Proc. Fourth Australia New Zealand Young Geotechnical Professionals Conference, House & Watson (eds), February 2000, Perth, Australia

The Response of Suction Caissons to Catenary Loading

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Abstract

Research into the performance of suction caissons has developed in response to the demand from the offshore hydrocarbon industry for a versatile foundation solution capable of anchoring a range of alternative structures. Suction caissons are capable of providing large anchoring capacities in all directions. The simple installation procedure and high reliability has seen suction caissons employed in a range of water depths and within hydrocarbon fields from marginal to high potential. As exploration is directed toward increasing water depths, anchoring demands on the proposed structures become greater and subsequently a more detailed understanding of the limitations to caisson capabilities and performance is required.

Using the fixed beam geotechnical centrifuge facility at The University of Western Australia (UWA), the installation and response of a dimensionally scaled prototype caisson to inverse catenary chain loading was modelled with the objective of establishing a relationship between the caisson geometry, soil characteristics and the monotonic holding capacity. The installation and tensile resistances were recorded to determine the necessary installation pressures and uplift capacity of the caisson. Theory suggests that the lateral capacity is dependent upon the frictional resistance between the caisson and soil, which may be back derived through calibration of the theoretical and experimental response of the caisson to axial loading.

This paper presents the data from a series of centrifuge tests, comparing the results with the theoretical monotonic capacity of laterally loaded caissons. A smooth walled model caisson was installed and subsequently loaded with an anchor chain in normally consolidated kaolin clay. The data exhibited excellent repeatability between identical tests and a similar correlation with the adopted upper-bound plasticity solution for laterally loaded caissons.

1 INTRODUCTION

The versatility and cost effectiveness of suction caisson foundations has initiated significant research into the capabilities of and limitations to their applications.

In moderate water depths, suction caissons may be used within clusters as a mooring for such structures as floating, production, storage and offloading (FPSO) facilities. For these applications each caisson is attached to the structure with a chain that forms an inverse catenary profile within the soil between the mudline and the point of attachment, imposing a predominantly horizontal load on the anchor.

The first catenary moored structure using suction caisson foundations was at the Gorm field offshore Denmark (Senpere and Auvergne, 1982). The soil profile at Gorm comprises dense, fine sand overlying soft clay above stiff clay, proving the suitability of suction caissons in a diversity of soil types. Most recently, the first suction caisson installations within calcareous soils have been undertaken in the calcareous silty sediments of the Timor Sea for the Laminaria hydrocarbon field (Schrøder and Finnie, 1999).

As offshore hydrocarbon exploration exploits deeper waters, a greater understanding of the combined axial and lateral capacity of suction caissons is warranted.

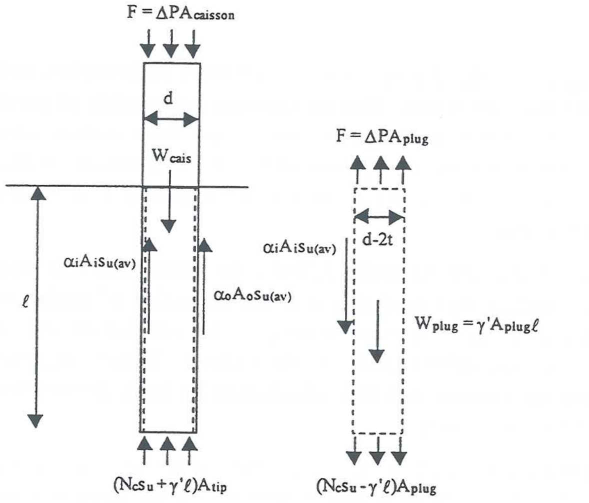
Very little experimental research has been published on the response of suction caissons to lateral loading. Experimental work in progress at The University of Western Australia has the objective of developing a design methodology capable of specifying an optimal caisson geometry for a given soil profile and design load configuration. This research involves the experimental modelling of suction caissons (of various geometries) subjected to loads ranging form purely horizontal to purely vertical in a range of typical offshore soil profiles. This paper presents the results of the first series of centrifuge tests on a caisson monotonically loaded within normally consolidated kaolin clay.

2 BACKGROUND

2.1 Installation

Of concern during the installation of suction caissons is whether the caisson will reach the target installation depth before upheaval of the internal soil plug. Soil plug failure will not be further discussed in this paper since the aspect ratio of the model caisson and the adopted soil profile suggest a limiting aspect ratio well in excess of the model geometry, (House *et al.*, 1999). Furthermore, the adopted experimental installation method (jacked) eliminates the likelihood of soil plug upheaval due to the absence of the uplift force on the plug experienced during suction penetration.

A free body diagram of the caisson and internal soil plug during suction installation is shown in Figure 2.1.



(a) Caisson (b) Soil plug

Figure 2.1 Free body diagram of caisson and soil plug

The required installation pressure is derived from the free body diagram of Figure 2.1(a) and is defined by

$$\Delta P_{caisson} = \frac{(N_c s_u + \gamma'\ell)A_{tip} + \alpha_i \overline{s_u} A_i + \alpha_o \overline{s_u} A_o - W}{A_{plug}} \quad (1)$$

where:
N_c = bearing capacity factor (taken as 9)
s_u = undrained shear strength (at caisson tip)
γ' = bulk density
ℓ = embedded caisson length
A_{tip} = tip area of caisson
A_{plug} = sectional area of soil plug
A_i = internal area of caisson in contact with soil
A_o = external area of caisson in contact with soil
α_i = internal friction factor
α_o = external friction factor
W = submerged caisson weight

2.2 Lateral capacity

A least upper-bound to the undrained collapse load of a laterally loaded caisson in clay may be predicted using theory discussed in detail by Murff and Hamilton (1993) and refined for suction caisson analyses by Randolph *et al.* (1998). The iterative method (described below) varies three geometric parameters (defining the failure mechanism) and one optimisation parameter to solve for the minimum collapse load satisfying the kinematic constraints of the adopted failure mechanism.

(a) Assume a failure mechanism (geometry)

(b) Identify velocity field for failure mechanism

(c) Derive function between plastic strain rate and energy dissipation

(d) Set external work done by imposed loads to total plastic energy dissipation

(e) Repeat steps (a) to (d) until minimum collapse load is determined

A schematic representation of the proposed kinematic failure mechanism is shown in Figure 2.2. The geometric parameters are the depth of the failure wedge, radial extent of the failure wedge and depth to the centre of rotation.

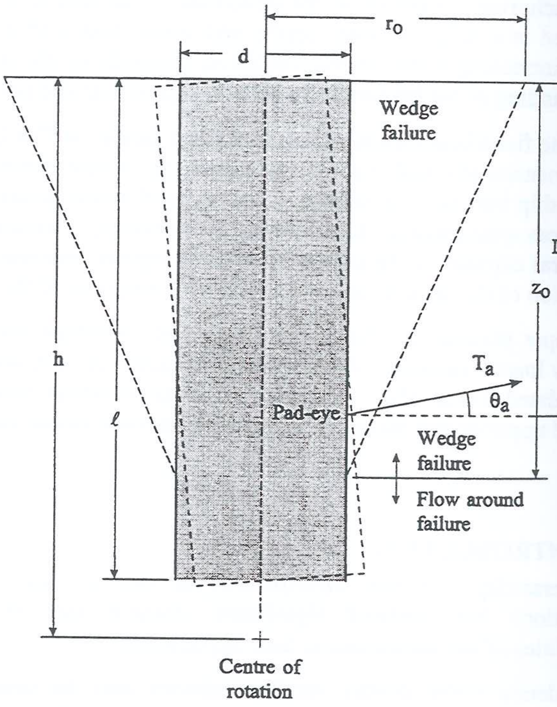
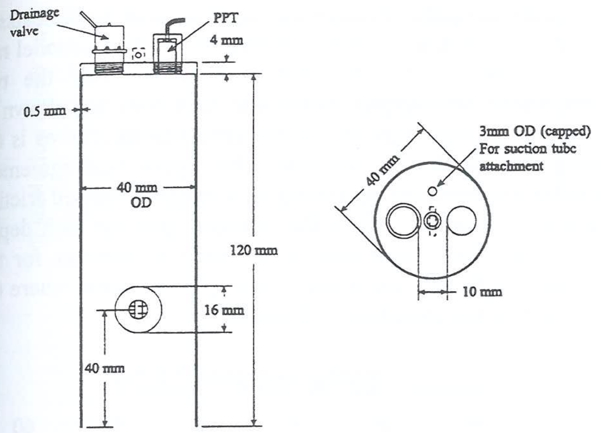


Figure 2.2 Upper bound failure mechanism

Subsequent to the experimental soil characterisation tests and using a mobilised friction coefficient calculated from the caisson installation data, the least upper bound to the caisson holding capacity is predicted using the aforementioned model.

3 EXPERIMENTAL MODELLING

A model suction caisson was fabricated in the UWA workshop from 6061 T6 aluminium and tested in the centrifuge within a sample comprising normally consolidated kaolin clay. The caisson had a dry mass of 32.3 g excluding attachments (representing a prototype submerged weight of approximately 550 kN), with a wall thickness of 0.5 mm and a stiffened region at the padeye of 1 mm wall thickness. The geometry of the model suction caisson is identified in Figure 3.1.



(a) Elevation (b) Plan

Figure 3.1 Geometry of model suction caisson

The normally consolidated kaolin sample was prepared by self-weight consolidation of a clay slurry on the UWA fixed beam geotechnical centrifuge (Randolph *et al.*, 1991). Commercially available dry kaolin clay powder was mixed to a slurry with a water content of 120 % (twice the liquid limit) and subsequently de-aired under a vacuum of approximately 100 kPa. A slurry depth of 280 mm was placed over a 10 mm sand drainage layer at the base of the strongbox and consolidated under an accelerated self-weight (120 g) for a period of approximately 2 days. Previous characterisation of the same clay at UWA (Stewart, 1991) suggested a fully consolidated model depth of 180 mm would be achieved with a target strength gradient (prototype scale) of approximately 1.1 kPa/m. Key properties of the kaolin clay are detailed in Table 1.

Soil characterisation tests were undertaken using the T-bar penetrometer (Stewart and Randolph, 1991) before each caisson installation associated with a catenary load test. The T-bar was penetrated and extracted at model rates of 3 mm/sec and 1 mm/sec respectively.

PROPERTY	VALUE
Specific gravity, G_s	2.60
Liquid limit, LL (%)	61
Plastic limit, PL (%)	27
s_u/σ'_v (Normally consolidated)	0.187
Consolidation coefficient, c_v (m^2/year)	1.3
Clay density, γ'_c (kN/m^3)	5.9

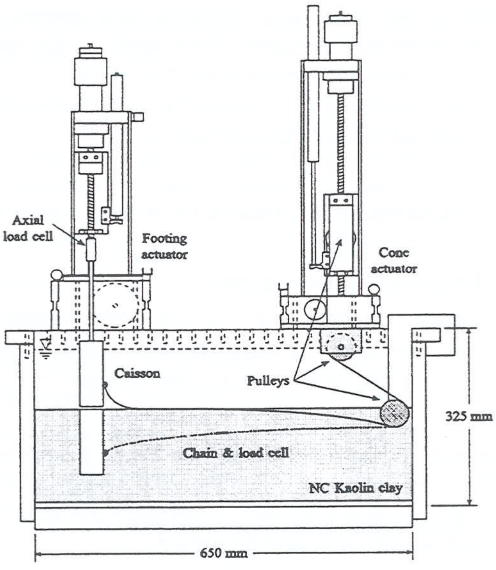
Table 1 Kaolin clay properties, after Stewart (1991)

The undrained shear strength of the normally consolidated kaolin clay samples was determined using the T-bar penetrometer (Stewart and Randolph, 1991). The T-bar has a load cell attached to the shaft above the tip from which the installation resistance is directly determined. The bearing pressure, q, is derived by dividing the installation resistance by the bar area and the shear strength is subsequently estimated by dividing the bearing pressure by a bar factor N_b.

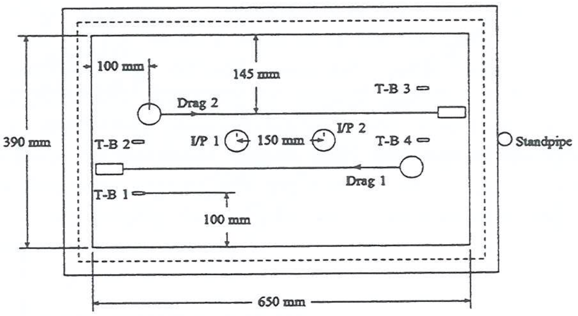
$$s_u = \frac{q}{N_b} \tag{2}$$

For the bar used it is standard convention to adopt an N_b value of 10.5, based on an average between the theoretical upper and lower bound plasticity solutions for a perfectly smooth (adhesion factor, $\alpha = 0$, $N_b = 9.14$) and a perfectly rough bar ($\alpha = 1$, $N_b = 11.94$), (Randolph and Houlsby, 1984).

The geometry of the experimental arrangement limited the modelling to two catenary load tests and a maximum of two installation / pull-out tests per strongbox. An elevation and plan schematic of the experimental arrangement is shown in Figure 3.2.



(a) Elevation



(b) Plan

Figure 3.2 Experimental arrangement

Each installation commenced with the caisson submerged and suspended above the mudline. The loading arm attached to the caisson was lowered into the sample with the drainage valve on the lid of the caisson open. Installation was undertaken at a model rate of 0.5 mm/sec. An axial load cell

(3 kN model capacity) attached to the loading arm measured the penetration resistance throughout the duration of the jacked installation. The anchor chain was pinned alongside the anchor wall throughout the penetration phase. A miniature pore pressure transducer (PPT) attached to the lid of the caisson recorded the internal caisson pressure.

After the caisson was installed to the target depth, the centrifuge was stopped, the chain unclipped and attached to the pulley arrangement and the drainage valve on the caisson lid was closed. The loading arm attached to the caisson was removed before the centrifuge was ramped back up to the target normal acceleration and the sample allowed to re-consolidate.

The catenary (inverse) load tests were performed by vertically displacing the actuator associated with the chain and pulley arrangement. The pulleys were geared such that one unit of vertical displacement of the actuator represented a chain displacement of 2 units. A chain drag rate of 0.2 mm/sec (monotonic) was adopted to maintain load control yet also ensure an undrained response. The depth of the end pulley was set such that the catenary profile of the anchor chain agreed with the solutions of Neubecker and Randolph (1995), for the predicted ultimate lateral capacity.

4 EXPERIMENTAL RESULTS & DISCUSSION

4.1 Sample characterisation

The normally consolidated kaolin clay sample was characterised by two T-bar tests before each of the two anchor tests. The undrained shear strength profiles are shown in Figure 4.1.

A comparison of the first (T-bars 1 & 2) and second series (T-bars 3 & 4) of T-bar tests highlighted evidence of a strength increase of approximately 40 % over the duration of the first anchor test and re-consolidation period.

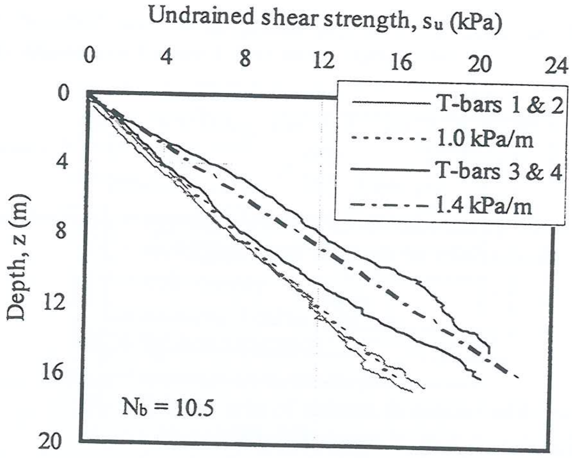


Figure 4.1 Undrained shear strength

The variability between the profiles of T-bars 3 and 4 suggests that the sites may have been subjected to soil disturbance upon extraction of the embedded end pulley used for monotonic drag test 1. T-bar pull out tests (immediately after penetration) showed a remoulded shear strength approximately 75 % that of the peak undrained shear strength.

4.2 Installation and axial loading

To ensure adequate drainage through the valve on the caisson lid, installation tests were undertaken at a constant model rate of 0.25 mm/s. The penetration resistances for the two installations preceeeding monotonic load tests are shown in Figure 4.2. Superimposed on the experimental curves is the theoretical installation response which gave best agreement with the experimental data using an average mobilised friction ratio of 0.3, and k is the shear strength gradient with depth. Note that minor corrections were made to account for the buoyancy effects in the initial stages of installation where the caisson was not completely submerged.

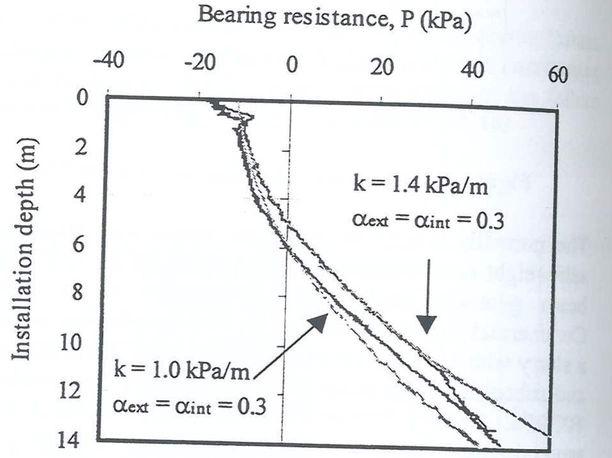


Figure 4.2 Installation resistances

The penetration resistance and internal caisson pressure rose sharply upon contact of the internal soil plug with the top cap of the anchor. Penetration was stopped at this point which for both installations was approximately 95 % of the target installation depth. Since the installation method was one of jacking, the internal soil plug upheaval may only be attributable to soil displaced by the caisson skirts.

Following the second monotonic load test, one installation / pull-out test was undertaken. The pull-out test was performed with a drained top cap at a constant rate (model) of 0.25 mm/sec. The experimental response is shown in Figure 4.3.

The caisson pull-out response is classically frictional, suggesting that the caisson-soil interface cohesion was mobilised with no contribution of reverse end bearing. The pressure response within the lid of the caisson showed that no negative pressures were developed between the top of the caisson and the internal soil plug, proving that the soil plug was not extracted with the caisson, verifying the expectations for a drained pull-out test.

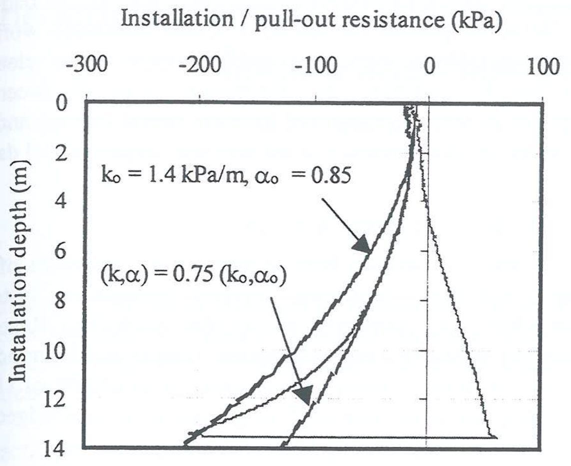


Figure 4.3 Axial pull-out response

The theoretical pull-out capacity is superimposed on the experimental data, showing that the experimental performance experiences a transition as the skirt friction apparently reduces quadratically. Best agreement is found by application of a reduction factor not only on the shear strength (as experienced in the T-bar pull-out tests) but inexplicably also on the mobilised friction ratio.

4.3 Inverse catenary loading

The load development response of the suction caissons subjected to monotonic chain loading is presented in Figure 4.4. Load development is slow as the initial slack in the chain was taken up. Post-peak behaviour exhibits significant strain softening of the kaolin clay.

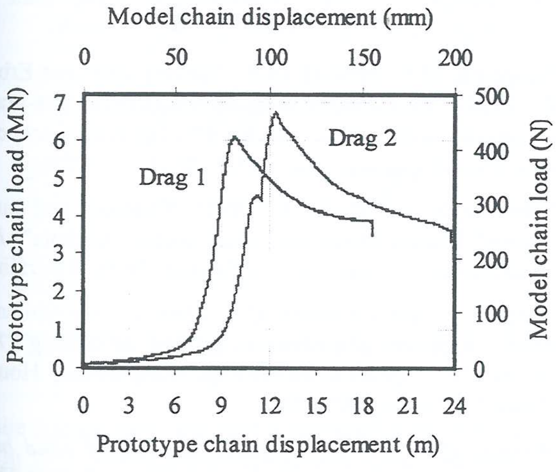


Figure 4.4 Load development (monotonic loading)

The pressure inside the lid of the caisson was monitored as the monotonic chain load was applied. It is observed that no significant pressure response was experienced up to the point of peak holding capacity, beyond which the differential pressure development is a response to the passive suction caused by caisson displacements.

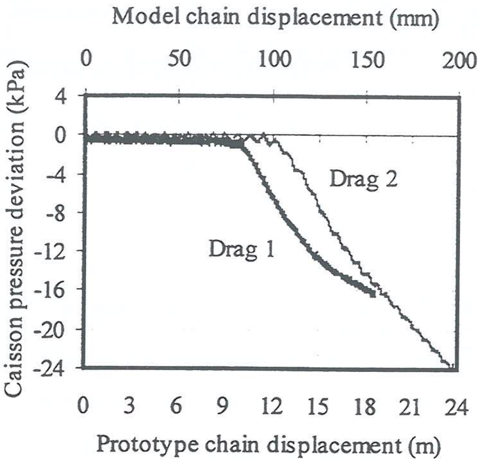


Figure 4.5 Pressure response (monotonic loading)

The total caisson displacement was approximately 1.5 and 2 diameters for drag tests 1 and 2 respectively.

Using the mobilised friction coefficient back derived from the installation data, the predicted least upper bound to the caisson holding capacity (2 sided failure mechanism) for each of the monotonic load tests is detailed in Table 2.

PARAMETER	TEST 1	TEST 2
Strength gradient, k (kPa/m)	1.0	1.4
Adhesion factor, α	0.3	0.3
Ultimate capacity, P_{ult} (MN)	4.7	6.7
Depth to centre of rotation, h (m)	14.4	14.4
Depth of soil wedge, z_o (m)	11.2	13.2
Radius of soil wedge, r_o (m)	10.1	11.0

Table 2 Predicted caisson holding capacity

Good agreement is observed between the experimental and predicted capacity for Test 2, although for Test 1 the theoretical model under-predicted the measured ultimate holding capacity by approximately 20 %. One possible reason for this under-prediction is that the sample may have experienced a strength increase over the period of reconsolidation between the caisson installation and lateral load tests. The reconsolidation period would also serve to increase the mobilised friction ratio, although as discussed in the following section this has a very minor influence on the predicted holding capacity for the experimental arrangement studied.

5 PARAMETRIC STUDY

To observe the sensitivity of the holding capacity model to the soil strength parameters and proportion of mobilised friction, a series of parametric analyses were undertaken using the experimental caisson geometry subjected to a purely lateral load. For normally consolidated soils (s_{uo} = 0 kPa) of various strengths, the predicted least upper bound to the caisson capacity is shown in Figure 5.1 for a two sided failure mechanism and a range of friction ratios.

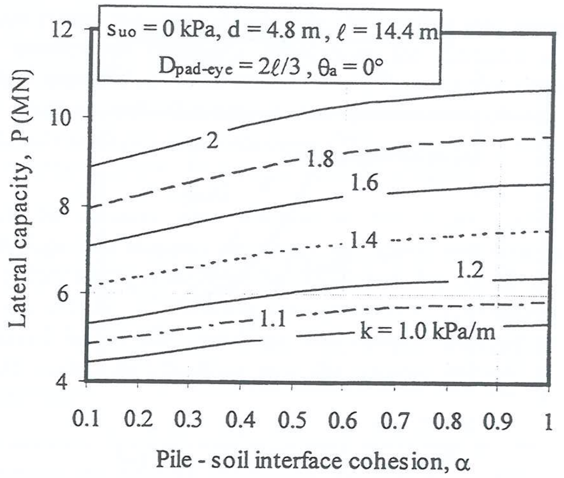


Figure 5.1 Influence of shear strength gradient

Similarly, the influence of mudline shear strength, s_{uo}, on the ultimate holding capacity of the same caisson within a 'typical' soil is shown in Figure 5.2.

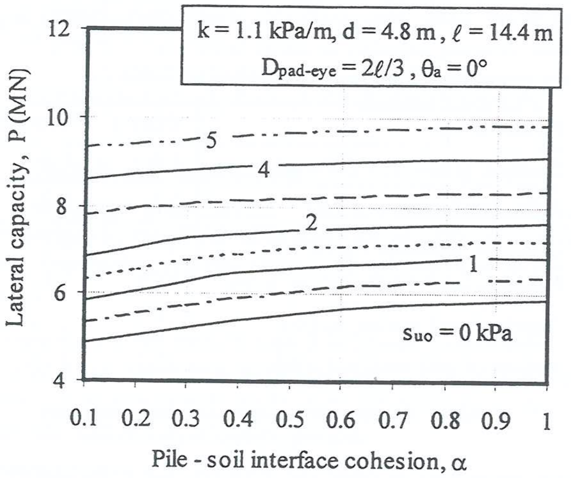


Figure 5.2 Influence of mudline shear strength

From the above two figures, it is apparent that for the experimentally modelled caisson, the predicted upper bound to the holding capacity is relatively insensitive to the caisson-soil interface cohesion ratio, α.

6 CONCLUSIONS AND FURTHER WORK

The results presented herein support the theory that the least upper bound plasticity solution provides a good prediction to the holding capacity of suction caissons subjected to quasi-lateral loading. Further refinement of the model will facilitate the prediction of caisson performance when subjected to load inclinations typical of those experienced when suction caissons are used for taut wire or fibre rope moorings.

More data are required on the performance of suction caissons in a range of soil profiles before a design methodology may be developed to assist in the optimal suction anchor selection for a specific requirement.

Kinematic data are essential in identifying the optimal pad-eye attachment depth on the anchor. Other proposed work at UWA includes a series of 3-dimensional finite element analyses to calibrate the theoretical load displacement response of caissons subjected to quasi-lateral loading and the predicted failure geometry of the soil with experimental data.

7 ACKNOWLEDGEMENTS

The research described here is part of the activities of the Special Research Centre for Offshore Foundation Systems, established and supported under the Australian Research Council's Research Centres Program. The author is supported by an Australian Postgraduate Award. The efforts of UWA workshop and centrifuge staff are gratefully acknowledged.

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