the other depending on the undrained shearing resistance of the soil, i.e.,

$$p_u = 1.2N_p d_c s_{u(tip)} \tag{8}$$

where L is the embedded length of the caisson and d is its diameter. $s_{u(tip)}$ is the undrained shear strength of the soil at the depth of tip of the caisson.

The uplift capacity factor, N_p, which has taken account of the effects of the bottom resistance and the soil-wall friction, may be expressed as: (Deng & Carter, 1999a)

$$N_p = 7.9\left(\frac{L}{d}\right)^{-0.18}. \qquad (9)$$

where L/d is the aspect ratio. In equation (8), d_c is the usual embedment factor for undrained bearing capacity, given by $1+0.4(L/d)$. The theoretical relationship between N_p and L/d (or s_{utip}/kd, where k is the undrained strength gradient with the depth) has been plotted in Fig. 6, together with the lower bound solutions for bearing capacity factor N_c suggested by Houlsby and Wroth (1983). There is good agreement between the two analytical estimates of N_p and N_c.

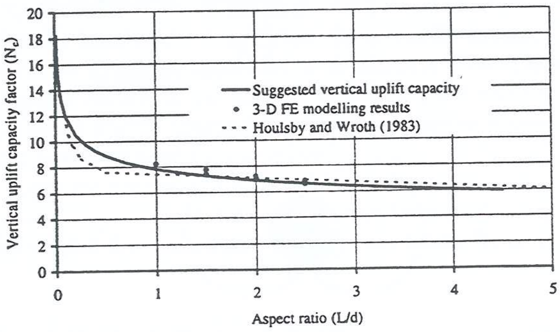

Figure 4 Vertical uplift capacity ratio

Clearly, the ultimate vertical uplift load can be expressed simply as:

$$V_u = p_u A \qquad (10)$$

4.2 Lateral Capacity – Undrained

Previous investigators (e.g., Randolph and Houlsby, 1984; Murff and Hamilton, 1993) have carried out studies of the ultimate behaviour of piles in clays under lateral loading. These studies were of pile segments or complete piles subjected to lateral loading applied at the top of the pile, and hence the solutions do not take into account the influence of the relative depth to the loading point, D/L (where D is the depth of the load application point from the soil surface).

Studies by Deng and Carter (1999b) have shown that a good approximation for the ultimate horizontal load that may be applied to a suction caisson, H_u, can be expressed approximately as:

$$H_u = N_h A s_{u(2/3L)} \qquad (11)$$

where, $s_{u(2/3L)}$ denotes the undrained shear strength at a depth equivalent to 2/3 of the caisson length. It was also found that N_h could be described by an expression of the form

$$N_h = \frac{\alpha}{\sqrt[4]{\left(\frac{\alpha}{6.3}-\frac{D}{L}\right)^4+\left(\beta\frac{D}{L}\right)^4}} \qquad (12)$$

The dimensionless parameters α and β are related to the aspect ratio, L/d, and they can be expressed as follows

$$\alpha = 7.02\left(\frac{L}{d}\right)^{-0.3785} \qquad (13)$$

and

$$\beta = 1.58e^{-0.775\left(\frac{L}{d}\right)} \qquad (14)$$

The above solutions are applicable to the case of soil strength increasing linearly with depth (e.g., normally consolidated soils). The method may also be applied to cases where foundations are embedded in overconsolidated soils. In the latter case, the soil strength $s_{u(2/3L)}$ appearing in expression (11) should be replaced by the strength of the soil at the depth at which only horizontal displacement occurs when horizontal load is applied.

The above solutions are plotted in Fig. 7 for cases where the aspect ratio, L/d, is 1.0, 1.5, 2.0 and 2.5.

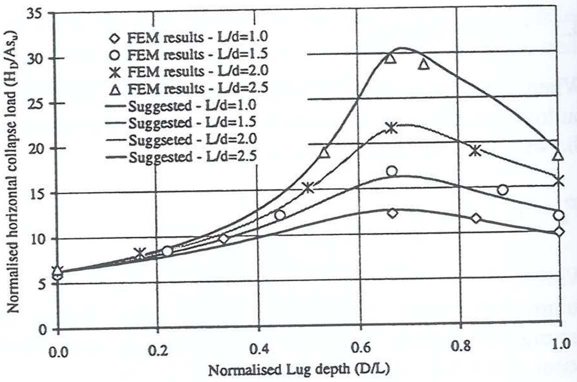

Figure 5 Lateral capacity factor for undrained conditions

4.3 Inclined Uplift Capacity – Undrained

Deng and Carter (1999b) found by fitting a curve to the results of finite element analyses that the ultimate inclined uplift load can be expressed to sufficient accuracy as:

$$\frac{V}{V_u}+\left(\sqrt{1-\left(\frac{H}{H_u}\right)^2}-1\right)^2-1=0 \qquad (15)$$

in which V_u is the ultimate value for purely vertical load, given by equation (10), and H_u is the ultimate lateral resistance for purely horizontal load applied at a lug depth, D. The value of H_u is given by equation (11). V and H are the vertical and horizontal components of the ultimate inclined load applied to the axis of the caisson, and obviously,

$$\frac{V}{H} = \tan\theta_a \qquad (16)$$

where, θ_a is the angle of load inclination (with reference to the horizontal plane). The solution given by equation (15) is plotted in Fig. 8.

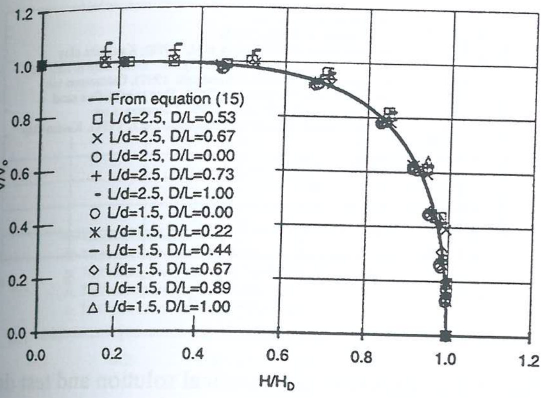

Figure 6 Inclined uplift capacity (undrained)

In practice, load is usually applied to anchor caissons by the attachment of a chain to a lug fixed to the sidewall of the caisson. A load applied directly to the caisson wall at a depth D is statically equivalent to the case of the same load applied at the centre line, at a depth of $D+(d/2)\tan\theta_a$. Therefore, the failure load for the more usual case of sidewall attachment can be obtained from equation (15), after making appropriate allowance for the location of the fixing point.

4.4 Vertical Uplift Capacity – Fully Drained

In cases where the subsoil is fully drained, the pullout capacity of the caisson is equal to the total friction developed between the soil and the caisson wall. At any depth h, the friction between the soil and the caisson wall, f_s, can be expressed as:

$$f_s = \sigma_h{}'\tan\delta \qquad (17)$$

where, σ'_h is the effective horizontal stress acting on the soil element at that depth, δ is the interface friction angle. The interface friction angle may be influenced by the soil type (internal friction angle and *OCR*) and the aspect ratio. Therefore it has been suggested (Deng and Carter, 1999a) that the drained vertical uplift capacity of a cylindrical caisson can be expressed as:

$$p_{u(net)} = 9.1\left(\frac{L}{d}\right)^{0.5372}(1-\sin\phi')(OCR)^{\sin\phi'}\tan\phi'\sigma'_{v(bottom)} \qquad (18)$$

where, γ' is the effective unit weight of soil, ϕ' is the effective friction angle, *OCR* is the over-consolidation ratio, and $\sigma'_{v(bottom)}$ is the vertical effective stress at the location of bottom of the caisson.

Equation (18) has considered the friction along both the external and the internal caisson wall. The effects of the the relationship between the interface friction angle, δ, and the soil friction angle, φ´ and the correlation of horizontal effective stress in the soil to the original in situ effect stress have also been reflected on.

4.5 Vertical Uplift Capacity – Partially Drained

The pullout capacity of suction caissons subjected to uplift loading under partially drained conditions can be estimated by the following formula (Deng & Carter, 1999a):

$$p_{u(net)} = N_f\, p_{u(drained)} + N_b s_{u(tip)} \qquad (19)$$

where

$$N_b = [0.13 - 0.446\ln(T_k)]1.6^{\frac{L}{d}} \qquad (20)$$

$$N_f = 0.632 - 0.091\ln\left(\frac{L}{d}\right) \qquad (21)$$

In expression (19), the first component is the friction resistance developed along the caisson wall and the second component is the resistance developed at the bottom of the caisson. Therefore N_f is called the friction factor and N_b is the bottom breakout resistance factor. $p_{u(drained)}$ is the drained pullout capacity and $s_{u(tip)}$ is the initial undrained strength at the tip of the caisson. T_k is a non-dimensional load rate parameter that can be defined as:

$$T_k = \frac{C_v}{vd} \qquad (22)$$

where, C_v is the coefficient of consolidation of the soil and v is the load rate (steady velocity) at which the caisson is pulled from the ground.

It was found that when $T_k > 0.6$, the pullout behaviour is effectively fully drained, so $N_b = 0$. When $T_k < 0.002$, the pullout behaviour can be considered as effectively undrained. The upper limit of N_b is determined by the undrained condition. Hence, equations (19) to (21) are applicable whenever $0.002 < T_k < 0.6$.

4.6 Lateral capacity – fully drained

Broms (1964) developed a simple but reliable method for the estimation of lateral capacity for suction caissons in sands. Broms' expression for the ultimate lateral capacity of a caisson embedded in saturated sand can be expressed as:

$$Q_u = \frac{\gamma' dL^3\tan^2(45^\circ + \frac{\phi}{2})}{2(D+L)} \qquad (23)$$

where Q_u is the ultimate lateral load, ϕ is the angle of internal friction of the sand, γ' is the submerged unit weight of the sand, L is the embedded depth of the caisson, d is its diameter, and D is the distance from the point of application of the lateral force to the sand surface.

5 EVALUATION OF THEORETICAL SOLUTIONS

The theoretical solution for inclined uplift capacity can be verified by the solutions obtained by an upper bound technique using the theory of soil plasticity (Randolph et al, 1998). The theoretical solutions discussed above can also be examined in the light of available experiment data. The published experimental data include laboratory model-scale and centrifuge results, as well as some field test results. The test results include 76 individual tests from 12 independent studies, which are detailed in Deng & Carter (1999c).

5.1 Evaluation By Upper Bound Techniques

A three-dimensional upper bound technique was developed by Randolph et al (1998), based on the flow mechanism proposed by Murff & Hamilton (1993). This technique is focused on the capacity of suction caissons subjected to quasi-horizontal loading form a mooring chain. This method was extended to include quasi-vertical loading of suction caissons for TLP anchorages by Deng & Carter (1999b) by adopting the uplift capacity factor, as shown in expression (9), when considering the energy dissipation at the caisson tip due to soil movement in the vertical direction. Detailed comparison between the theoretical solution for inclined uplift capacity and the upper bound techniques can be found in Deng & Carter (1999b).

It was found that the differences between the theoretical solutions and the upper bound solutions for cases where the load is applied vertically along the sidewall are less than 5% for most aspect ratios. While for cases where purely horizontal load is applied the differences are 1% to 20%. A detailed comparison for inclined load cases is given in Fig.9.

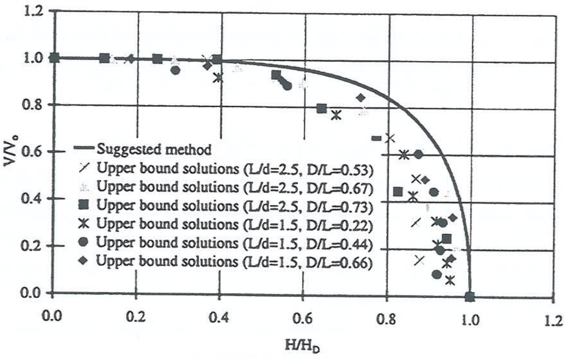

Figure 7 Comparison of theoretical solutions and upper bound solutions

As shown in Fig.9, the upper bound solutions for the ultimate loads are generally lower than the theoretical solutions. The reasons for this are likely due to:

- The theoretical solutions were derived from the finite element predictions. Generally finite element results tend to be over-estimates and it was also assumed in the finite element analysis that the interface between the soil and the caisson is perfectly bonded.
- The upper bound method is not entirely rigorous and has some limitations. One of the limitations is that this upper bound solution is not kinematically complete, since it ignores interaction between the top and the bottom of the 'flow' region (below the conical wedge) and the adjacent soil.

5.2 Evaluation By Test Data

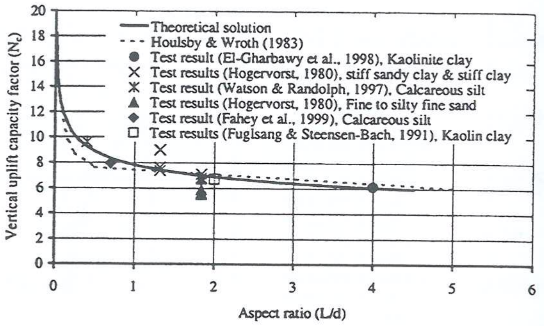

Figure 8 Comparison of theoretical solution and test data for undrained vertical uplift capacity (after Deng & Carter, 1999c)

Detailed evaluation can be found in Deng & Carter (1999c). Figures 10, 11, 12 and 13 show comparisons of the test results and the theoretical solutions for vertical uplift capacity (undrained), lateral capacity (undrained), inclined uplift capacity (undrained) and vertical uplift capacity (drained) respectively. From the comparisons, the following conclusions can be drawn.

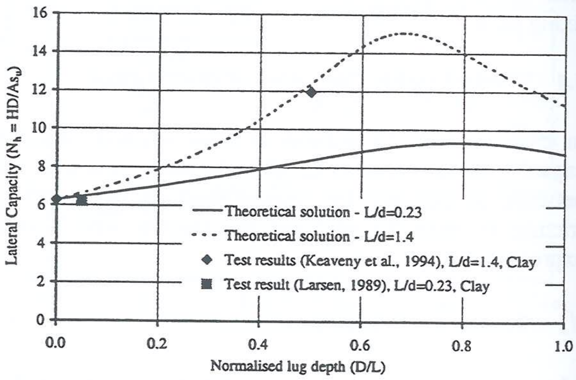

Figure 9 Comparison of theoretical solution and test data for undrained lateral capacity (after Deng & Carter, 1999c)

- The suggested theoretical method for estimating the undrained vertical capacity of suction caissons seems to apply reasonably well to a wide range of soils, including stiff clay, kaolinitic clay, sandy clay, calcareous silt, fine sand, and silty fine sand.

For estimation of the lateral capacity of suction caissons, the suggested theoretical solution is reasonable for cases where the subsoil is clay or has significant cohesion and deforms under undrained conditions.

- The suggested theoretical method for the estimation of the inclined uplift capacity of suction caissons under undrained conditions appears to be reasonably accurate for design purposes.

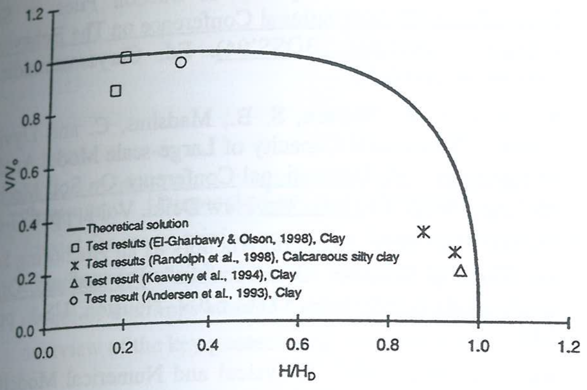

Figure 10 Comparison of theoretical solution and test data for undrained inclined uplift capacity (after Deng & Carter, 1999c)

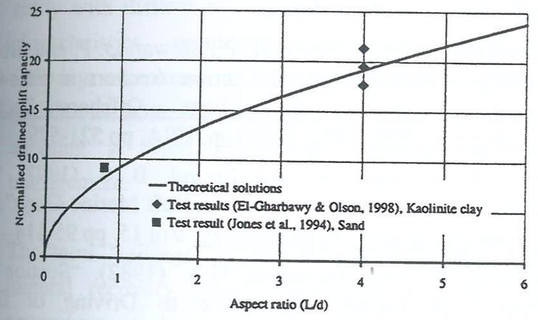

Figure 11 Comparison of theoretical solution and test data for drained vertical uplift capacity (after Deng & Carter, 1999c)

Six tests included measurement of the drained lateral capacity. The comparison shows that the differences between the prediction of Broms' formula and the measured lateral capacity are between 0.0 ~ 21.7%, which indicates that Broms' formula appears to be reliable for cases where the subsoil is sand or noncohesive soil. Twenty-three individual test results were recorded for vertical uplift capacity (partially drained). The differences between test results and estimation by expression (19) are 2.3 ~ 23.3% for aspect ratio L/d = 0.75, 4.5 ~ 10.8% for L/d = 1.0, 4.3 ~ 24.7% for L/d = 1.5, 10.7 ~ 47.7% for L/d = 2.0 and 114.6% for L/d = 4.0. It may be seen that for aspect ratios less than 2, the test results agree reasonably well with the theoretical solutions. It is also noted that the theoretical solutions generally overestimate the measured capacities. However, for those tests with an aspect ratio larger than 2, the theoretical solutions generally significantly overestimate the measured responses. Use of the suggested formula (19) in design together with an appropriate factor of safety is therefore recommended for aspect ratio less than 2 only.

6 CONCLUSIONS

In this paper, theoretical solutions for estimation of the uplift capacity of suction caissons, for cases where the subsoil is drained, partially drained or undrained and the load is applied horizontally, vertically or inclined, have been presented. These methods have been verified by the results from a relatively large number of laboratory and field tests and solutions obtained from an upper bound technique using the theory of soil plasticity. The following conclusions were reached.

- Different failure mechanisms will be developed when the point of load application, the angle of load application or the subsoil's drainage condition is different.

- The suggested theoretical method for estimating the undrained vertical capacity of suction caissons, equations (8) and (9), seems to apply reasonably well to a wide range of soils, including stiff clay, kaolinitic clay, sandy clay, calcareous silt, fine sand, and silty fine sand.

- For estimation of the lateral capacity of suction caissons, the use of equation (13) and (14) is suggested for cases where the subsoil is clay or has significant cohesion and deforms under undrained conditions. The use of Broms' formula, equation (25), is suggested for cases where the subsoil is sand or is non-cohesive and deforms in a fully drained manner.

- The suggested theoretical method for the estimation of the inclined uplift capacity of suction caissons under undrained conditions, equations (17) and (18), appears to be reasonably accurate for design purposes.

- The suggested formula for the estimation of drained vertical uplift capacity, equation (20), can be used for a wide range of aspect ratios and for various kinds of soils.

- The suggested formulae for estimating the vertical uplift capacity for partially drained conditions, equations (21) to (23), appear to be reasonably accurate only for those cases where the aspect ratio of the caisson is less than 2.

- It should also be noted that the theoretical methods for the estimation of uplift capacity of suction caissons presented in this paper consider only two-sided (anti-symmetric) failure mechanisms.

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